

ICE Manual of Bridge Engineering

THIRD EDITION

Edited by
Gerard Parke
and
Nigel Hewson

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Preface to the third edition

The success of the first two editions of this book has enabled the editors to offer the individual chapter authors the opportunity to enhance their previous contributions and, where relevant, to revise their work to comply with the latest codes of practice and best practice. The opportunity has also been taken to expand and clarify certain areas of the text and to include two new chapters; one on temporary works design and one on bridge aesthetics.

The 12 years that have passed since the second edition have seen some spectacular headlines in the world of bridges, from the highs of building bigger and longer, such as the 1014m cable-stayed span of the Russky Bridge and the 2023m suspension span of the Çanakkale 1915 Bridge, to the lows of the Viadotto Polcevera and Florida International University pedestrian bridge collapses. Less spectacular, but with just as great an impact, the widespread use of sophisticated computer software programmes is now taken for granted, changing the way that bridges are designed, constructed and maintained.

This manual aims to provide lucid explanations for both engineering students and practising bridge design engineers, offering a sound background together with advanced knowledge on the design of various bridge types, along with their subsequent inspections and economic and sustainable maintenance.

The editors would like to thank all of the individual contributors to this third and enriched edition for their expertise and time so freely given. The editors also wish to thank the publishers of this text, ICE Publishing, the publishing division of the Institution of Civil Engineers, for their continued support in collating this work.

This third edition is dedicated to two of the original editors of the first edition, namely Professor John Harding and Dr Michael Ryall, both of whom had considerable experience and knowledge in the design of bridge structures.

Gerard Parke, University of Surrey

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Chapter 1

The history and aesthetic development of bridges

D Bennett David Bennett Associates

This chapter on the history and aesthetic development of bridges looks at the evolution and progress of bridges from their earliest conception by humans. Following a timeframe from the Palaeolithic period to the present, all the various materials employed in construction are examined in relation to bridge development. Aesthetic design in bridges – especially in the twentieth century – is looked at in detail and the chapter ends with an essay on the search for aesthetic understanding in bridge design.

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The early history of bridges

The age of timber and stone

The bridge has been a feature of human progress and evolution ever since the first hunter-gatherers became curious about the fertile land, animals and fruit flourishing on trees on the other side of a river or gorge. Early humans also had to devise ways to cross streams and deep gorges to survive. A boulder or two dropped into a shallow stream works well as a stepping stone, as many of us have discovered, but for deeper flowing streams a tree dropped between banks is a more successful solution. So the primitive idea of a simple beam bridge was born.

Today, in the forests of Peru and the foothills of the Himalayas, crude rope bridges span deep gorges and fast-flowing streams to maintain pathways from village to village for hill tribes. Such primitive rope bridges evolved from the vines and creepers that early humans would have used to swing through the forest and to cross streams. Here is the second basic idea of a bridge – the suspension bridge.

For thousands of years during the Palaeolithic period, which lasted to around 10000 BC, we know that humans were living as nomads, hunting and gathering food. Slowly it dawned on early humans that instead of following herds of deer or buffalo, or foraging for plant food haphazardly, things could be better managed if the animals were kept in herds nearby and plants were grown and harvested in fields.

In this period, the simple log bridge served many purposes. It needed to be sufficiently broad and strong to take cattle, a level and solid platform to transport food and other materials, and movable so that it could be withdrawn to prevent enemies from using it. Narrow tree trunk bridges were inadequate and were replaced by double log beams spaced wider apart, on which short lengths of logs were placed and tied down to create a pathway. The pathways were planed by sharp scraping tools and any gaps between them plugged with branches and earth to

create a level platform. For crossings over wide rivers, support piers were formed from piles of rocks in the stream. Sometimes stakes were driven into the riverbed to form a circle and then filled with stones, creating a crude cofferdam. Around 4000 BC, early Bronze Age ‘lake dwellers’ were living in timber houses built out over the lakes, in the area that is now Switzerland. To ensure that their houses did not sink early, humans evolved ways to drive timber piles into the lake bed. From this discovery came the timber pile and the trestle bridge.

Primitive bridges were essentially post and lintel structures, either made from timber or stone or a combination of both. Sometime later, the simple rope and bamboo suspension bridge was devised; these developed into the rope suspension bridges that are in regular use today in the mountain reaches of China, Peru, Columbia, India and Nepal.

It took humans until 4000 BC to discover the secrets of arch construction. In the Tigris–Euphrates valley the Sumerians began building with adobe – a sun-dried mud brick – for their palaces, temples, ziggurats and city defences. Stone was not plentiful in this region and had to be imported from Persia, so it was used sparingly. The brick module dictated the construction principles employed, to scale any height and to bridge any span. And through trial and error it was the arch and the barrel vault that was devised to build their monuments and grand architecture at the peak of their civilisation. The ruins of the magnificent barrel-vaulted brick roof at Ptsephon and the Ishtar Gate at Babylon, are a reminder of Mesopotamian skill and craftsmanship. By around 2475 BC, the Egyptians had also mastered the arch and used it frequently in constructing relieving arches and passageways for their temples and pyramids.

Without doubt, the arch is one of the greatest discoveries of humankind. The arch principle was the essential element in all building and bridge technology over later centuries. Its dynamic and expressive form gave rise to some of the greatest bridge structures ever built.

Earliest records of bridges

The earliest written record of a bridge appears to be of a bridge built across the Euphrates around 600 BC as described by Herodotus, the fifth century Greek historian. The bridge linked the palaces of ancient Babylon on either side of the river. It had 100 stone piers, which supported wooden beams of cedar, cypress and palm to form a carriageway 35 ft wide and 600 ft long. Herodotus mentions that the floor of the bridge would be removed every night as a precaution against invaders.

In China, it would appear that bridge building evolved at a faster pace than the ancient civilisations of Sumer and Egypt. Records exist from the time of Emperor Yao in 2300 BC on the traditions of bridge building. Early Chinese bridges included pontoons or floating bridges and probably looked like the primitive pontoon bridges built in China today. Boats called sampans about 30 ft long were anchored side by side in the direction of the current and then bridged by a walkway. The other bridge forms were the simple post and lintel beam, the cantilever beam and rope suspension cradles. Timber beam bridges, like those of Europe, were often supported on rows of timber piles of soft fir wood called 'foochow poles', so called because they were grown in Foochow. A team of builders would hammer the poles into the riverbed using a cylindrical stone fitted with bamboo handles. A short crosspiece was fixed between pairs of poles to form the supports that would carry timber boards, which were then covered in clay to form the pathway over the river.

In later centuries, Chinese bridge building was dominated by the arch, which they copied and adapted from the Middle East as they travelled the silk routes that opened during the Han dynasty, around 100 AD.

Through Herodotus, we learn about the Persian ruler Xerxes and the vast pontoon bridge he built, consisting of two parallel rows of 360 boats, tied to each other and to the bank and anchored to the bed of the Hellespont, which is the Dardanelles today. Xerxes wanted to get his army of 2 million men and horses to the other bank to meet the Greeks at Thermopylae. It took 7 days and 7 nights to get the army across the river. Sadly for Xerxes, his massive army was defeated at the Battle of Thermopylae in 480 BC, the remnants of which retreated back over the pontoon bridge to fight another day. The Persians were great bridge builders and built many arch, cantilever and beam bridges. There is a bridge still standing in Khuzestan at Dezful over the river Dez, which could date anywhere from 350 BC to 400 AD. The bridge consists of 20 voussoir arches, which are slightly pointed and has a total length of 1250 ft. Above the level of the arch springing are small semicircular spandrel arches, which give the entire bridge an Islamic look, hence the uncertainty of its Persian origins.

The Greeks did not build many bridges over their illustrious history, being a seafaring nation who lived on self-contained islands and in feudal groups scattered across the Mediterranean. They exclusively used post and lintel construction in evolving a classical order in their architecture, and built some of the most breathtaking temples, monuments and cities the

world has ever seen, such as the Parthenon, the Temple of Zeus, the cities of Ephesus, Miletus and Delphi, to name but a few. They were quite capable of building arches like their forbears the Etruscans when necessary. There are examples of Greek voussoir arch construction that compare with the Beehive tombs at Mycenae, such as the ruins of an arch bridge with a 27 ft span at Pergamon in Turkey.

The Romans

The Romans, on the other hand, were masters of practical building skills. They were a nation of builders who took arch construction to a science and high art form during their domination of Mediterranean Europe. Their influence on bridge building technology and architecture has been profound. They conquered the world as it was then known, built roadways, canals and cities that linked Europe to Asia and North Africa, and produced the first true bridge engineers in the history of humankind. The Romans understood that the establishment and maintenance of their empire depended on efficient and permanent communications. Building roads and bridges was therefore a high priority.

The Romans also realised, as did the Chinese in later centuries, that timber structures, particularly those embedded in water, had a short life, and were prone to decay, insect infestation and fire hazards. Prestigious buildings and important bridge structures were therefore built of stone. But the Romans had also learnt to preserve their timber structures by soaking timber in oil and resin, as a protection against dry rot, and coating them with alum, for fireproofing. They learnt that hardwood was more durable than softwoods, and that oak was best for substructure work in the ground and alder for piles in water, while fir, cypress and cedar were best for the superstructure above ground.

They understood the different qualities of the stone that they quarried. Tufa, a yellow volcanic stone, was good in compression but had to be protected from weathering by stucco – a limewash. Travertine was harder and more durable and could be left exposed, but was not very fire resistant. The most durable materials, such as marble, had to be imported from distant regions of Greece and even as far away as Egypt and Asia Minor (Turkey). The Romans' big breakthrough in material science was the discovery of lime mortar and pozzolanic cement, which was based on the volcanic clay that was found in the village of Puzzoli. They used it as mortar for laying bricks or stones and often mixed it with burnt lime and stones to create a waterproof concrete.

The Romans realised that voussoir arches could span farther than any unsupported stone beam, and would be more durable and robust than any other structure. The Romans always built semicircular arches, with the thrust from the arch going directly down on to the support pier. This meant that piers had to be large. If they were built wide enough, at about one-third of the arch span, then any two piers could support an arch without shoring or propping from the sides. In this way it was possible to build a bridge from shore to shore, one span at a

time, without having to form the entire substructure across the river before starting the arches. They developed a method of constructing the foundation on the riverbed within a cofferdam or watertight dry enclosure, formed by a double ring of timber piles with clay packed into the gap between them to act as a water seal. The water inside the cofferdam was then pumped out and the foundation substructure built within it. The massive piers often restricted the width of the river channel, increasing the speed of flow past the piers and increasing the scour action. To counter this, the piers were built with cutwaters, which were pointed to cleave the water so it would not scour the foundations.

The stone arch was built on a wooden framework built out from the piers and known as centring. The top surface was shaped to the exact semicircular profile of the arch. Parallel arches of stones were placed side by side to create the full width of the roadway. The semicircular arch meant that all the stones were cut identically and that no mortar was needed to bind them together once the keystone was locked into position. The compression forces in the arch ensured complete stability of the span. The Romans did build many timber bridges, but these have not stood the test of time, and today all that remain of their achievement after 2000 years are a handful of stone bridges in Rome, and a few scattered examples in France (see Figure 1.1), Spain, North Africa, Turkey and other former Roman colonies. But what still stand today, be they bridges or aqueducts, rank among the most inspiring and noble of bridge structures ever built, considering the limitations of Roman technology.



Figure 1.1 Pont du Gard, Nîmes

The Dark Ages and the brothers of the bridge

When the Roman Empire collapsed, it seemed that the light of progress around the world went out for a long while. The Huns, Visigoths, Saxons, Mongols and Danes did little building in their raids across Europe and Asia to plunder and destroy. It was left to the spread of Christianity and the strength of the Church to start the next boom in road building and bridge building, around 1000 AD.

It was the Church that had preserved and developed both spiritual understanding and the practical knowledge of building during this period. And, not surprisingly, bridge building, among many skills and crafts, became associated with it.

A group of friars of the Altopascio order near Lucca in northern Italy lived in a large dwelling called the Hospice of St James. The friars were skilled at carpentry and masonry, having built their own priory, no doubt helping with others. The surrounding countryside was wild and dangerous, and the refuge they built was a popular resting place for pilgrims and travellers using the ancient road from Tuscany to Rome. In 1244, Emperor Frederick II required that the hospice build a proper bridge across the White Arno for pilgrims and travellers. With their skills and practical knowledge, the friars set up a cooperative to build the bridge. After completing the bridge over the White Arno, their fame spread through Italy and France. This sparked off an interest in bridge building in other ecclesiastical orders. In France, a group of Benedictine monks established the religious order of the Frères Pontiffs (brothers of the bridge) to build a bridge over the Durance.

And so the 'brothers of the bridge' order became established among Benedictine monks, spreading from France to England by the thirteenth century. The purpose of the order, apart from its spiritual duties, was to aid travellers and pilgrims, to build bridges along pilgrimage routes or to establish boats for their use, and to receive them in hospices built for them on the bank. The brothers of the bridge were great teachers, who strove to emulate and continue the magnificent work of the Roman bridge builders.

The most famous and legendary bridge of this period was built by the Order of the Saint Jacques du Haut Pas, whose great hospice once stood on the banks of the Seine in Paris on the site of the present church of that name. They built the Pont Esprit over the Rhône but their masterpiece was the neighbouring bridge at Avignon. It was truly a magnificent and record-breaking achievement for its time. Its beauty has inspired writers, poets and musicians over the centuries. Sadly all that remains today at Avignon are just four out of the 20 spans of the bridge and the chapel where the supposed creator of the bridge was interred and later canonised as Saint Bénézet.

While the Pont d'Avignon was being built in France, another monk of the Benedictine order in England, Peter of Colechurch, was planning the building of the first masonry bridge over the Thames. A campaign for funds was launched with enthusiasm; it was not only the rich townspeople, the merchants and money lenders, who made generous donations, but also the common people of London gave freely. Until the sixteenth century, a list of donors could be seen hanging in the chapel on the bridge. The structure that was built in 1206 was Old London Bridge (see Figure 1.2) and ranks after the Pont d'Avignon in fame. It was such a popular bridge that buildings and warehouses were soon erected on it. It became so fashionable a location that the young noblemen of Queen Elizabeth's household resided in a curious four-storey timber building imported piece by piece from the Netherlands, called Nonsuch House.



Figure 1.2 Old London Bridge

Towns continued to sponsor and promote the building of stronger and better bridges and roads. They did not always get the brothers of the bridge to build them, because they were often committed to other projects for many years in advance. Instead, guilds of master masons and carpenters were formed and spread across Europe offering their services. Even government officials were united in this community enterprise and began to grasp the initiative and drive for better road and bridge networks across the country (Figure 1.3 shows an example of a medieval fortified bridge). Soon the vestiges of the Dark Ages and feudalism were transformed to the age of enlightenment and the Renaissance. The Ponte de Vecchio in Florence, built towards the end of this period, marks the turning point of the Dark Ages. It was a covered bridge erected in 1345, lined with jewellery shops and galleries, with an upper passageway

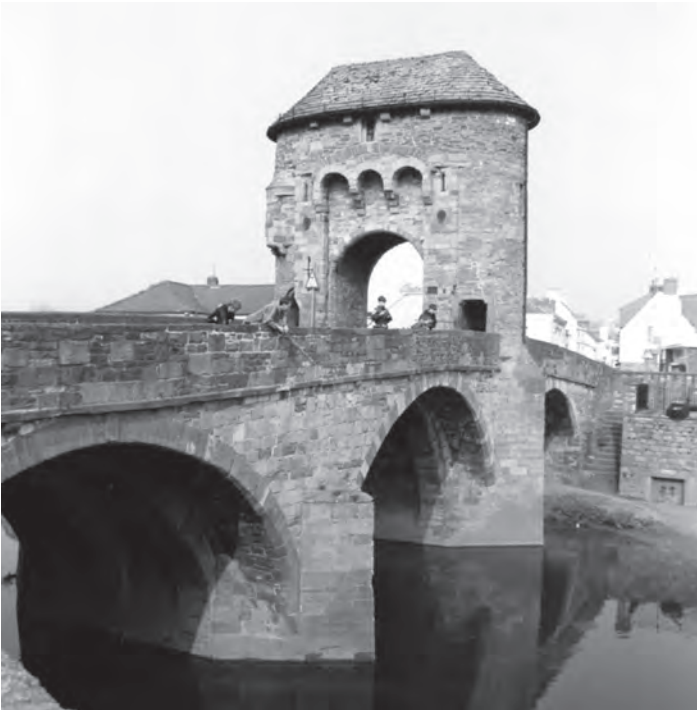


Figure 1.3 Monnow Bridge, Monmouth – an example of a medieval fortified bridge

added later, that was a link between the royal and government palaces, the Uffizi and Pitti Palaces. The piers, which are 20 ft thick, support the overhanging building as well as the bridge spans. The most innovative features of the bridge are the arch spans, which are extremely shallow compared with any previous arches ever built, or indeed many contemporary European bridges. Each span was built as a segmental arch; this is unusual for bridge builders of that period because they could not possibly have determined the thrust from the arches mathematically with the level of knowledge they possessed. How they achieved this is not known (as is also the case for the segmental arches of Pont d'Avignon). The architect of this radical design was Taddeo Gaddi, who had studied under the great painter Giotto, and was regarded as one of the great names of the Italian Renaissance.

The Renaissance

Not since the days of Homer, Aristotle and Archimedes in Hellenistic times have such great feats of discovery in science and mathematics, and such works of art and architecture been achieved, as during the Renaissance. Modern science was born in this period through the enquiring genius of Copernicus, Da Vinci, Francis Bacon and Galileo, and in art and architecture through Michelangelo, Brunelleschi and Palladio. During the Renaissance there was a continual search for the truth, explanations of natural phenomena, greater self-awareness and rigorous analysis of Greek and Roman culture. Bridge building, particularly in Italy, was regarded as a high art form. Much emphasis was placed on decorative order and pleasing proportions, as well as the stability and permanence of its construction. Bridge design was architect-driven for the first time, with Da Vinci, Palladio, Brunelleschi and even Michelangelo all experimenting with the possibilities of new bridge forms. The most significant contribution of the Renaissance was the invention of the truss system, developed by Palladio from the simple king post and queen post roof truss, and the founding of the science of structural analysis, with the first book ever written on the subject *Discorsi e Dimostrazioni Matematiche, intorno a due nuove scienze* (*Discourses and Mathematical Demonstrations Relating to Two New Sciences*), published by Galileo Galilei in 1638.

Palladio did not build many bridges in his lifetime; many of his truss bridge ideas were considered too daring and radical and his work lay forgotten until the eighteenth century. His great treatise *Four Books of Architecture*, published in 1520, in which he applied four different truss systems for building bridges, was destined to influence bridge builders in future years, when the truss replaced the arch as the principal form of construction. Bridge builders during the Renaissance were clever material technologists who were preoccupied with the art of bridge construction and how they could build with less labour and materials. It was a time of inflation, when the price of building materials and labour was escalating. The most famous bridge builders in this era were Ammannati, Da Ponte and Du Cerceau.

Which Renaissance bridge is the most beautiful: Florence's Santa Trinita, Venice's Rialto or Paris's Pont Neuf? Arguably the most famous and celebrated bridge of the Renaissance was the Rialto bridge designed by Antonio Da Ponte in Venice.

John Ruskin (1853) said of the Rialto: 'The best building raised in the time of the Grotesque Renaissance; very noble in its simplicity, in its proportions, and its masonry.' Its designer was 75 years old when he won the contract to build the Rialto, and was 79 when it was finished. It has a single segmental arch span of 87 ft 7 in, which rises 25 ft 11 in at the crown. The bridge is 75 ft 3 in wide, with a central roadway, shops on both sides and two small paths on the outside, next to the parapets. Two sets of arches, six each side of the large central arch, support the roof and enclose the 24 shops within it. It took three and half years to build and kept all the stonemasons in the city fully occupied in work for two of those years.

Equally innovative and skilful bridge construction was progressing across Europe. In the state of Bohemia, across the Moldau at Prague, was built the longest bridge over water, the Karlsbrücke in 1503, and the most monumental and imperial bridge of the Renaissance. It took a century and half to completely finish. It was adorned with statues of saints and martyrs and terminates on each bank with an imposing tower gateway. In France at this time, a fine example of the early French Renaissance, the Pont Neuf, was being designed (Figure 1.4). It was the second stone bridge to be built in Paris; although its design and construction did not represent a great leap forward in bridge building, it occupies a special place in Parisian hearts. Designed by Jacques Androuet Du Cerceau, the two arms of the Pont Neuf that join the Île de la Cité to the left and right banks was a massive undertaking. Although all the arches are semicircular and not segmental, no two spans are alike, as they vary from 31 to 61 ft in span and also differ on the downstream and upstream sides of each arch and were built on a skew of 10%. Du Cerceau wanted the bridge to be a true unencumbered thoroughfare, bereft of any houses and shops. But the people of Paris demanded shops and houses, which resulted in modification to the few short-span piers that had been constructed.

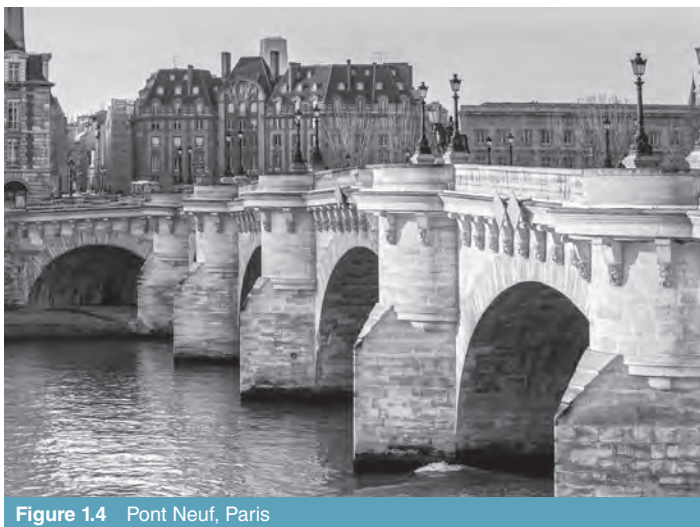


Figure 1.4 Pont Neuf, Paris

The Pont Neuf has stood now for 400 years and was the centre of trade and the principal access to and from the crowded island when it was built. The booths and stalls on the bridge became so popular that all sorts of traders used it, including booksellers, pastry cooks, jugglers and peddlers. They crowded the roadway until some 200 stalls and booths were packed into every niche along the pavement. The longer left bank half of the Pont Neuf was extensively reconstructed in 1850 to exactly the same details, after many years of repairs and attention to its poor foundations. The right bank half, with the shorter spans, has been left intact. The entire bridge has been cleared of all stalls and booths and is used today as a road bridge.

The finest examples of late French Renaissance bridges built during the seventeenth century are the Pont Royale and Pont Marie, both of which are still standing today. The Pont Royale (Figure 1.5) was the first bridge in Paris to feature elliptical arches and the first to use an open caisson to provide a dry working area in the riverbed. The foundations for the bridge piers were designed and constructed under the supervision of François Romain, a preaching brother from the Netherlands, who was an expert in solving difficult foundation problems. Both the bridge architect François Mansart and the builder Jacques Gabriel called on Romain after they ran into foundation problems. Romain introduced dredging in the preparation of the riverbed for the caisson, using a machine that he had developed. After excavations were finished, the caisson was sunk to the bed, but the top was kept above the water level. The water was then pumped out and the masonry work of the pier was built inside the dry chamber. The five arch spans of the Pont Royale increase in span towards the centre and, although it has practically no ornamentation, the bridge blends beautifully into its river setting and the bankside environment.



Figure 1.5 Pont Royale, Paris (courtesy of J Crossley)

The Renaissance brought improvements in both the art and science of bridge building. For the first time, bridges began to be regarded as civic works of art. The master bridge builder had to be an architect, structural theorist and practical builder, all rolled into one. Without doubt the finest exhibition of engineering skills in this era was the slender elliptical arched bridge of Santa Trinita in Florence, designed by Bartolomeo Ammannati

in 1567. Many scholars are still mystified to this day as to how Ammann arrived at such pleasing, slender curves to the arches.

Eighteenth-century bridge building

The Age of Reason

In this period, masonry arch construction reached perfection, owing to a momentous discovery by Perronet and the innovative construction techniques of John Rennie. Just as the masonry arch reached its zenith 7000 years after the first crude corbelled arch in Mesopotamia, it was to be threatened by a new building material – iron – and the timber truss, as the principal construction for bridges in the future.

This was the era when civil engineering as a profession was born, when the first school of engineering was established in Paris at the École de Paris during the reign of Louis XV. The director of the school was Gabriel, who had designed the Pont Royal. He was given the responsibility of collecting and assimilating all the scientific information and knowledge there was on the science and history of bridges, buildings, roads and canals.

With such a vast bank of collective knowledge, it was inevitable that building architecture and civil engineering should be separated into two fields of expertise. It was suggested that it was not possible for one man, in his brief life, to master the essentials of both subjects. Moreover, it also became clear that the broad education received in civil engineering at the Corps des Ponts et Chaussées at the École de Paris was not sufficient for the engineering of bridge projects. More specialised training was needed in bridge engineering. In 1747, the first school of bridge engineering was founded in Paris at the historic École des Ponts et Chaussées. The founder of the school was Trudaine, and the first teacher and director was a brilliant young engineer named Jean-Rudolphe Perronet.

Perronet has been called the father of modern bridge engineering for his inventive genius and design of the greatest masonry arch bridges of that century. In his hands, the masonry arch reached perfection. The arch he chose was the curve of a segment of a circle of larger radius, instead of the familiar three-centred arch. To express the slenderness of the arch, he raised the haunch of the arch considerably above the piers. He was the first person to realise that the horizontal thrust of the arch was carried through the spans to the abutments and that the piers, in addition to carrying the vertical load, also had to resist the difference between the thrusts of the adjacent spans. He deduced that if the arch spans were about equal and all the arches were in place before the centring was removed, the piers could be greatly reduced in size.

What remains of Perronet's great work? Only his last bridge, the glorious Pont de la Concorde in Paris, built when he was in his eighties. It is one of the most slender and daring stone arch bridges ever built in the world. 'Even with modern analysis,' suggests Professor James Finch, author of *Engineering*

and Western Civilization (1951), 'we could not further refine Perronet's design.'

With France under the inspired leadership of Gabriel and then Perronet, the rest of Europe could only admire and copy these great advances in bridge building. In England, a young Scotsman, John Rennie, was making his mark, following in the footsteps of the great French engineers. He was regarded as the natural successor to Perronet, who was a very old man when Rennie started his career. Rennie was a brilliant mathematician, a mechanical genius and a pioneering civil engineer. In his early years, he worked for James Watt to build the first steam-powered grinding mills at Albion Mills in London, and later designed canals and drainage systems to drain the marshy fens of Lincolnshire. He built his first bridge in 1779 across the Tweed at Kelso. It was a modest affair with a pier width-to-span ratio of 1:6 and a conservative elliptical arch span. Rennie picked up the theory of bridge design from textbooks and from studies and discussion about arches and voussoirs with his mentor John Robison of Edinburgh University. He designed bridges with a flat, level roadway rather than the characteristic hump of most English bridges. This was a radical departure from convention, and his bridges were much admired by all the townspeople, farmers and traders who transported material and cattle across them. The first bridge at Kelso was a modest forerunner to the many famous bridges that Rennie went on to build, namely Waterloo, Southwark and New London Bridge (Figure 1.6). What then was Rennie's contribution to bridge building? For Waterloo Bridge, the centring for the arches was assembled on the shore and then floated out on barges into position. So well and efficiently did this system work that the framework for each span could be put into position in a week. This was a fast erection speed and as a result Rennie was able to halve bridge construction time. So soundly were Rennie's bridges built that 40 years later Waterloo Bridge had settled only 5 in. Rennie's semi-elliptical arches, sound engineering methods and rapid assembly technique, together with Perronet's segmental arch, divided pier and understanding of arch thrust, changed bridge design theory for all time.



Figure 1.6 John Rennie's New London Bridge – under construction

The carpenter bridges

The USA, with its vast expansion of roads and waterways, following in the wake of commercial growth in the eighteenth century, was to become the home of the timber bridge in the nineteenth century.

The USA had no tradition or history of building with stone, so early bridge builders used the most plentiful and economical materials that were available: timber. The Americans produced some of the most remarkable timber bridge structures ever seen, but they were not the first to pioneer such structures. The Grubenmann brothers of Switzerland were the first to design quasi-timber truss bridges in the eighteenth century. The Wettingen Bridge over the Limmat just west of Zürich was considered their finest work. The bridge combined the arch and truss principle, with seven oak beams bound close together to form a catenary arch to which a timber truss was fixed. The span of the Wettingen was 309 ft, far exceeding any other timber bridge span.

Of course, numerous timber beam and trestle bridges were built in Europe and the USA. However, to bridge deep gorges, broad rivers and boggy estuaries, such as those that ran through North America, and support the heavy loads of chuckwagons and cattle, something more robust was needed. The answer, according to the Grubenmanns, was a timber truss arch bridge, but it was not a true truss.

Palmer, Wernwag and Burr, the so-called American carpenter bridge builders, designed more by intuition than by calculation and developed the truss arch to span farther than any other wooden construction. This was the third and last of the three basic bridge forms to be discovered. The first person who made the truss arch bridge a success in the USA and who patented his truss design was Timothy Palmer. In 1792, Palmer built a bridge consisting of two trussed arches over the Merrimack; it looked very like one of Palladio's truss designs, except that the arch was the dominant supporting structure. Palmer's 'Permanent Bridge' over the Schuylkill, built in 1806, was his most celebrated. When the bridge was finished, the president of the bridge company suggested that it would be a good idea to cover the bridge to preserve the timber from rot and decay in the future. Palmer went further than that and timbered the sides as well, completely enclosing the bridge. Thus, America's distinctive covered bridge was established. Enclosing the bridge stopped snow getting in and piling up on the deck, which would have caused it to collapse from the extra load.

Wernwag was a German immigrant from Pennsylvania, who built 29 truss-type bridges in his lifetime. His designs integrated the arch and truss into one composite structure rather more successfully than Palmer's. Wernwag's famous bridge was the Colossus over the Schuylkill just upstream from Palmer's 'Permanent Bridge'; this was composed of two pairs of parallel arches, linked by a framing truss, which carried the roadway. The truss itself acted as bracing reinforcement and consisted of heavy verticals and light diagonals. The diagonal elements were remarkable because they were iron rods, and were the first iron rods to be used in a long-span bridge. In its day, the

Colossus was the longest wooden bridge in the USA, having a clear span of 304 ft. Fire destroyed the bridge in 1838. It was later replaced by Charles Ellet's pioneering suspension bridge.

Theodore Burr was the most famous of the illustrious triumvirate. Burr developed a timber truss design based on the simple king and queen post truss of Palladio. He came closest to building the first true truss bridge; however, it proved unstable under moving loads. Burr then strengthened the truss with an arch. It is significant that here the arch was added to the truss rather than the other way round. Burr arch trusses were quick to assemble and modest in cost to build, and for a time were the most popular timber bridge form in the USA.

By 1820, the truss principle had been well explored and, although the design theory was not understood in practice, it had been tested to the limit. It was left to Ithiel Town to develop and build the first true truss bridge, which he patented and called the Town lattice. It was a true truss because it was free from arch action and any horizontal thrust. It was so simple to build that it could be nailed together in a few days and cost next to nothing compared with other alternatives. Town promoted his timber structure with the slogan 'built by the mile and cut off by the yard'. He did not build the Town lattice truss bridges himself, but issued licences to local builders to use his patent design instead. He collected a dollar for every foot built, and two if a bridge was built without his permission. By doubling the planking and wooden pins to fasten the structure together, Town made his truss carry the early railroads.

The railroad and the truss bridge

With the arrival of the railways in the USA, bridge building continued to develop in two separate ways. One school continued to evolve stronger and leaner timber truss structures while the other experimented with cast iron and wrought iron, which slowly replaced timber as the principal construction material.

The first patent truss to incorporate iron in a timber structure was the Howe truss. It had top and bottom chords and diagonal bracing in timber, with vertical members of iron rods in tension. This basic design, with modifications, continued right into the next century. The first fully designed truss was the Pratt truss, which reversed the forces of the Howe truss by putting the vertical timber members in compression and the iron diagonal members in tension. The Whipple truss, designed in 1847, was the first all-iron truss – a bowstring truss – with the top chord and vertical compression members made from cast iron and the bottom chord and diagonal bracing members made from wrought iron. Later, Fink, Bollman, Bow and Haupt in the USA, along with Cullman and Warren in Europe, developed the truss to a fine art, incorporating wire strand cable, timber and iron to form lightweight but strong bridges that could carry railways. Figures 1.7 and 1.8 show examples of various truss types.

The stresses and fatigue loading from moving trains in the late nineteenth century caused catastrophic failure of many timber truss and iron truss bridges. The world was horrified by the tragedy and death toll from collapsing bridges. At one

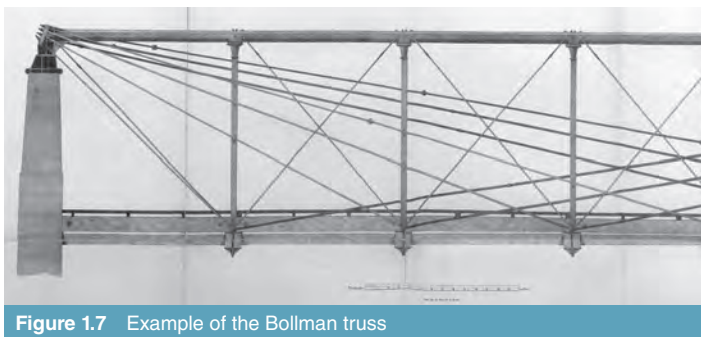


Figure 1.7 Example of the Bollman truss

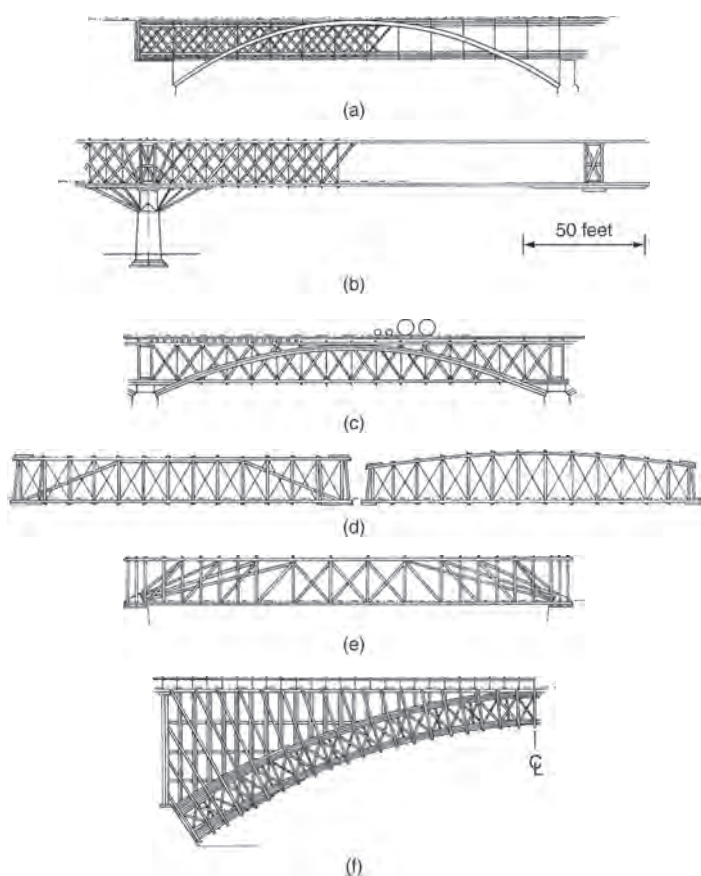


Figure 1.8 US patent truss types: (a) Wilton, 1839; (b) Howe, 1844; (c) Stone-Howe, 1849; (d) Pratt, 1844; (e) Hassard, 1846; (f) Adams, 1848

stage, as many as one bridge in every four used by the railway network in the USA had a serious defect or had collapsed. By the turn of the century, the iron truss railway bridges had been replaced by stronger and more durable structures. Design codes and safety regulations were drawn up and professional associations were incorporated to train, regulate and monitor the quality of bridge engineers.

In the nineteenth century, the truss, the last of the three principal bridge forms, had at long last been discovered. With the coming of the Industrial Revolution, and the rapid growth of the machine age – dominated by the railway and motor car – a huge burden was placed on civil engineering, material

technology and bridge building. Many new and daring ideas were tried and tested, and many innovative bridge forms were built. There were some spectacular failures. As many as seven major new bridge types were to emerge during this period: the box girder, the cantilever truss girder, the reinforced and prestressed concrete arch, the steel arch, glued segmental construction, cable-stayed bridges and stressed-ribbon bridges.

The past 200 years: bridge development in the nineteenth and twentieth centuries

The Industrial Revolution, which began in Britain at the end of the eighteenth century, gradually spread and brought with it huge changes in all aspects of everyday life. New forms of bulk transportation, by canal and rail, were developed to keep pace with the increasing exploitation of coal and the manufacture of textiles and pottery. Coal fuelled the hot furnaces to provide the high temperatures to smelt iron. Henry Bessemer invented a method of producing crude steel alloy by blowing hot air over smelted iron. Siemens and Martins refined this process further to produce the low carbon steels of today. High temperatures were also essential in the production of cement, which Joseph Aspdin discovered by burning limestone and clay on his kitchen stove in Leeds in 1824. Wood and stone were gradually replaced by cast iron and wrought iron construction, which in turn was replaced by first steel and then concrete – the two primary materials of bridge building in the twentieth century.

Growing towns and expanding cities demanded continuous improvement and extension of the road, canal and railway infrastructure. The machine age introduced the steam engine, the internal combustion engine, factory production lines, domestic appliances, electricity, gas, processed food and the tractor. Faster assembly of bridges was essential, and this meant prefabricating lightweight, but tough, bridge components. The heavy steam engines and longer goods trains imposed larger stresses on bridge structures than ever before. Bridges had to be stronger and more rigid in construction and yet had to be faster to assemble to keep pace with progress. Connections had to be stronger and more efficient. The nut and bolt were replaced by the rivet, which was replaced by the high-strength friction grip bolt and the welded connection.

The arrival of the automobile resulted in a road network that eventually criss-crossed the entire countryside from town to city, over mountain ranges, valleys, streams, rivers, estuaries and seas. Even bigger and better bridges were now needed to connect islands to the mainland and countries to continents in order to open up major trading routes. The continuous search and development for high-strength materials of steel, concrete, carbon fibre and aramids, today combined with sophisticated computer analysis and dynamic testing of bridge structures against earthquakes, hurricane wind and tidal flows, has enabled bridges to span even farther. In the last two centuries, bridge spans have leapt from 350 ft to over 6000 ft. This is the

age of the mighty suspension bridge, the elegant cable-stayed bridge, the steel arch truss and the glued segmental and cantilever box girder bridges.

The key events and achievements of this large output of bridge building are briefly summarised to illustrate the rapid pace of change and many bridge ideas that were advanced. In the past two centuries, more bridges were built than in the entire history of bridge building prior to that!

The age of iron (1775–1880)

Of all the materials used in bridge construction – stone, wood, brick, steel and concrete – iron was used for the shortest time. Cast iron was first smelted from iron ore successfully by Dud Dudley in 1619. It was another century before Abraham Derby devised a method to economically smelt iron in large quantities. However, the brittle quality of cast iron made it safe to use only in compression, in the form of an arch. Wrought iron, which replaced cast iron many years later, was a ductile material that could carry tension. It was produced in large quantities after 1783 when Henry Cort developed a puddling furnace process to drive impurities out of pig iron.

But iron bridges suffered some of the worst failures and disasters in the history of bridge building. The vibration and dynamic loading from a heavy steam locomotive, and from goods wagons, create cyclic stress patterns on the bridge structure as the wheels roll over the bridge, going from zero load to full load then back to zero. Over a period of time, these stress patterns can lead to brittle failure and fatigue in cast iron and wrought iron. In one year alone in the USA, as many as one in every four iron and timber bridges had suffered a serious flaw or had collapsed. Rigorous design codes, independent checking and new bridge building procedures were drawn up, but not soon enough to avert the worst disaster in iron bridge history, the Tay Bridge disaster in 1879. It marked the end of the iron bridge.

Significant bridges

- 1779 Iron Bridge in Coalbrookdale, the first cast iron bridge, designed as an arch structure by Pritchard for owner and builder Abraham Darby III (Figure 1.9).



Figure 1.9 Iron Bridge in Coalbrookdale

- 1790 Buildwas Bridge, the second cast iron bridge built in Coalbrookdale, designed by Thomas Telford, using only half the weight of cast iron of the Iron Bridge.
- 1807 James Finlay builds first elemental suspension bridge – the Chain Bridge – in wrought iron, over the Potomac.
- 1823 Gaunless Bridge, George Stephenson's wrought iron 'lenticular' girder bridge for the Stockton and Darlington Railway.
- 1826 Menai Suspension Bridge, famous eyebar, wrought iron chain suspension bridge over the Menai Straits, by Thomas Telford.
- 1834 The Fribourg Bridge, the world's longest iron suspension bridge.
- 1841 Whipple patents the cast iron 'bowstring' truss bridge.
- 1846 Wheeling Suspension Bridge, Charles Ellet's record-breaking 1000 ft span, iron wire suspension bridge.
- 1850 Britannia Bridge, first box girder bridge concept, built in wrought iron by Robert Stephenson (Figure 1.10).
- 1853 Murphy designs a wrought iron Whipple truss, with pin connections.



Figure 1.10 Britannia Bridge, Anglesey



Figure 1.11 Royal Albert Bridge, Saltash

- 1858 Royal Albert Bridge, Saltash – Brunel's famous tubular iron bridge – over the Tamar (Figure 1.11).
- 1876 The Ashtabula Bridge disaster in USA; more than 90 people died when this iron modified Howe truss collapsed, plunging a train and its passengers into the deep river gorge below.
- 1879 The Tay Bridge disaster, Dundee, Scotland; a passenger train with 75 people plunged into the Tay estuary, as the supporting wrought iron girders collapsed in high winds.
- 1884 The Garabit Viaduct, St Flour, France – Gustav Eiffel's truss arch in wrought iron was the prototype for future steel truss construction. Eiffel would have preferred steel but chose wrought iron because it was more reliable in quality and cheaper.
- 1916 The Hell Gate Bridge, New York – the first 977 ft steel arch span in the world was designed by Gustav Lindenthal.
- 1931 The Bayonne Bridge, New York – the first bridge to be built with a cheaper carbon manganese steel, which is the composition of most modern steel, rather than nickel steel.
- 1932 Sydney Harbour Bridge – this famous steel arch was built using 50 000 t of nickel steel, and a main span of 503 m. Its design was based on the Hell Gate Bridge.
- 1978 New River Gorge Bridge, West Virginia – with a span of 518 m, this was the world's longest steel arch span until the 2000s, when two bridges in China pushed their spans up to 552 m.
- 2020 The Guangxi Pingnan Third Bridge in China extended the maximum span length for this type of bridge to 575 m using concrete-filled steel tubes for the main arch sections.

In the UK, the Tyne Bridge, another steel truss arch structure, was built in the 1920s (see Figure 1.12).



Figure 1.12 Tyne Bridge, Newcastle upon Tyne

The arrival of steel

Steel is a refined iron, where carbon and other impurities are driven off. Techniques for making steel are said to have been known in China in 200 BC and in India in 500 BC. However, the process was very slow and laborious and, after a great deal of time and energy, only minute amounts were produced. It was very expensive, so it was only used for edging tools and weapons until the nineteenth century. In 1856, Henry Bessemer developed a process for bulk steel production by blowing air through molten iron to burn off the impurities. This was followed by the open hearth method, patented by Charles Siemens and Pierre-Émile Martin in Birmingham, England, in 1867, which is the basis for modern steel manufacture today. It took a while for steel to supersede iron, because it was expensive to manufacture. But when the world price of steel dropped by 75% in 1880, it suddenly became competitive with iron. It had vastly superior qualities, in both compression and tension; it was ductile and not brittle like iron, and was much stronger. It could be rolled, cast or even drawn, to form rivets, wires, tubes and girders. The age of steel opened the door to tremendous advances in long-span bridge building technology. The first bridges to exploit this new material were in the USA, where the steel arch, the steel truss and the wire rope suspension bridges were pioneered. Later, Britain led the world in the cantilever truss bridge and the steel box girder bridge deck.

The historical progress of the principle of building bridges in steel covering the period from 1880 to the present is described next.

The steel truss arch

When steel prices dropped in the 1870s and 1880s, the first important bridges to use steel were all in the USA. The arches of St Louis Bridge over the Mississippi and the five Whipple trusses of the Glasgow Bridge over the Missouri were the first to incorporate steel in truss construction. St Louis, situated on the Mississippi and near the confluence of the Missouri and Mississippi, was the most important town in midwest USA, and the focal point of north–south river traffic and east–west overland routes.

- 1874 The St Louis Bridge – James Eads builds the first triple-arch steel bridge.

The cantilever truss

Arch bridges had been constructed for many centuries in stone, then iron and later, when it became available, in steel. Steel made it possible to build longer-span trusses than cast iron without any increase in the dead weight. Consequently, this made cantilever long-span truss construction viable over wide estuaries. The first and most significant cantilever truss bridge to be built was the rail bridge over the Firth of Forth near Edinburgh, Scotland, in 1890. The cantilever truss was rapidly adopted for the building of many US railroad bridges until the collapse of the Quebec Bridge in 1907 (Figure 1.13).

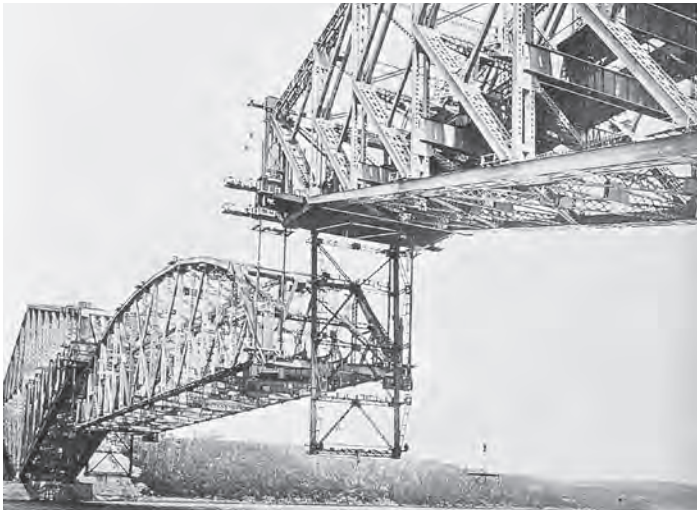


Figure 1.13 Quebec Bridge, Canada

- 1886 The Fraser River Bridge, Canada – believed to be the first balanced cantilever truss bridge to be built. All the truss piers, links and lower chord members were fabricated from Siemens–Martin steel. It was dismantled in 1910.
- 1890 The Forth Rail Bridge, Edinburgh, Scotland – when finished, the world's longest spanning bridge at 1700 ft.
- 1891 The Cincinnati–Newport Bridge, Cincinnati – with its long through-cantilever spans and short truss spans, this was the prototype of many rail bridges in the USA.
- 1902 The Viar Viaduct, France – this rail bridge between Toulouse and Lyon was an elegant variation of the balanced cantilever, with no suspended section between the two cantilever arms.
- 1919 The Quebec Bridge – completion of the second Quebec Bridge, the world's longest cantilever span.
- 1927 Carquinez Bridge – the last of the long cantilever truss bridges to be built in the USA, although a second identical bridge was built alongside it in 1958 to increase traffic flow.

The suspension bridge

The early pioneers of chain suspension bridges were James Finlay, Thomas Telford, Samuel Brown and Marc Seguin, but they had only cast and wrought iron available in the building of their early suspension bridges. It was not until Charles Ellet's Wheeling Suspension Bridge had shown the potential of wire suspension using wrought iron that the concept was universally adopted. Undoubtedly, the greatest exponent of early wire suspension construction and strand spinning technology was John Roebling. His Brooklyn Bridge was the first to use steel for the wires of suspension cables.

Suspension bridges are capable of huge spans, bridging wide river estuaries and deep valleys and have been essential in establishing road networks across countries. They have held the record for longest span from 1826 to the present day,

interrupted only between 1890 and 1928, when the cantilever truss held the record.

- 1883 Brooklyn Bridge – following the completion of the Wheeling Suspension Bridge, pioneered by Charles Ellet, John Roebling went on to design the Brooklyn Bridge, the first steel wire suspension bridge in the world.
- 1931 George Washington Bridge – the heaviest suspension bridge to use parallel wire cables rather than rope strand cable, and the longest span in the world for nearly a decade (Figure 1.14).
- 1950 Tacoma Narrows Bridge – the second Tacoma Narrows Bridge, rebuilt after the collapse of the first bridge with a deep stiffening truss deck, set the trend for future suspension bridge design in the USA.
- 1957 Mackinac Bridge – Big Mac is the longest overall suspension bridge in the USA.
- 1965 Verrazano Bridge–Narrows – the last big suspension bridge to be built in the USA, also held the record for the longest span until 1981.



Figure 1.14 George Washington Bridge, New York

- 1967 Severn Bridge – the first bridge to have a slim, aerodynamic bridge deck, eliminating the need for deep stiffening trusses like those of US suspension bridges. It set the trend for future suspension bridge construction.
- 1981 Humber Bridge – the longest span in the world when completed, with supporting strands that were inclined in a zigzag fashion rather than the parallel arrangement preferred by the Americans.
- 1998 Great Belt East Bridge – the Great Belt crossing is the longest bridge in Europe. For a short while the main span of the East Bridge held the record for the longest span in the world.
- 1998 Akashi Kaikyo Bridge – is one of a family of long-span bridges linking the islands of Honshu and Shikoku. Its main span of 1991 m made it the longest span in the world for the next 24 years.
- 2022 Çanakkale 1915 Bridge – under construction in Turkey at the time of writing, this bridge will become the longest span in the world when completed.
- 2022 Strait of Messina bridge, Italy – is planned with a main span of 3300 m, well beyond any other suspension bridge previously constructed. On and off over the last 40 years, this project was on again at the time of writing and, if finally constructed, is likely to remain the longest span for years to come.
- Lower Yarra River. By far the worst collapse was on the West Gate Bridge, a single cable-stay structure with a continuous box girder deck. A deck span section 367 ft long and weighing 2000 t, buckled and crashed off the pier support on to some site huts below, where workmen and engineers were having their lunch; 35 people were killed in the tragedy. After this accident, further construction of steel box girder deck bridges was halted until better design standards, new site checking procedures and a fabrication specification were agreed internationally.
- 1936 Elbe Bridge – one of the early plate girder bridges on the German autobahn.
- 1948 Bonn–Beuel bridge – a later development of the plate girder into a flat arch, to reduce material weight.
- 1952 Cologne Deutz Bridge – the first slender steel box girder bridge in the world.
- 1974 Rio–Niterói Bridge – built in Brazil with a 300 m span to become the world's longest span of this type.
- 1970s Failure of box girders at Milford Haven in Wales and West Gate Bridge in Australia halted further building of the steel box girder bridge decks for a time.

Since the 1980s, steel box girder bridges have again become popular for medium-span bridges in countries where the steel industry remains strong.

Steel plate girder and box girder

Since the development of steel and of the I-beam, many beam bridges were built using a group of beams in parallel, which were interconnected at the top to form a roadway. They were quick to assemble but were only practical over relatively short spans for rail and road viaducts. The riveted girder I-beam was later superseded by the welded and friction grip bolted beam. However, relatively long spans were not efficient, as the depth of the beam could become excessive. To counter this, web plate stiffeners were added at close intervals to prevent buckling of the beams. Another solution was to make each beam into a hollow box, which was very rigid. In this way, the beam depth could be reduced and material could be saved. Steel box girder beams could be quickly fabricated and were easy to transport. Their relatively shallow depth meant that high approaches were not necessary. Most of this pioneering work was carried out during and after the Second World War, when there was a huge demand for fast and efficient bridge building for spans of up to 1000 ft. The major rebuilding programme in Germany witnessed the construction of many steel box girder and concrete box girder bridges in the 1950s and 1960s. For spans greater than 1000 ft, suspension and cable-stay bridges are generally more economical to construct.

In the 1970s, the world's attention was focused on the collapse of four steel box girder bridges under construction. These bridges were in Vienna over the Danube, in Milford Haven in Wales (when four people were killed), a bridge over the Rhine in Germany and the West Gate Bridge in Melbourne over the

Concrete and the arch

Although engineers took a long time to realise the true potential of concrete as a building material, today it is used everywhere in a vast number of bridges and building applications. Concrete is a brittle material, like stone, good in compression, but not in tension, so if it starts to bend or twist it will crack. Concrete has to be reinforced with steel to give it ductility, so naturally its emergence followed the development of steel. In 1824, Joseph Aspdin made a crude cement from burning a mixture of clay and limestone at high temperature. The clinker that was formed was ground into a powder, and when this was mixed with water it reacted chemically to harden back into a rock. Cement is combined with sand, stones and water to create concrete, which remains fluid and plastic for a period of time, before it begins to set and hardens. It can be poured and placed into moulds or forms while it is fluid, to create bridge beams, arch spans, support piers – in fact, a variety of structural shapes. This gives concrete special qualities as a material, and scope for bold and imaginative bridge ideas.

François Hennebique was the first to understand the theory and practical use of steel reinforcement in concrete, but it was Robert Maillart (1872–1940) who was first to pioneer and build bridges with reinforced concrete. Eugène Freyssinet, Maillart's contemporary, was also keen to experiment with concrete structures and went on to discover the art of prestressing, giving the bridge industry one of the most efficient methods of bridge deck construction in the world. Both these men were great engineers and champions of concrete bridges. What they achieved set the trend for future developments in concrete

bridges – precast bridge beams, concrete arch, box girder and segmental cantilever construction. Concrete box girder bridge decks are incorporated in many modern cable-stay and suspension bridges.

Jean Muller and contractors Campenon Bernard were responsible for building the first matchcast, glued segmental, concrete box girder bridge in the world. This technique is now used by many bridge builders across the world. The box girder span can be precast in segments or cast in place using a travelling formwork system. These bridges can be built as balanced cantilevers each side of a pier or launched from one span to the next.

Concrete has been used in building most of the world's longest bridges. The relative cheapness of concrete compared with steel, with the ability to rapidly precast or form prestressed beams of standard lengths, has made concrete economically attractive. Lake Pontchartrain Bridge, a precast concrete segmental box girder bridge in Louisiana, is the longest bridge in the USA, with an overall length of 23 miles.

The concrete arch

- 1898 Glenfinnan Viaduct – the first concrete arch bridge to be built in England.
- 1905 Tavanasa Bridge – a breakthrough in the stiffened arch slab.
- 1922 St Pierre de Vouvray – early concrete bowstring arch of Freyssinet.
- 1929 Plougastel Bridge – unique construction concept, which used prestressing for the first time.
- 1930 Salginatobel Bridge – one of the most aesthetic arch spans of Maillart (Figure 1.15).
- 1936 Alsea Bay Bridge – one of Conde McCullough's fine 'art deco' bridges in Oregon (since demolished).
- 1964 Gladesville Bridge – use of precast prestressed segmental construction for the arch span.
- 1964 Krk Bridge, Croatia – at 204m, it was the longest concrete arch span in the world at the time.
- 1995 Jiangjhe Bridge, China – became the longest span in the world when completed, with a main span of 300m
- 2016 Qinglong Railway Bridge, China – with a span of 445m it is currently the longest span of this type in the world and, at 295m above the river it crosses, it is also one of the world's highest.



Figure 1.15 Salginatobel Bridge – one of the most aesthetic arch spans of Maillart (courtesy of EH)

Concrete box girders

- 1950s–1960s Many motorway bridges and viaducts were built in Europe and the USA using concrete box girder construction. Some were precast segmental construction; some were cast in place.
- 1952 Shelton Road Bridge – first matchcast, glue segmental, box girder construction in the world, developed by Jean Muller.
- 1956 Lake Pontchartrain Bridge – the second longest bridge in the world, is a precast segmental box girder bridge with 2700 spans and runs for 23 miles across Lake Pontchartrain near New Orleans. A second identical bridge, built alongside the original one in 1969, was 69m longer.
- 1963 Medway Bridge – the first European river bridge to be built using concrete box girder construction (Figure 1.16); at 150m, it had the longest span of this type in the world at the time.
- 1976 Hamana Bridge, Japan – built using the balanced cantilever technique, a span of 240m was achieved.
- 2006 Shibano Yangtze River Bridge, China – at 330m, currently the world's longest concrete box girder span without cable stays.



Figure 1.16 Medway Bridge, Kent

Cable-stay bridges

Cable stays are an adaptation of the early rope bridges, and of guy ropes for securing tent structures and the masts of sailing ships. When very rigid, trapezoidal box girder bridge decks were developed for suspension bridges, this allowed a single plane of stays to support the bridge deck directly. This meant that fewer cables were needed than for a conventional suspension system, there was no need for anchorages and therefore construction was cheaper. Cost and time have always been the principal motivators for change and innovation in bridge engineering.

The first modern cable-stay bridges were pioneered by German engineers just after the Second World War, led by Fritz Leonhardt, René Walter and Jörg Schlaich. The cable-stay bridge is probably the most visually pleasing of all modern long-span bridge forms. In recent times, the development of the cable-stay and box girder bridge deck has continued with

the work of Danish engineers COWI, bridge engineers Carlos Fernández Casado of Spain, R Greisch of Belgium, Jean Muller International, Sogelerg and Michel Virlogeux of France.

Cable-stay history

- 1955 Strömstrund Bridge, Norway – the first cable-stay bridge.
- 1956 North Bridge, Düsseldorf – using an early harp arrangement for a family of cable-stay bridges over the Rhine, it was the prototype for many cable-stay bridges.
- 1959 Severin Bridge – the first to adopt an A-frame tower and the first bridge to use a fan configuration for the stays; a very efficient bridge form.
- 1962 Lake Maracaibo Bridge – an unusual composite cable-stay and concrete frame support structure for a bridge built in Venezuela, using local labour.
- 1962 Nord Elbe Bridge, Hamburg – the first bridge to use a single plane of cables; the deck was a stiffened rectangular box girder.
- 1966 Wye River Bridge, England – a single cable-stay from the mast supports the continuous steel box girder bridge deck; the Erskine Bridge, built in 1971, was a better example of this construction.
- 1974 Brotonne Bridge – the first cable-stay bridge to use a precast concrete box girder deck and a single plane of cable stays (Figure 1.17).
- 1984 Coatzaocalcos II Bridge, Mexico – elegant pier and mast tower, combining the rigidity of the A-frame with the economy of a single foundation.
- 1995 Pont de Normandie – a breakthrough in the design of very long cable-stay spans.
- 2008 Sutong Yangtze River Bridge, China – pushes cable-stayed clear span beyond 1000m.
- 2012 Russky Bridge, Vladivostok – currently the world longest pure cable-stayed span, at 1104m, with a steel deck.



Figure 1.17 Brotonne Bridge, Sotteville, France

- 2016 Yavuz Sultan Selim Bridge, Istanbul – a hybrid mixture of cable-stayed and suspension, the main span of 1410m is currently the longest span of this type. It is also one of the widest, at 58.4m, and tallest, with towers at 322m high.

Aesthetic design in bridges

Introduction

Is it possible that there is a universal law or truth about beauty on which we can all agree? We can probably argue that, no matter what our aesthetic taste in art, literature or music, certain works have been universally acclaimed as masterpieces because they please the senses, evoking admiration and a feeling of well-being. Music, literature and painting can appeal to an audience directly, unlike a building or bridge, whose beauty has to be ‘read’ through its structural form, which has been designed to serve another more fundamental purpose. Judging what is great from many competent examples must come from an individual’s own experience and understanding of past and contemporary styles of expression. The desire to please or to shock is not fundamental in the design of bridges, whose primary purpose is to provide a safe passage over an obstacle, be it a river or gorge or another roadway. A bridge, taken in its purest sense, is no more than an extension of a pathway, a roadway or a canal. We do not regard roads, paths and canals as ‘art forms’ that evoke aesthetic pleasure, as we do with buildings. Hence, it is reasonable to ask why a bridge should be an art form. In the very early years of civilisation, bridges were built to breach a chasm or stream to satisfy just that purpose. They had no aesthetic function. Later on, when great civilisations placed a temporal value on the quality of their buildings and heightened their religious and cultural beliefs through their architecture, these values transferred to bridges. And like all the important buildings of a period, when stone and timber were the principal sources of construction material, work was done by skilled craftsmen. Masons would cut, chisel and hew stones; carpenters would saw, plane and connect pieces of timber falsework or centring to support the masonry structure. It took many years to ‘fashion’ a bridge. Each stone was carefully cut to fit precisely into position. Hundreds of stonemasons would be employed to work on the important bridges. Voussoirs and key stones were sculptured and tooled in the architectural style of the period. Architecture was regarded as an integral part of bridge construction, and this tradition continued into the age of iron, where highly decorative wrought iron and cast iron sections were expressed on the external faces of the bridge. Well into the middle of the twentieth century, arch bridges in concrete and steel were cloaked in masonry panels to imitate the Renaissance, Classical and Baroque periods.

Gradually, however, as the pace of industrial change intensified, by the expansion of the railways, and by the building of road networks, a radical step change in the design and construction of bridges occurred. Bridges had to be functional, they had to be quick to build, low in cost and structurally

efficient. They had to span farther and use fewer materials in construction. Less excavation for deep piers and foundations under water meant faster construction, whereas short continuous trestle supports across a wide valley were simple to construct and required shallow foundations. Under these pressures, standardisation and prefabrication of bridges displaced aesthetic consideration in bridge design. Of course, there were exceptions when prestigious bridges were commissioned in major commercial centres to retain the quality and character of the built environment. And sometimes even these considerations were sidelined in the name of progress and regeneration, as was the case in the aftermath of the two world wars. When economic stability returns to a nation after the ravages of war, and living standards start to rise, so does interest in the arts and the quality of the built environment.

After the Second World War, for example, rebuilding activity had to be fast and efficient, with great emphasis placed on prefabrication, system-built housing and the tower block to rehouse as many people as possible. In Germany, rebuilding the many bridges that were demolished led to the development of the plate girder and box girder structures. Box girder bridge structures, with standardised sections, proliferated the road networks and motorways of Europe, were built over viaducts, interchanges, flyovers and river crossings. In this period, the shape and form of the bridge was dictated by the contractor's preference for repetition and simplicity of construction.

Given this history, it is hardly surprising to find that many of our towns, urban areas and motorway networks are blighted by ugly, functional bridging structures, whose presence now causes a public outcry.

Bridge aesthetics in the twentieth century

Over the centuries, as the various forms of bridges evolved in the major towns and cities, the architectural style of the period was superimposed on them, to create order and homogeneity. Classical, Romanesque, Byzantine, Islamic, Renaissance, Gothic, Baroque, Georgian and Victorian architectural styles adorn many historic bridges today, such as the Renaissance Rialto Bridge in Venice, the Romanesque Ponte Sant'Angelo in Rome and the French Gothic of the Pont de la Concorde in Paris. They are recognisable symbols of their eras, of imperialist ambition and nationhood, where the dominant form of construction was the arch. But with the arrival of steel and concrete in the early part of the twentieth century, new structural forms emerged in building and bridge design that radically changed both the architecture and visual expression of bridges. The segmental arch was replaced by the flat arch, the flat plate girder and box girder beam; the cantilever truss was replaced by the cable-stay and the suspension bridge. The decorative stone-clad bridges of the past were slowly replaced by the minimalism of highly engineered structures.

Undoubtedly, during the period from the 1920s to the 1940s the greatest concentration of bridge building was in the USA.

This was in step with the massive industrial and commercial expansion throughout the country, and emergence of the high-rise building – the skyscraper. And in building bridges – the great suspension, steel arch and cantilever truss bridges – those that were important were the subject of much debate about appearance, and harmony with their surrounding environment. Champions of aesthetic bridge design emerged – David Steinman, Conde McCullough, Gustav Lindenthal and Othmar Ammann. All of these were engineers. Steinman was the most flamboyant and outspoken individual among this group and wrote books and articles on the subject. Conde McCullough's 'art deco' bridges – inspired by the bridges of Robert Maillart – were aesthetic masterpieces of the concrete arch and steel cantilever truss bridge.

In the 1950s and 1960s, the bridge building boom moved to Europe following the war years, with a plethora of utilitarian structures built in the name of economy. Architectural and aesthetic considerations were reduced to a minor role. Bland, insensitive and crude bridge structures and viaducts appeared across the open countryside, and through towns and across cities. Concern about the impact these bridges would have on the built environment brought Fritz Leonhardt, one of Germany's leading bridge engineers, to Berlin in the 1950s. He was part of a small team whom the government highways department made responsible for incorporating aesthetics into bridge design. He worked with a number of leading German architects, particularly Paul Bonatz, and through this association and from extensive field studies of bridges, he evolved a set of criteria on the design of good-looking bridges. He set this out in his book on bridge aesthetics *Brücken (Bridges)* (Bonatz and Leonhardt, 1956). Figure 1.18 shows an example of one of his bridges.



Figure 1.18 An example of Fritz Leonhardt's work – Maintalbrücke Gemünden

Although bridge design was dominated by civil engineers in the twentieth century, somehow the aesthetic vision of the early pioneers, such as Roebling, Eiffel and Maillart, and later that of Steinman and McCullough and their contemporaries, was never seriously addressed in contemporary bridge design in the

UK during the middle and later years of the twentieth century. It appears that the education and training of British civil engineers did not include an understanding of the architecture of the built environment.

Was this also true in other parts of Europe after the war? It is possible that in France, with the emergence of bridges such as those at Plougastel (Figure 1.19), Orly Airport Viaduct, Tancarville and Brotonne, and, more recently, examples such as the Pont de l'Isère (Figure 1.20), the second Garabit Viaduct and Pont de Normandie, a conscious effort was made to build beautiful bridges. In conversation with Jean Muller and Michel Virlogeux, comparing their educational background and training with that of the great Eugène Freyssinet, it would seem that they all had some education and teaching on bridge aesthetics at university. This might explain why their bridges look elegant and thoroughly well engineered. It also appears that senior personnel in government bridge departments in France, who appoint consultants and commission the building of the major bridges, have the same commitment to build visually pleasing bridge structures. Many of them have been schooled in bridge engineering at the University of Paris. Awareness of bridge aesthetics at engineering school is a critical factor. And having developed a design that fully reconciles aesthetics, it is then sent out for tendering. Contractors in France are not given the opportunity to propose cheaper alternative designs, only the opportunity to propose construction innovations in building the chosen design economically. Not surprisingly, aesthetically designed bridges are competitive on price, as the major constructors in France have, over the years, invested in new technology and sophisticated erection techniques to build efficiently.



Figure 1.19 Plougastel, France (courtesy of JMI)

In England in the 1990s, two unconnected, yet controversial, events marked a watershed in bridge aesthetics and gave recognition to the role of architects in bridge design. The first of these events was the Bloomers Hole Footbridge competition run by the Royal Fine Arts Commission (RFAC) on behalf of the District Council of Thamesdown. The competition, which was run by the rules of the Royal Institute of British Architects (RIBA), was open to anyone – bridge engineers, architects,



Figure 1.20 Pont de l'Isère, France

civil engineers and so on. The entrants had to submit an artistic impression of the bridge and accompany it with notes explaining its construction and how it would be built and describing its special qualities for the location. The bridge was to be a new pedestrian crossing over the upper reaches of the Thames in a very unspoilt setting in Lechlade. Each entrant was given a reference number, so that the judges had no knowledge of the name of the entrant. The winning design, out of 300 entries, was created by an architect. The president of the RFAC, speaking on behalf of the judging panel, described the winning design as a 'beautiful solution of great simplicity and elegance entirely appropriate to its rural setting' – but it was not built. The residents of Lechlade labelled the design a 'yuppie tennis racket from hell' and planning permission was withheld. Nevertheless, the imaginative design ideas that resulted from this competition prompted many local authorities and development corporations, particularly the London Docklands Development Corporation (LDDC), to follow suit. Coincident with the competition was a second watershed event – a design study for the proposed East London River Crossing by Santiago Calatrava that took the bridge world by storm. Calatrava's dramatic, rapier-slim bridge concept arching over the Thames showed how a well-engineered bridge design can produce a pleasing aesthetic – it seemed that everyone wanted Calatrava to design a bridge for them.

During the past three decades in the UK, architectural style has been a confusing cocktail of past and present influences, high-tech and neoclassical, romantic modernism and minimalism, which has in some ways marginalised the influence and appreciation of architecture. As a result, highway authorities that commission bridges have paid more attention to structural efficiency, cost control and long-term durability. Aesthetic consideration, if addressed at all, was treated as an appendage, and the first item to be dropped if the tender price was high. The reason for this was simple: both the client and design consultant were civil engineers with little empathy towards modern architecture and the aesthetic judgement of architects on bridge design. Unfortunately, earlier this decade a recent exhibition on 'living bridges' at the Royal Academy has confirmed this point of view. The architecture-inspired ideas tended to make bridges look and function like buildings... and failed. But despite this setback the 'old school' attitudes of civil and bridge engineers

are slowly being replaced by a new generation of engineers and clients who have recognised the value of working with architects.

The search for aesthetic understanding

Why have architecture and bridge engineering not found a common language over the centuries, as has happened in building structures? There have been periods of bridge building when both ideals were combined in bridges. Engineers such as Lindenthal, Ammann, Steinman and McCullough in the USA were advocates of visually pleasing bridges. In Europe, individuals such as Freyssinet, Maillart, Leonhardt, Menn, Muller and Caltrava and consultant groups such as Arup, COWI and Casado were recognised for their aesthetic design of bridges. All of them will own up to the fact that they employed or worked alongside architects. Ammann worked closely with Cass Gilbert, the architect of the gothic Woolworth Tower, arguably the most beautiful skyscraper ever built. Steinman built many great bridges, and tried hard to add flair and style to his designs, but he had to teach himself aesthetics at university. 'In my student days when we were taught bridge design, I never heard the word "beauty" mentioned once. We concentrated on stress analysis, design formulae and graphic methods, strength of materials, locomotive loading and influence lines, pin connections, gusset plates and lattice bars, estimating, fabrication and erection and so on... But not a word was said about artistic design, about the aesthetic considerations in the design of engineering structures. And there was no whisper of thought that bridges could be beautiful,' writes Steinman (1945) in an article on the beauty of bridges that appeared in the *Hudson Engineering Journal*. So how did Steinman learn to develop his skill in aesthetic design? 'For my graduation thesis in 1908 at Columbia University I chose to design the Henry Hudson Memorial Bridge [Figure 1.21] as a steel arch. I worked on the idea for a year and a half before my graduation. I was determined to make this design a model of technical and analytical excellence. But this was not all... I was further determined to make my design a model of artistic excellence.' Steinman read everything he could on the subject of beauty, and aesthetics in design. He discussed the subject with friends who were studying architecture, but they could not help him much. 'They were trained in masonry architecture, in classic orders, ornamentation and mouldings... steel was an unfamiliar material.' Instead, he visited existing bridges, to observe and reason why some were ugly and others were thrilling to look at. He spent a lot of time climbing and walking over the Washington Arch Bridge, to study its design from every artistic angle because it inspired him. 'In my thesis I included a thorough discussion and analysis of the artistic merits of my design. When I finished the thesis and turned it over to Professor William Burr... he gave me the unusual mark of 100 per cent.' Figure 1.22 shows another of Steinman's bridges – St John's Bridge in Portland, Oregon – which was opened by Steinman in 1931.

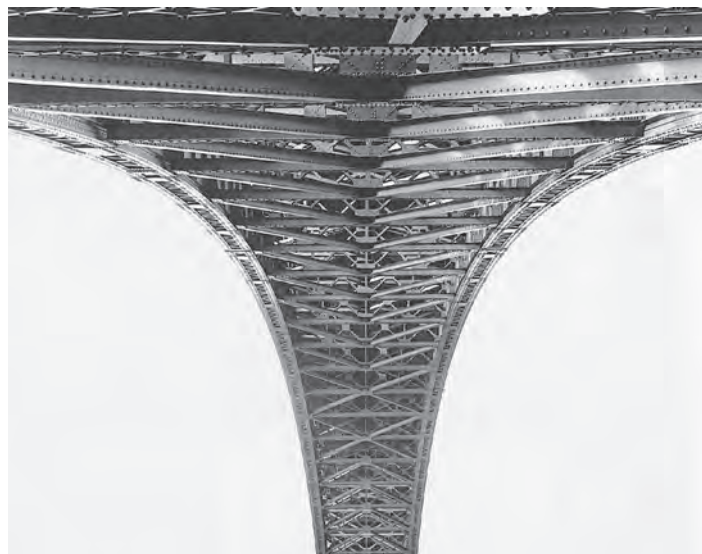


Figure 1.21 Henry Hudson Memorial Bridge, New York (courtesy of Steinman Consulting)



Figure 1.22 St John's Bridge, Portland, Oregon

In the eighteenth century, Perronet took exception to any design that was not pleasing to the eye. In discussing the Nogent Bridge on the upper Seine, he remarked, 'Some engineers, finding that the arches... do not rise enough near the springing, have given a large number of degrees and a larger radius to this part of the curve... but such curves have a fault disagreeable to the eye.' The proportions, the visual line and aesthetic of the arch, were important factors in Perronet's mind. He was trained as an architect. The Pont Neuilly, built over the Seine in Paris in 1776, was one of the most admired bridges in architecture. James Finch, author of *Engineering and Western Civilization* (1951), called Nueilly 'the most graceful and beautiful stone bridge ever built'. Sadly, it was demolished in 1956 to make way for a new steel arch bridge. It would have been useful to

have studied Nueilly today, but one has to recognise that the stone arch is an obsolete technology and has been replaced by the cable-stay, steel arch and concrete box girder bridge.

In the 1960s, Leonhardt suggested the use of the Greek 'golden section' to solve the problem of 'good order' and harmony of proportions. He blames a lack of education for the poor understanding of the importance of aesthetics. Too much emphasis placed on material economy, which is why there are so many ugly structures. 'The whole of society, especially the public authority, the owner builders, the cost consultants and clients are just as much to blame as the engineers and architects,' argues Leonhardt. In his search for an explanation on good aesthetics, he referred to the work of Vitruvius and Palladio and believed that in architecture the idea of good proportion, order and harmony is very appropriate in bridge design. Many engineers have regarded his book *Brücken* (Bonatz and Leonhardt P, 1956) as the definitive guide to bridge aesthetics, but the majority may not have fully appreciated the moral, philosophical and esoteric arguments that he explored. The section on the origins of the golden mean and golden section will generally appeal to the more numerate engineers, who are used to working with mathematical formulae to find solutions.

The Greek philosophers tried to define aesthetic beauty through geometric proportion after years of study and observation. The suggestion was that a line should be divided so that the longer part is to the short part as the longer part is to the whole. The resulting section was known as the golden section and was roughly divided into irrational ratios of 5:8, 8:13, 13:21 and so on. The ratios must never be exact multiples.

This is a dangerous precedent to set, as the golden section can be applied to a bridge just as deflection or stress calculations are done. What Leonhardt concluded in his book, after considering how aesthetics in design were assimilated in both buildings and bridges, was that aesthetics could only be learnt by practice and by the study of attractive bridges. He warned that designers must not assume that the simple application of rules on good design will in itself lead to beautiful bridges. He recommends that models be made of the bridge to visualise the whole design, in order to appraise its aesthetic values. Ethics and morality play a part in good design, according to Leonhardt. Perhaps the words that he was searching for were *integrity* and *purity of form*. There has been a tendency to design gigantic and egotistic statements for bridge structures out of the vanity and ambition of the client. The recent competition for Poole Harbour Bridge was a case in point. It may never be built because of its high cost and because of its lack of integration into the local community it must also serve. One solution that was modest in ambition, but high on community value, with small shops, houses and light industrial buildings built along the length of a new causeway, was entirely appropriate, but alas it was not designed as a 'gateway' structure and did not win.

Jon Wallsgrove, formerly of the Highways Agency in the UK, suggests that the proportions of a bridge – the relationship of the parts to each other and to the whole – could be distilled to the number seven. He made this observation after

researching many books written on aesthetics and beauty over the centuries. The reason for this is that the brain can apparently recognise ratios and objects up to a maximum of seven without counting. He suggests that the ratio of, say, the span to the height of a bridge, or the span to overall length for example, should not exceed seven – for example 1:7; 2:3; 1:2:4 and so on. When the proportions are less than seven they are instantly recognised and appear right and beautiful. The use of shadow line, edge cantilevers and modelling of the surface of the bridge can improve the aesthetic proportion by reducing the visual line of the depth or width of a section, since the eye will measure the strongest visual line of the section, not the actual structural edge.

Fred Gottemoeller – a bridge engineer and architect – concurs with the view that aesthetics in bridge design has largely been ignored in the USA today by the bridge profession and client body. In his book *Bridgescape*, Gottemoeller (1998) sums up the dilemma facing many bridge engineers on the question of aesthetics: 'Aesthetics is a mysterious subject to most engineers, not lending itself to the engineer's usual tools of analysis, and rarely taught in engineering schools. Being both an architect and engineer, I know that it is possible to demystify the subject in the mind of the engineer. The work of Maillart, Muller, Menn and others prove that engineers can understand aesthetics. Unfortunately such examples are too rare. The principle of bridge aesthetics should be made accessible to all engineers.' Gottemoeller has written a clear-sighted, practical book on good bridge design, in a style and language that should appeal to any literate bridge engineer. It is not a book full of pretty reference pictures – the ideas have to work on the intellect through personal research.

It may take time before the new generation of bridge engineers with greater awareness and sensitivity of bridge aesthetics will soften attitudes towards working with architects out of choice. It is doubtful that the basic training and education of civil engineers will change very much in the coming decade. Many academics will feel that there is no need for aesthetics to be included in a degree course and that it should be something an individual should learn in practice. Like it or not, those that are attracted to bridge design and civil engineering do so because they have good analytical and numerate skills. It is pointless putting a paintbrush in the hands of those who hate painting and then expect them to awaken to aesthetic appreciation. In general, the undergraduate engineer has taken the civil engineering option because calculus is preferred to essay writing, technical drawing to abstract art, and scientific experiment to an appraisal of a Thomas Hardy novel. Encouragement in the visual arts and aesthetics will come with practice, and from working alongside architects who are more able to sketch ideas on paper, model the outline of bridge shapes and look for the visible clues to see if a scheme fits well with the surrounding landscape. Architects can help with aesthetic proportion – of structural depth-to-span length, pier shape and spacing, the detailing of the abutment structure, the colour and texture of the finished surface of a bridge, and the preparation of scale models. After all, they have been trained to do this.

The growing trend today is to appoint a team of designers from partnerships between engineers and architects to ensure that aesthetics in design is fully considered. This is a healthy sign. The LDDC successfully forged partnerships between architects and engineers in the design of a series of innovative and creative footbridges sited in London's Docklands. Architects such as the Percy Thomas Partnership, Sir Norman Foster & Partners, Lifschutz Davidson and Chris Wilkinson, in particular, have made the transfer from building architecture to bridge architecture effortlessly. In France, the architect Alain Spielmann has specialised in bridge architecture for nearly 30 years, and has worked with many of France's leading bridge consultants and been involved in the design of over 40 bridge schemes. He is following a tradition in France, where architects such as Arsac and Lavigne have worked closely with bridge engineers. Without doubt the most significant bridge project of the 2000s, the Millau Viaduct in central France, which was won in competition by architect Sir Norman Foster & Partners and a team of leading French bridge designers, has redefined the role of the architect and bridge engineer for the future.

Each period in history will no doubt uncover monsters and marvels of bridge engineering, as they have done with buildings. Succeeding generations can learn to distinguish between good and bad design. What is an example of bad design? We may look on Tower Bridge today as a wonderful, monumental structure, the gateway into the Pool of London, but as a bridge it is ostentatious, with grossly exaggerated towers for such a short span. Some might regard it as a building with a drawbridge, but as a building it serves no real function other than to glorify the might of the British Empire. It would have made more sense to have built two great towers rising out of the water some way upstream of an elegant bridge, located where the bridge is now sited. And if individuals care about the quality of architecture of the built environment, they should voice their opinion and express their views on good and bad design. Silent disapproval is no better than bored indifference. It is worth reflecting that when Tower Bridge was being designed, the Garabit Viaduct and the Brooklyn Bridge had been built. Both bridges and their famous designers were to inspire the engineering world for many decades, but alas not the Victorians.

Civic pride has, over the centuries, compelled governments and local highway authorities to attempt to build pleasing bridges in our cities and important towns, in order to maintain the quality of the built environment. We all agree that the linking of places via bridges symbolises cooperation, communication and continuity and that the bridge is one of the most important structures to be built. It is the modest span bridges over motorways, across canals and waterways in built-up urban areas that are most devoid of any sensitivity with their surroundings – the built environment and the urban fabric of our community. These featureless structures are in such profusion – plate girder bridge decks carrying trains over a busy high street and dirt-stained urban motorway overbridges – that they are the only bridges most of us see as we journey through a town or a city. The cause of this blight stems largely from legislative doctrine on bridge design imposed by highway authorities, whose remit

is to ensure that the design conforms to a set of rules on how it should perform and how little it will cost. It encourages the mediocre, the mundane and unimaginative design to be passed as 'fit for purpose'. What can be done to improve things? The way forward has already been shown by the footbridges commissioned by the LDDC in the UK, by the bridges built by Caltrans along the west coast of the USA in the 1960s, by the bridges built by the Oregon Highways Department in the 1930s and 1940s and the bridges commissioned by SETRA in France in the 1980s, for example along the A75 Clermont–Ferrand highway (Figure 1.23). So it can be done.



Figure 1.23 The A75 Clermont–Ferrand highway bridge (the Millau Viaduct)



Figure 1.24 Bedford – The Butterfly Bridge (courtesy of Wilkinson Eyre)

Vitruvius identified three basic components of good architecture: firmness, commodity and delight. Many subsequent theorists have proposed different systems or arguments by which the quality of architecture can be analysed and their meaning understood. The tenets Vitruvius identified provide a simple and valid basis for judging the quality of buildings and bridge structures today. 'Firmness' is the most basic quality

a bridge must possess and relates to the structural integrity of the design, the choice of material and the durability of the construction. ‘Commodity’ refers to the function of the bridge, and how it serves the purpose for which it was designed. This quality is rarely lacking in any bridge design, whether it is ugly or good to look at. ‘Delight’ is the term for the effect of the bridge on the aesthetic sensibilities for those who come in contact with it. It may arise from the chosen shape and form of the bridge (see Figure 1.24), the proportion of the span to the pier supports, the rhythm of the span spacing and how well the whole structure fits in with the surrounding environment. It is the component that is most lacking in bridges built in the middle half of the twentieth century.

The argument that good design costs more is facile: good design requires a good design team. Look at the bridges of Roebling, Steinman, Maillart and Freyssinet –they were won in competition because they were economic to build and because the designer had considerable knowledge about construction and a gift for visual delight. They also worked closely with talented architects.

The fact that bridges have been designed by bridge engineers and civil engineers for only 300 out of the past 4000 years in

the history of bridge building has not been lost on those who lobby for better-looking bridges. Before that, it was the domain of the architect and master builder. It is reassuring to know that at the beginning of the twenty-first century we seem to be learning from the lessons of the past.

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Chapter 2

Bridge aesthetics

K Brownlie BEAM Architects

Each mind perceives a different beauty [...] every individual ought to acquiesce in his own sentiment without pretending to regulate those of others.

David Hume (1757)

Bridge aesthetics have long been debated and there is much useful published advice in circulation. Attempts to define rules for visual appearance, however, face considerable difficulties in offering universally relevant advice, given the subjective nature of the discussion. This chapter explores themes surrounding holistic bridge design and the consideration of visual and user experience. Based on the premise that the application of guidelines is no substitute for creative and cultural awareness, the chapter expands on a wide range of subjects, including the role of the designer and the case for aesthetic quality, learning from the past, the need for balance, the relationship between cost and value, and the subject of taste. It further discusses the impact of setting, structural type and modal form, and the subjects of order, proportion, shaping, materials and lighting.

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There are no rules

Any guidance directed at engineering professionals might reasonably be expected to offer rigorously tested, strictly defined and applicable rules. Aesthetics, however, occupy ranging subjective territory and attempts to define rules for visual appearance face considerable difficulties in offering universally relevant advice. Many publications have sought to provide a comprehensive guide to bridge aesthetics but should be consulted with an understanding of the context in which they were authored. Notable among these are *Brücken (Bridges)* by Fritz Leonhardt (1974) and *Bridgescape: The Art of Designing Bridges* by Frederick Gottemoeller (1998). Many institutions and authorities have also sought to quantify the subject, including, for example, the UK based Concrete Bridge Development Group (CBDG) via its Technical Guide 4 *The Aesthetics of Concrete Bridges* (CBDG, 2001), and any number of US State Departments of Transportation, who publish detailed aesthetic guidelines. Such publications do not age, in the sense that the same tenets and themes run reliably through them all, no matter when they were authored. The array of available advice provides a valuable resource but is not a panacea. The application of guidelines is no substitute for the application of creative experience and cultural awareness. The great German engineer Jörg Schlaich noted: ‘Nothing narrows the design imagination

more than rules and recipes.’ Consult guidelines, including these, only with a creative and critical eye.

Bridge designers

The question of creative collaboration is a perennial topic of discussion among bridge designers. The theme is not new. In his book *Engineers of Dreams: Great Bridge Builders and The Spanning of America*, Henry Petroski (1995) points to the misaligned views of two bridge engineering luminaries in Gustav Lindenthal’s contemporary review of John Waddell’s 1916 book *Bridge Engineering*:

The author’s repeated reference to aesthetic appearances based on nothing more than the curves in the top or bottom chords will appear to others as rather naïve. This chapter on architecture is well meant as an earnest plea to bridge engineers to show themselves in their work as cultured men. The author, however, in undertaking to furnish instruction and guidance to seekers of the aesthetic in bridge construction, has set himself a task that is evidently beyond his scope. For any bridge structure requiring architectural consideration the bridge engineer will do well to consult a competent architect; and experience has shown that not every architect is competent here.

Throughout modern history, every unsung bridge engineer is matched by the forgotten architect of an engineering triumph.

Soundbite culture and the cult of the individual do not favour the promotion of teamwork. Arguments over the relationship between architecture and engineering will remain current so long as the professions are distinct and its practitioners are taught in a narrow field, but the nature of the creative process and the value of professional interaction are subjects that must be diligently and constantly revisited.

In his authoritative compendium of essays *Architect and Engineer: A Study in Sibling Rivalry*, Andrew Saint (2007) concludes of the historic relationship between the professions: 'Like any true partners they must be seen as different equals who must perpetually be learning how to live together.' Indeed, engineers and architects are trained differently, think differently, act differently. To embrace and exploit these differences is to form the basis of a successful creative partnership.

If it can be agreed that bridges deserve an elevated status in the built environment then there can be no doubt that they must be *designed*, in the most comprehensive meaning of the term. Successful bridge designs can arise from the hand of skilled and enlightened engineers but are more likely to be the product of collaborative partnerships between a diverse range of disciplines. There is no formula to establish the appropriate professional balance for each situation but what is clear is that only an open-minded and fully integrated process will deliver the quality of design that each project should aspire to. It does not matter what the designer's profession is; bridges are designed by bridge designers.

Bridge design

Modern bridge design has witnessed a pendulum-like swing from engineering pragmatism to edifice building, and back towards a more balanced relationship between utility and aesthetics. The increasing embrace of aesthetics within engineering practice and the adoption of bridges into the architectural canon has created a tentative new normal in which the importance of visual quality is widely recognised in both professional and public arenas. One fundamental issue that must underpin the understanding of this situation however is that design and aesthetics are not separable; visual quality is an integral characteristic of design not an additive layer and must be managed from the outset of the process. Nor is visual appearance merely a matter of surface but a wholly integrated combination of situation, composition, form, materiality and detail. Design is a holistic exercise.

A specific issue worthy of note is the relationship between 'visual experience' – the way a bridge appears to viewers – and 'user experience' – the way a bridge is experienced by those in its proximity, particularly in passing over or beneath it. Distinct types of user will experience a bridge in very different ways; the occupant of a fast-moving vehicle and a pedestrian will perceive the bridge environment differently and each should solicit an appropriate design response. The 'view of' and the 'view on' a bridge are overlapping and inter-related concerns that should inform the design process but are not always given due consideration (Figure 2.1). Bridges are for everyone.



Figure 2.1 Equality in the 'view on' and the 'view of': Millennium Bridge, London

The aesthetic obligation

Bridge design is a creative as well as a technical discipline. The view that bridges are merely utilitarian and do not bear the same aesthetic responsibility towards the built environment as buildings is sometimes voiced but never justified. In visual terms, transport infrastructure is a near ubiquitous feature of our visual field and bridges are among the most prevalent and notable examples we encounter. It is a condition of the type that they stand apart from the visual mass of townscape or break the continuity of open landscape and are thus among the most visible of constructions, with a clear obligation for aesthetic quality. This does not mean that each structure must be the centre of focus; a well-designed bridge might enhance or simply befit its place but an ill-conceived bridge imposed on a townscape or landscape can be ruinous to its context. Bridges that are less visible cannot be exempted from the aesthetic obligation; with a design life, on average, twice that of any building, an incongruous bridge can be a long time ugly while its surroundings continue to evolve. No bridge is 'just' a bridge.

Engineering imperatives should not be used to justify ignoring the aesthetic obligation. Larger bridge projects, in particular, might evolve in a technical vacuum but their visibility and usage increases the necessity of careful aesthetic design. As technology permits ever greater structures, it does not follow that such advances can be traded for visual quality. 'Big' and 'beautiful' are not the same thing.

There will always be projects that tend towards the spectacular by virtue of scale, location or cultural ambition. Most bridge projects, however, are more modest, and these 'common' constructions particularly warrant aesthetic care to maintain high standards of visual quality across the bridge register. We must aspire to the 'good ordinary' (Figure 2.2) as well as the *extraordinary*.

But is it art?

Popular bridge design industry narratives may well influence designer's attitudes and actions. Just as most Architects still gravitate to the now historically remote tenets of modernism,



Figure 2.2 ‘Good ordinary’ – military infrastructure elevated to refined civic architecture: Tay Bridge, Aberfeldy, designed by William Adam (1733) for General George Wade’s Highlands military transport road (image © Jim Barton, <https://www.geograph.org.uk/photo/4656319>)



Figure 2.3 New in conversation with old: Gateshead Millennium Bridge and Tyne Bridge, Newcastle (image: Anthony Holmes)

many bridge designers follow the assumed wisdom of ideas such as ‘structural art’ as defined in *The Tower and the Bridge: The New Art of Structural Engineering* by David P Billington (1983). While there is, of course, much to commend pursuit of ‘excellence in efficiency, economy, and elegance’, the idea of a bridge as an objet d’art is not the whole picture. The intrinsic relationship of bridges with place adds an important layer of consideration that object design lacks. Bridge architecture is not just about appearance but is a much broader form of expression. Designers should be confident in determining ‘why it is’ as well as ‘what it is’. ‘Structural narrative’ as well as ‘structural art’ may begin to better describe more rounded objectives for bridge designers.

The constrained typology

Bridges are strictly limited to a few generic structural types constraining designers to the arrangement of a few typical elements. The variant bridge types are set, and the genus is unlikely to expand. It is, therefore, not surprising that so many bridges are formally similar, with only superficial aspects of detail to differentiate them. Despite these constraints, creative designers continue to deliver bespoke solutions in response to each situation. This is largely accounted for by the variable factors of context; not just of geography but also of climate, culture, politics, economics and a raft of other influences.

There is much demand from project sponsors for designs that are both ‘unique’ and ‘iconic’. Designers should be careful of pursuing or promising either; the former is highly unlikely and the latter debatable, since it implies that a design should always define its place rather than be defined by it. A successful design is always a conversation with context (Figure 2.3).

Learning from the past

The constrained typology means that each generic type has been exhaustively explored by generations of engineers and (almost) every conceivable form has been previously contemplated or constructed. Bridges of the past continue to provide

tangible lessons for the present and online resources allow the curious designer to access a vast global portfolio of examples. It is, of course, critical not to consider past designs as a pattern book; they rather provide the means for designers to approach new projects with an informed outlook (Figure 2.4).



Figure 2.4 Historic structure type reinterpreted: Royal Victoria Dock Bridge, London (image: Liline sur Flickr, https://commons.wikimedia.org/wiki/File:Royal_Victoria_Dock_Bridge.jpg)

The still relevant 1914 volume *A Book of Bridges* by Frank Brangwyn and Walter Shaw Sparrow advises: ‘A pontist never need be idle; not only has he a thousand problems to consider, but in all his walks and rides he is a wayfarer with his hobby.’

A balanced approach

The eminent American structural engineer Hardy Cross is credited with the aphorism ‘The first requirement of a beautiful bridge is that it must stand up long enough for us to look at.’

The typological evolution of bridges has always been checked by the two poles of design innovation and design failure, the latter sometimes with catastrophic impact. Bridge design remains a serious and necessarily conservative engineering pursuit, with structural integrity as the key concern. Projects driving doggedly towards a fixed aesthetic vision risk unreasonable compromise to engineering efficiency with

predictable outcomes. A cautious approach however does not in itself deliver good design, particularly in terms of aesthetics. A broad but useful generalisation is to note that structurally optimised design will always look better than expedited or standardised design. Over-engineered structures are conspicuous, even to a lay audience. A bridge that does no more than faithfully express the flow of forces through the structure is a beautiful bridge (Figure 2.5).



Figure 2.5 Elegant expression of the flow of forces: Autobahn A8 Wendlingen–Kirchheim, Germany (image: SBP)

Cost versus value

The relationship between cost and value is sometimes just as subjective as aesthetics but it is an important distinction that decision makers need to fully appreciate. Visual quality is a valuable asset and while value-engineering may reduce cost it does not usually add value as billed. Cost and value are not the same.

Although it is rarely cost-neutral to build out a refined design, economic construction is not destined to be inelegant and in competent hands can result in great visual clarity. Properly establishing the basic components of a cultivated composition at the genesis of the design provides an inherent quality that will usually withstand the fraught path to realisation. The key challenge is to ensure the best aesthetic outcome with the available budget. Maximise the value.

Procurement methodology can have a significant impact on the visual quality of projects. Designers must assist project promoters to understand the implications of alternative procurement routes and the strategies by which visual quality can be protected through the process. Design-build procurement is employed to control cost and isolate risk but is centred around the desire to deliver the employer's requirements in the most expedient manner and may erode aesthetic quality. A 'value culture' is needed but such academic notions have considerably less impact than on-the-ground design initiatives that simultaneously control cost and protect visual quality. This is the role and the value of the good designer; estimators are not aesthetes.

Structural form

The form of a bridge is at least partly self-determining, since the applicable structural types for any given span, mode and situation are limited to a very few applicable options. The span

ranges of various bridge types nevertheless overlap sufficiently to allow comparisons against project constraints. Key decisions over whether to suspend the deck from above or support it from below are fundamental to both viewer and user experience. In situations where this is an available choice, the form and feeling of a project may be significantly altered.

It is always worth rehearsing the basic principle that the inverted form of any structure describes an opposing workable type, substituting tension (e.g. catenary) for compression (e.g. arch) or vice versa. The visual differences between opposing systems are self-evident, often contrasting *lightness* against *mass* or *openness* against *containment*. A traditional design tool used by architects is to invert or reflect plans in order to see them anew. The opposite view is often revealing. Structural inversion is well illustrated in many texts but eloquently so in *Fußgängerbrücken* by Jörg Schlaich and Rudolf Bergermann (1992).

Modal type

What a bridge carries is a major determinant in its visual character. Heavier loads dictate heavier structure and in the formal resolution of different modal types it is difficult to see beyond the wisdom of modernist doctrine of 'form follows function'. There is potentially much to be regretted where a highway bridge attempts to replicate the structural spirit of a footbridge, or a footbridge looks capable of supporting rail traffic. Honesty is a worthy attribute.

Footbridges benefit from greater flexibility in layout, allowing for fluid paths attuned to the desire lines of the pedestrians they serve. Horizontal alignment is a natural starting point for the design of a footbridge, but a straight line should not be; we are not Romans. A good discipline for the conception of all bridges, but especially pedestrian bridges, is to think in terms of designing journeys, not structures (Figure 2.6).



Figure 2.6 Bridge as a curated journey: Kids' Bridge, Perth WA (image: Stephen Heath/BEAM)

The independent object

While the need to satisfy technical and aesthetic objectives is common to all architectural typologies, bridges have a unique physical condition that allows them to be developed and perceived as sculpture in a way that is rarely fitting for buildings.

Bridges are seen in the round – there is no front or back to consider and soffits must be designed with as much care as the other visible faces in the holistic consideration of three-dimensional form (Figure 2.7).

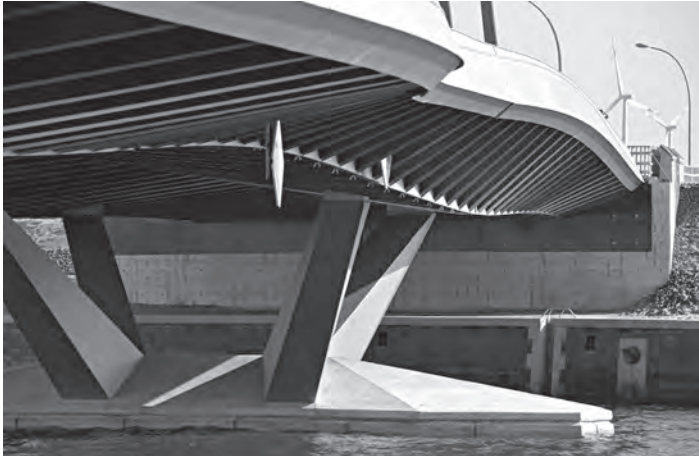


Figure 2.7 Bridge soffit as elevation: Brücke Baakenhafen West, Hamburg

The integrated object

In contrast to the notion of the ‘independent object’, every bridge is just a single component of a greater route and is, by definition, an integral part of a wider context. No bridge can be properly considered in isolation but should be visibly ‘stitched’ into its setting if it is to assume the camouflage of place. Both the independent object and the integrated object must be considered in the design process; good bridge design does not begin and end at the joint or the bearing.

Legibility

Bridges are widely appreciated, at least in part because they are *legible*. A legible design is not by rights also an attractive one, but certainly has an enhanced chance of gaining the viewer’s approval.

The architectural historian Sir James Richards eloquently observes in *The National Trust Book of Bridges* (1984): ‘Regarded as works of architecture, bridges possess the attraction that they can be appreciated without knowledge or understanding of what occurs out of sight, for the logic of their design is self-evident. A sound architectural precept is said to be that form follows function. In the case of a bridge the form *is* the function.’

Legibility might be defined as comprising two parts – clarity and meaning. In other words, first whether the design may be simply understood by the viewer and thereafter whether the design conveys any additional meaning through its disposition and form.

The question of clarity is based largely around the expression of structure; the innate legibility of, say, a thrust arch or a simple suspension bridge will provide comfort to the viewer and user, whereas a complex engineering system is likely to

obfuscate and confuse. The temptation to employ contrived or hybrid structural forms is accompanied by considerable aesthetic risk (Figure 2.8). Purity of form will always prevail.



Figure 2.8 Visually confusing hybrid structure: Seri Saujana Bridge, Putrajaya (image: commons.wikimedia.org/wiki/File:Putrajaya_Bridge_073.jpg)

Meaning can further be conveyed through form; structure can be configured to signify a gateway, an asymmetric layout might be used to reinforce geographical hierarchy, or a circuitous route arranged to draw focus to a landscape feature. Many variants on this theme can be utilised to make tailored statements about the nature of a crossing or its context. It is important for design professionals to understand the implied meanings of a design such that they are both intended by the designer and legible to the viewer (Figure 2.9).



Figure 2.9 Visual clarity and meaning: Puente del Alamillo, Seville, Spain, noting technical ramifications of omitting back stays (image: https://commons.wikimedia.org/wiki/File:Puente_del_Alamillo_en_Sevilla.jpg)

Good order

The chief forms of beauty are order, symmetry and clear delineation.
Aristotle

Scholars might note that Aristotle and Plato both upheld the classical view of beauty as an objective quality, a view that had all but deteriorated by the time enlightenment philosophers

such as Hume and Kant asserted the subjective nature of our perception. By the eighteenth century, beauty was firmly in the eye of the beholder. Sticking with the Aristotelian view, however, those ‘chief forms’ are still relevant to the timeless pursuit of beauty, whether it is objective or subjective. First and foremost, good order is something that is an essential attribute in any bridge design. If all bridges were designed with only the requirement to provide an ordered approach to the span arrangement and in the coordination of elements, then the visual quality of the entire bridge inventory would be substantially improved (Figure 2.10). Solving individual design challenges locally without regard to the potential disturbance of order in the wider composition is a common problem.



Figure 2.10 Order, symmetry and clear delineation: Taplow Bridge, Buckinghamshire (image: © Lance McNulty/Knight Architects)

Proportions

Some guidelines, and indeed some designers, place great store in mathematically defined proportionality, and particularly the golden ratio or section. These formulae might appeal to the empirically minded and could indeed instil a sense of aesthetic order to a composition, but they should not be used as a substitute for a designer’s own intuition. It is surely in the interests of good practice that designs are composed organically and that designers build an imbibed understanding of proportion through experience. Designers will find that compositions that look ‘right’ will very often accord broadly with a proscribed ratio, and they should be able to generate this situation unaided.

There is also a sphere of study around anthropomorphic proportion that is well illustrated by Da Vinci’s famed drawing *The Proportions of the Human Body According to Vitruvius* or *The Canon of Proportion*. It depicts the geometric ideals of human proportions set out by the ancient Roman architect Vitruvius, who proposed the human figure as the generating source of proportion in the classical orders of architecture. How this might impact the composition of bridges is a complex and academic field but is nevertheless a sometimes-relevant consideration. The relationship between the key elements of a structure, for example the deck, tower and stays in a cable-stayed bridge might be likened to the human figure and compared with or

adjusted towards the ideal proportion. We see ‘rightness’, if not beauty, in our likeness.

Component shaping

Good proportions alone do not ensure aesthetic quality – the setting and shaping of individual components are essential in producing a refined composition.

Establishing a clear visual hierarchy between components is a good starting point. A design in which the arrangement of its constituent parts clearly reinforces the structural ‘diagram’ is likely to be more visually successful than one in which the order of primary structure, secondary components and appurtenances is confused. Such hierarchical clarity is a baseline for the development of more nuanced arrangements, if it is done with an understanding of compositional intent and implications.

Unarticulated structure can be detrimental to the appearance and reputation of a bridge, whereas effectively shaped or sculpted components are likely to have a demonstrably positive effect.

Component shaping should be viewed with a clear understanding of the value it provides. While cost-effective strategies can be engaged to refine a baseline design, the lowest-cost objectives that drive many procurement and contract types too often fail to adopt even low-cost high-value adjustments. The choices made between straight and tapered elements, between curved components and faceted or chorded equivalents, or between parabolic and circular profiles can have a huge impact on visual quality.

There are many effective means of ‘tailoring’ components to improve visual outcomes using sensible construction conventions. Sophisticated physical shaping is not always needed and strategies such as the simple manipulation of sunlight and shadow on surfaces, for example, through the judicious use of facets and seams, can be employed to alter the perceived size and shape of components. The eye will naturally follow the



Figure 2.11 Frederick Douglass Memorial Bridge, Washington DC (image: Nathan Porter/BEAM)

most prominent lines of any composition and an understanding of visual perception is a powerful asset to the designer (Figure 2.11).

Materiality

Structural materials are rarely an aesthetic choice but rather a resource to aesthetically manage. Since detailing and construction quality are likely to be the chief means of controlling the appearance of surfaces, every technical decision requires an awareness of its visual implications. The design and specification of concrete mix, formwork, jointing, weld details, and so on, are critical to aesthetic quality.

In modern construction environments, opportunities for the overt manipulation of surfaces, such as extensive concrete tooling are rare. Nevertheless, much available bridge design guidance is enthusiastic about the potential of additive materials or worked surfaces to transform the visual presence of projects. The judicious use of distinctive or contrasting materials can, of course, be visually powerful, although designers must develop an aptitude for working creatively within a more limited palette.

The high incidence of self-finished materials – concrete, stainless steel, weathering steel, aluminium and others – focuses design effort on finish and tone. The opportunity for colour arises, with some exceptions, only on painted steel superstructures and secondary components. It is less common to see bold colour than muted tones of white or grey, since an arresting colour somehow demands justification. Colour has the power to transform and define but is a subject that is as emotive and political as it is academic. Notwithstanding matters of taste, it is a loss to visual diversity that bridge designers so often default to neutral for fear of the alternative. Where greyscale is a cognitive choice, the impact of sunlight on the structure needs to be understood. The distinguished American architect Richard Meier (1984) explains his famously monochromatic work in terms of the ability of white to reflect and refract light: ‘The whiteness of white is never just white; it is almost always transformed by light and that which is changing; the sky, the clouds, the sun and the moon.’ More prosaically, it should be recognised that bridge structure almost never benefits from an effective cleaning or maintenance regime and that ‘day-one colour’ will last only as long as the environment and judicious detailing will allow it to.

Appurtenances

While bridges might themselves be considered urban furniture, the finer grain of added bridge furniture, including signage, services, lighting, barriers, parapets and gantries, has the equal capacity to enhance or undermine the quality of the visual and user experience. Essential non-structural components should be considered as a cohesive part of the composition to avoid the negative impact of discordant clutter.

Bridge appurtenances must be fully integrated but ‘follow-on’ disciplines often work in a professional silo without bigger-picture awareness. Lighting standards and signage, for

example, are typically set out according to the design of the highway rather than the structures that carry that highway, with a predictable impact on appearance. All elements should be planned with regard to the composition and rhythm of the structure, requiring designers to anticipate and accommodate these factors early in the design process. Integration and coordination of bridge furniture is essential.

Lighting

Lighting of the carriageway and lighting of the structure are generally considered as separate concerns, but each has a notable aesthetic affect. The distinction of ‘functional lighting’ and ‘architectural lighting’ implies that the latter has no function and is a dispensable luxury. It is a more sensible approach to consider all lighting as both functional and architectural in aiming to provide appropriate aesthetic and technical results.

The designer is charged with delivering appropriate curated lighting from almost limitless technical possibilities. Colour, active and interactive lighting can all be used to dramatic effect, but a measured approach is needed. In lighting bridges, it is wise to consider that just because something can be done it does not follow that it should be done. The architect and lighting designer Jonathan Speirs advised that designers should approach the subject through the eyes of a child, creating an image of the bridge using as few lines of light as possible. This advice resonates with the essence of bridge design itself (Figure 2.12).



Figure 2.12 Infinity Bridge, Stockton (https://commons.wikimedia.org/wiki/File:Night_Light.jpg)

A matter of taste

Published advice on the appearance of bridges is relatively consistent across both time and geography, but almost never directly addresses the subject of ‘taste’. International thinking promotes the universal applicability of contemporary, serious, *tasteful* design but does not account for the shifting sands of preference between cultures. Tastes differ not only between individuals but also, and especially, between peoples – there are distinct geographical divides in culture, informed by a vast

range of influences from climate to politics. It is essential for the ‘international’ designer to employ cultural awareness but also to avoid cultural appropriation. Within the constrained canon of bridges, designs can and perhaps should acknowledge local concepts of good taste – there is no moratorium. The ‘rightness’ of a design depends very much on where, and for whom, it is designed. The conventions of modernism and moderation serve bridge designers well, but assuming the design moral high ground does not.

Epilogue

From difficult challenges come creative solutions. Bridge design is exquisitely constrained and can be spectacularly challenging, conspiring to provide the right conditions for innovation and excellence. At its core, bridge building is a triumph of human endeavour over nature; it is an act of *civilisation*. That the task falls to civil engineers is an appropriate reminder that the human condition is not only one of survival but also of social advancement. In his 1954 publication *The Bridges of Britain*, Eric De Maré quotes Franklin Roosevelt: ‘There can be little doubt that in many ways the story of bridge building is the story of civilisation. By it we can readily measure an important part of a people’s progress.’

Bridges are engineering ‘ambassadors’ and are rightly subject to much greater scrutiny than less visible or tangible infrastructure types. It is right that they should also be rigorously tested in aesthetic discourse in all its wide and wonderful colours.

Does it matter what a bridge looks like? Yes, it does.

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Chapter 3

Loads and load distribution

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This chapter has been developed from that authored by MJ Ryall in the 2nd edition, published in 2008.

This chapter has been developed from that authored by Dr M. J. Ryall in the 1st edition published in 2000, much of which remains applicable to bridges today. While permanent dead load and superimposed dead loads are covered, it deals primarily with the intensity and application of the transient loads on bridge structures according to European, British and American international codes of practice. The predominant transient loading is usually due to the mass of traffic using the bridge, and some explanation of the history of the development of such loads is given because, for example, one might ask, 'What vehicle (or part of a vehicle) can possibly be represented as a knife-edge load (KEL)?' A steam roller perhaps! Without the historical background knowledge, it is blindly assumed to be appropriate, and the designer is none the wiser. Once the primary traffic loads have been established, consideration is given to secondary loads emanating from the horizontal movement of the traffic, along with environmental and construction loads.

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Introduction

The predominant loads on bridges comprise

- loads due to self-weight and superimposed loads
- the mass and dynamic effects of moving traffic
- transient loading, such as wind, temperature and seismic forces.

Other loads include those due to snow or water flow and those occurring during construction. All of the main loads that can be expected on a bridge at one time or another are shown in Figure 3.1. Different authorities and codes deal with these loads in slightly different ways, but the broad specifications and principles are the same worldwide. Actual loads and their values to be adopted in any bridge design should be confirmed with the bridge owner before commencing any design or assessment.

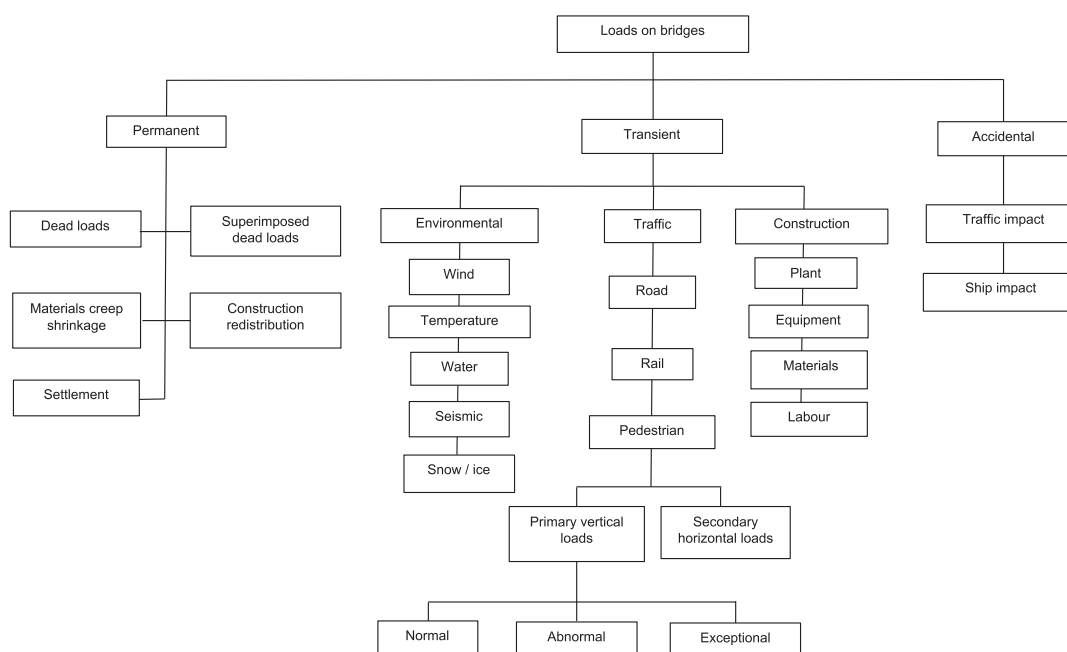


Figure 3.1 Loads on bridges

The following sections describe the loadings that are normally considered on bridges, as categorised in Figure 3.1, and cover the full range of bridge types, from overpasses to long-span bridges and from footbridges to suspension bridges.

Self-weight and superimposed dead loads

Self-weight

Defined as the dead loads from the self-weight of the structural elements of the bridge, consisting of the deck, substructure and foundations, the self-weight of typical materials used in bridge designs is usually taken as shown in Table 3.1.

Material	Weight: kN/m ³
Steel	78
Concrete	25
Aluminium	27
Timber	6

Table 3.1 Typical material weights

Whereas steel and aluminium self-weight are reasonably constant around the world, the weights for concrete and timber can be very variable in practice.

The self-weight of normal reinforced concrete varies depending on the type of aggregate used and the concrete mix, as well as the amount of reinforcement present. Weights can vary from 24 kN/m³ for lightly reinforced concrete, up to 26.5 kN/m³, where heavy aggregates are used, or the reinforcement ratios are high. The self-weight of the concrete to be used should be confirmed on a project-by-project basis and can often be obtained from the practice adopted locally.

Concrete using lightweight aggregates, such as Lytag, can have self-weights down to 16 kN/m³, while special lightweight concrete with self-weights down to 4 kN/m³ can be manufactured using foams, but these are generally not used in structural situations on bridgeworks.

High-density concrete using baryte or magnetite aggregates can have self-weights up to 40 kN/m³, while using iron or lead shot can produce concrete with self-weights over 60 kN/m³. These can be found in situations where the concrete is being used as a counterweight in the bridge structure.

The weight of timber is highly dependent on the type of timber being used, with weights varying from 4 kN/m³ to over 10 kN/m³. The type and weight of timber needs to be specified in any design where this is critical.

Construction sequence and material behaviour

The distribution of the effects from self-weight loads within a structure can be influenced by the construction sequence, elas-

tic shortening in different structural elements and the creep and shrinkage in concrete.

The way a bridge is built and the sequence in which the different elements are constructed can affect the final distribution of the self-weight effects within a structure. A steel beam placed simply supported and then connected to its supports can have a different load effect distribution from one placed on temporary supports, fixed to permanent supports and then having the temporary supports removed. A bridge built by the balanced cantilever technique (Figure 3.2(a)) can have a significantly different self-weight load effect distribution to one built span by span (Figure 3.2(b)). With bridgeworks, it is essential that the structure is analysed taking into account how it was built when deriving the distribution of the self-weight effects.

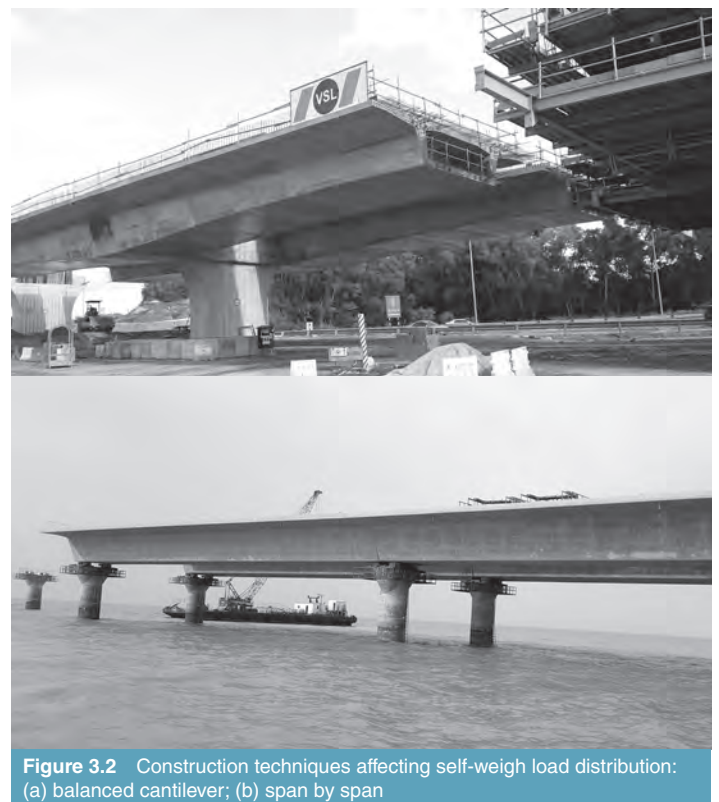


Figure 3.2 Construction techniques affecting self-weight load distribution: (a) balanced cantilever; (b) span by span

Shrinkage and creep of concrete

The shrinkage and creep characteristics of concrete induce internal stresses and deformations in bridge superstructures. Both effects also considerably alter external reactions in continuous bridges. The implications are critical at the serviceability limit state and affect not only the main structural members but also the design of expansion joints and bearings. The drying out of concrete due to the evaporation of absorbed water causes shrinkage. The concrete cracks; where it is restrained, by reinforcing steel or a steel or precast concrete beam, tension stresses are induced, while compression stresses are induced in the restraining element. A completely symmetrical concrete

section will shorten, resulting only in horizontal deformation and a uniform distribution of stresses, but a singly reinforced, asymmetrically doubly reinforced or composite section will be subjected to varying stress distribution and also curvatures, which could exceed the rotation capacity of the bearings. Creep is a long-term effect and acts in the same sense as shrinkage.

The effect is allowed for by modifying the short-term Young modulus of the concrete E_c by a reduction (creep) factor ϕ_c . As for shrinkage, both stresses and deformations are induced.

Shrinkage effects are induced in all concrete and composite bridges, whether they consist of precast elements or are constructed in situ. Generally, the effects are small and are considered insignificant in most cases. However, where concrete is restrained, such as for integral bridges where the columns are built into the deck, or the deck is cast in situ onto a prefabricated member (be it steel or concrete), shrinkage effects can be significant. Figure 3.3 illustrates how shrinkage of the in situ concrete deck affects a composite section.

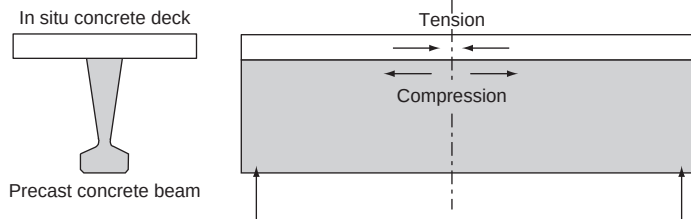


Figure 3.3 Effect of deck slab shrinkage on composite section

In this case, shrinkage produces compression in the top region of the precast concrete beam with tension in the slab and

bottom of the beam. When the deck slab concrete is poured, it flows more or less freely over the top of the precast beam and additional stresses are induced in the beam, owing to the wet concrete. As it begins to set, however, it begins to bond to the top of the precast beam and, because it is partially restrained by the precast beam below, shrinkage stresses are induced in both the slab and the beam.

Creep of concrete happens when it is subjected to any load, causing deformations to occur and redistribution of the self-weight in some cases. Generally, a fully concrete bridge will creep towards the state that would occur if the whole bridge were built instantaneously, redistributing the forces and moments from their as-built distribution towards their instantaneous distribution. The amount of redistribution is governed by the concrete mix, atmospheric conditions, concrete arrangement and construction timing, but can be of the order of 50–70% of the difference between the two distributions.

These effects of shrinkage and creep behaviour are described in more detail in Chapters 9 and 11.

Differential settlement

Differential settlement of the bridge foundations can redistribute the self-weight loading within the structure and cause problems in continuous structures or wide decks that are stiff in the lateral direction. It can result from differing soil conditions at the foundations of the bridge, varying pressures under the foundations or subsidence into voids, such as old mine workings.

Where differential settlements are expected, each foundation

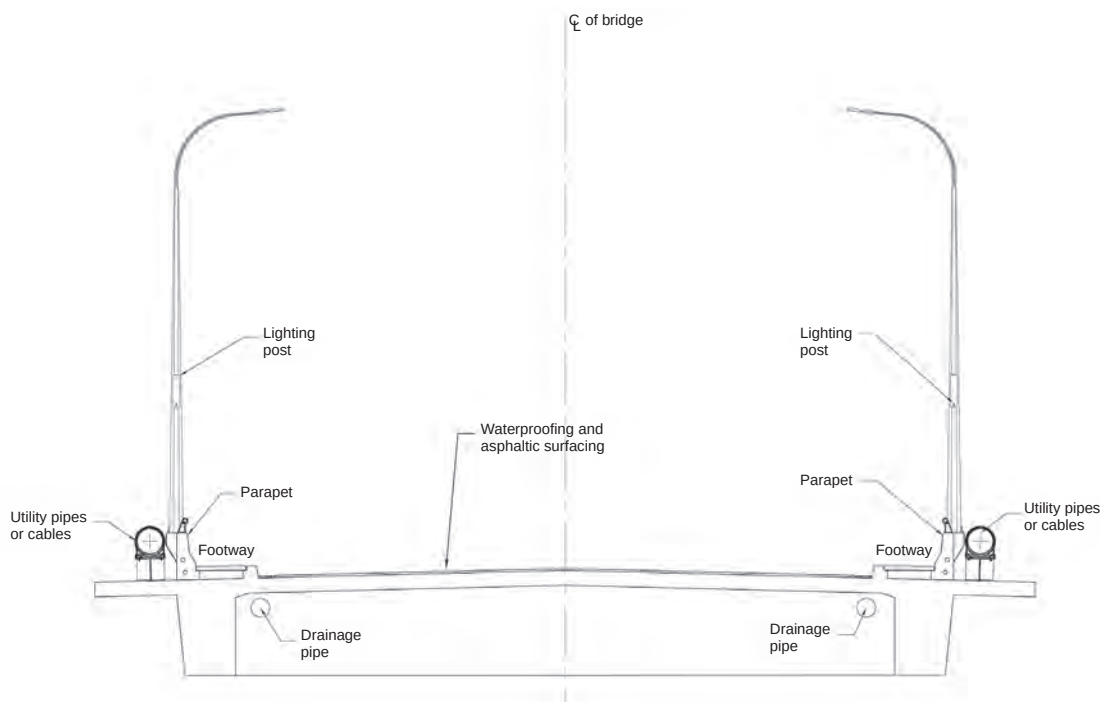


Figure 3.4 Superimposed dead load for a highway bridge

needs to be individually assessed to determine the range of settlements that may occur so that the worst case of expected differential settlement can be considered when designing the bridge.

The differential settlement effects are also modified by creep of the concrete, which can significantly reduce the loads and moments generated; this should be taken into account when considering the bridge design in the long term.

Superimposed dead loads

Superimposed dead loads (SIDLs) are defined as the self-weight of the non-structural elements, which remain essentially unchanged for the life of the bridge. They include the waterproofing, parapets, drainage, utilities, kerbs, footways and lighting posts, as well as surfacing on a road and pedestrian bridge and the trackwork on a rail bridge. Noise barriers or wind shielding can also be present on some bridges.

Highway and pedestrian bridges

A typical section through a highway bridge deck is shown in Figure 3.4 with the normal SIDL elements identified. Pedestrian bridges are similar, although with thinner surfacing and usually much narrower. Whereas the surfacing and parapets are usually required for the bridge to be in operation, any utilities, lighting, drainage or other accessories could be removed during maintenance or replacement, which should be considered in deriving the worst-case loading to be considered in the design.

Item	Weight
Parapets and footways	Derived from take-off of materials
Surfacing (asphalt)	23 kN/m ³
Waterproofing and protection layer	Typically, 20 mm thick, with a weight of 20 kN/m ³
Drainage	Derived from take-off of materials assuming full of water
Lighting	Derived from material take-off
Utilities	Derived from material take-off where available
Noise barriers or wind shields	Derived from material take-off of system used

Table 3.2 Typical highway bridge SIDLs

Road surfacing can vary in thickness, both during its initial application and over the life of the bridge. Variations in the finished deck level can give rise to an additional thickness of asphalt being used to provide a smooth road surface, while asphalt overlays are often applied as part of a road maintenance regime. Often, an additional thickness is allowed for any future overlays, while the load factors applied to this SIDL can be higher to cater for the variations that can occur in practice.

When deriving the SIDL to be applied in the design, the weight of each item needs to be derived based on the proposed configuration and known information. Typical steel and concrete weights used are given in Table 3.1, while Table 3.2 gives indicative values for other items.

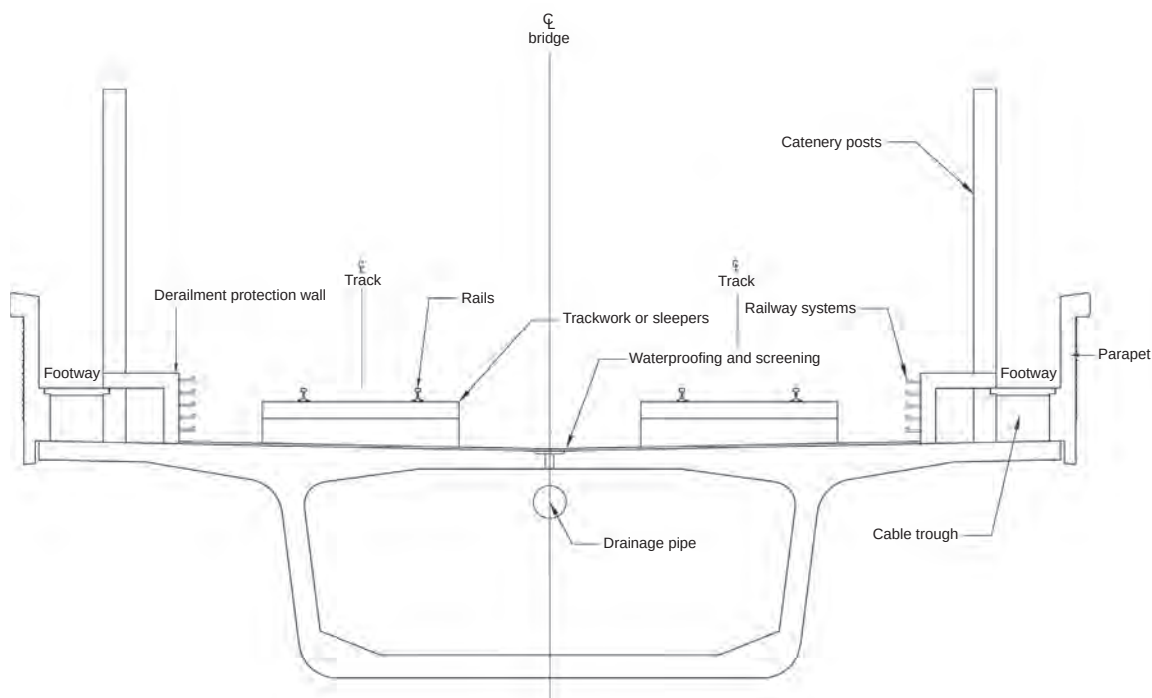


Figure 3.5 Superimposed dead loads for a rail bridge

Rail bridges

Figure 3.5 shows a typical section through a rail bridge deck with the superimposed loads identified. Rail bridges can support ballasted track or tracks supported directly on concrete plinths or slabs. Other SIDLs include rails, sleepers, parapets, drainage, walkways, cable troughs and cabling and third-rail or catenary systems for electric trains. Noise barriers can also be present on some bridges, especially in urban areas. Table 3.3 indicates typical SIDLs for a mass rapid transit (MRT) type system, although these are very much dependent on the train and systems adopted and should always be confirmed by the rail owner before starting any design or assessment.

Description	Weight
Trackform plinths	25 kN/m run/track
Running rails and fixings	2.1 kN/m run/track
Third rail, supports and fixings	0.5 kN/m run/track
Maintenance walkway and cable trough	6.5 kN/m run/track
Cable brackets and cables	2.0 kN/m run/track
Waterproofing and screed	1.5 kN/m ²
Miscellaneous equipment	0.3 kN/m run/track
Ballast	6.0 kN/m ²

Table 3.3 Typical rail bridge SIDL weights

Highway traffic loading

Brief history of highway loading specifications in the UK

Early loads

Over the years, much research and development has, understandably, concentrated on the specification of the live traffic loading model for use in the design of highway bridges. This has been a difficult process, with the aim of producing a simplified static load model that has to account for the wide range and distribution of vehicle types and the effects of bunching and vibration, both along and across the carriageway.

Prior to the Industrial Revolution, in the UK most bridges in existence were single or multispan masonry arch bridges. The live traffic loads consisted of no more than pedestrians, herds of animals and horses and carts, and were insignificant compared with the self-weight of the bridge.

The widespread construction of roads undertaken by J. L. McAdam in the latter half of the eighteenth century and the development of the traction engine brought with them the necessity of building bridges able to carry significant loads (Rose, 1953). In 1875, for the first time in the history of bridge design, a live loading was specified for the design of new road bridges.

This was proposed by Professor Fleming Jenkins (Henderson, 1954) and consisted of '1 cwt per sq. foot [approximately 5 kN/m²] plus a wheel loading of perhaps 10 tons on each wheel on one line across the bridge.' In the early part of the twentieth century, Professor Unwin suggested '120 lbs. per sq. foot [approximately 5.4 kN/m²] or the weight of a heavily loaded wagon, say 10 to 20 tons on four wheels. In manufacturing districts this should be increased to 30 tons on four wheels.'

The development of the automobile and the heavy lorry introduced new requirements. The numbers of vehicles on the roads increased, as did their speed and their weight. In 1904, this prompted the UK Government to specify a rigid axle vehicle with a gross weight of 12 t. This was the 'Heavy Motor Car Order' and was to be considered in all new bridge designs.

Standard highway loading train

The period between 1914 and 1918 marked a new era in the specification of highway loading. The armed forces made demands for heavy mechanical transport. The Ministry of Transport (MOT) was created immediately after the First World War, and in June 1922 introduced the standard loading train (Figure 3.6), which consisted of a 20 ton tractor pulling three 13 ton trailers (similar to loads actually on the roads at the time, as in Figure 3.7) and included a flat rate allowance of 50% on each axle to account for the effects of dynamic impact. This train was to occupy each lane width of 10 ft; where the carriageway exceeded a multiple of 10 ft, the excess load was assumed to be the standard load multiplied by one-tenth of the excess width. The load was therefore uniform in both the longitudinal and transverse directions.

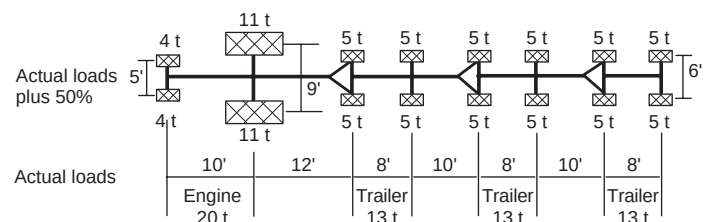


Figure 3.6 Standard load for highway bridges



Figure 3.7 Traction engine plus three trailers, c. 1910

Standard highway loading curve

The standard highway loading train prevailed until 1931, when the MOT adopted a new approach to design loading. It consisted of a uniformly distributed load (UDL) considered together with a single invariable knife-edge load (KEL). Although based on the standard loading train, it was easier to use than a series of point wheel loads.

The UDL was the well-known MOT loading curve shown in Figure 3.8. In view of the improvement in the springing of vehicles at the time and the advent of the pneumatic tyre, the total impact allowance was considered to diminish as the loaded length increased, while a reduction in intensity of loading with increasing span was recognised; hence, the longitudinal attenuation of the curve. The loading was constant from 10 ft to 75 ft and thereafter reduced to a minimum at 2500 ft. For loaded lengths less than 10 ft, a separate curve was produced to cater for the probability of high loads, owing to heavy lorries occupying the whole of the span, where individual wheel loads exert a more onerous effect.

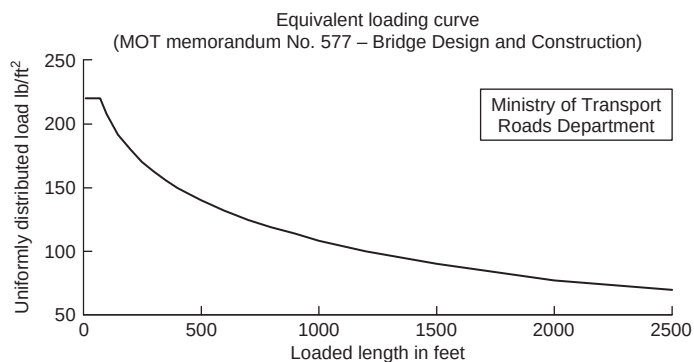


Figure 3.8 Original MOT loading curve (MOT, 1931)

The KEL represented the excess loading on the rear axle of the engine (i.e. $2 \times 11\text{ t} - 2 \times 5\text{ t} = 12\text{ t}$ when considering the vehicle loads shown in Figure 3.6). The UDL was applied to each lane in conjunction with a single 12 t KEL (per lane) to give the worst effect. The MOT also introduced Construction and Use (C&U) Regulations for lorries or trucks, which indicated the legally allowed loads and dimensions for various types of vehicle.

After the Second World War, Henderson (1954) observed that, in reality, the actual vehicles on the roads differed from the standard loading train or standard loading curve. There were those that could be described as 'legal' (i.e. those conforming to the C&U Regulations), and those carrying abnormal indivisible loads outside the Regulations, where special permission was required for transportation. The weight limits in effect at the time were 22 t for the former and 150 t for the latter, although it was possible for hauliers to obtain a special order to move greater loads.

Henderson observed that the abnormal load-carrying vehicles were generally well-deck trailers, having one axle front and rear for the lighter loads and a two-axle bogie at each end

for heavier loads – of which there were about three examples in existence – and that each axle had four wheels and was about 10 ft long

Henderson's conclusion was that 'both ordinary traffic and abnormal vehicles are *dissimilar in weight and arrangement of wheels* to those represented by the former loading trains' (Henderson, 1954). He therefore proposed the idea of defining traffic loads as *normal* (everyday traffic consisting of a mix of cars, vans and trucks) and *abnormal*, the latter consisting of heavy vehicles of 100 t or more. The abnormal loading could consist of two types, namely those conforming to the current C&U Regulations and those less-frequent loads in excess of 200 t. The latter loads would be confined to a limited number of roads and would be treated as special cases. Bridges en route could be strengthened, and precautions taken to prevent heavy normal traffic on the bridge at the same time.

In conjunction with the MOT and the British Standards Institution (BSI), Henderson developed the idea of considering two kinds of loading for design purposes, namely normal and abnormal, and that 'designs should be made on the basis of normal loading and checked for abnormal traffic'.

Normal highway loading

The widely adopted MOT loading curve with a UDL plus a KEL would constitute normal loading, defined as HA loading. Experience showed the extreme improbability of more than two carriageway lanes being filled with the heaviest type of loading, and although no qualitative basis was possible, Henderson proposed that two lanes should be loaded with the full UDL and the remainder with one-half the UDL, as shown in Figure 3.9.

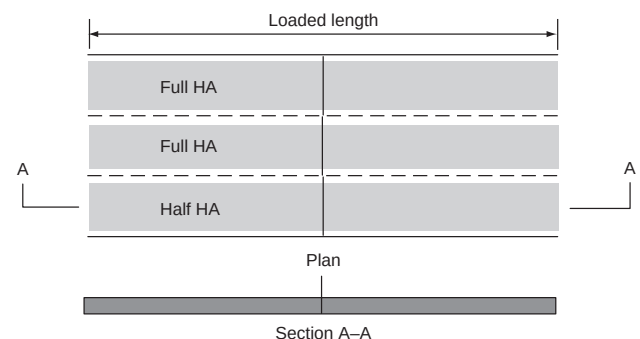


Figure 3.9 Normal loading

Any attempt to state a sequence of vehicles representing the worst concentration of ordinary traffic that can be expected must have been a guess, but it seemed reasonable to propose the following:

- 20 ft (6 m) to 75 ft (22.5 m) – lines of 22 t lorries in two adjacent lanes and 11 t lorries in the remainder.
- 75 ft (22.5 m) to 500 ft (150 m) – five 22 t lorries over 40 ft (12 m) followed and preceded by four 11 t 5 ft (10.5 m) and 5 t vehicles over 35 ft (10.5 m) to fill the span.

These were found to correspond well to the MOT loading curve. For spans in excess of 75 ft (22.5 m), an equivalent UDL (in conjunction with a KEL) was derived by equating the moments and shear per lane of vehicles with the corresponding effects under a distributed load. Henderson emphasised that these loadings could be looked on only as a guide. A 25% increase was considered appropriate for the impact of suspension systems.

A more severe concentration of load was considered appropriate for short-span members and units supporting small areas of deck. A heavy steam roller had wheel loads of about 7.5 t, similar to the weight of the then 'legal' axle; adding 25% for impact gave 9 t. It seemed suitable to use two 9 t loads at 3 ft (0.915 m) spacing on such members. Separate loading curves were proposed to give a UDL on the basis of this loading.

Abnormal loading

Henderson (1954) proposed that abnormal loading be referred to as HB loading, defined by the now-familiar HB vehicle, which, although hypothetical, was based on existing well-deck trailers having two bogies, each with two axles and four wheels per axle. Each vehicle was given a rating in units (1 unit being 1 t) and referred to the load per axle. Thus, 30 units meant an axle load of 30 t. Henderson proposed 30 units for main roads and at least 20 units on other roads. In 1955, because of the increasing weights of abnormal loads, the upper limit was increased to 45 units. Since abnormal vehicles travel slowly, no impact allowance was made.

Variations

The standard loading curve has undergone several revisions over the years, as more precise information about traffic volumes and weights has been gathered and processed. The basic philosophy of the normal and abnormal loads has been retained; indeed, a colloquium convened at Cambridge in 1975 to examine the basic philosophy concluded that the status quo should be maintained (IABSE, 1975). This is still the current view and the major changes that have taken place are reflected in BS 153 (BSI, 1954), BE 1/77 (DoT, 1977); BS 5400 (BSI, 1978) and

Memorandum BD 37/01 (DoT, 1988), which each contained the HA loading model of a UDL in conjunction with a KEL.

One interesting phenomenon that has occurred over the years is that the maximum permitted lorry load to be included in the HA loading has increased significantly from the original 12 t to 40 t in 1988. The increase with time is illustrated in Figure 3.10. In 2000, the maximum was raised to 44 t (in certain circumstances) by the Road Vehicles Regulations, in line with EC Directive 96/53/EC. The highest limit is in the Netherlands at 50 t on five or six axles.

Since 2010, the Eurocode standards have been adopted in the UK, although the UK National Annex creates a very different loading regime from that used in other countries adopting the Eurocodes.

Eurocode highway loading

The European models for traffic loading are embodied in Eurocode 1, Part 2 (BSI, 2003) and are identified in Table 3.4. These consist of a series of UDLs and wheel loading to be applied across a number of lanes arranged on the carriageway.

Load model	Definition
LM1	General (normal) loading due to lorries or lorries plus cars
LM2	A single axle for local effects
LM3	Special vehicles for the transportation of exceptional loads
LM4	Crowd loading

Table 3.4 European load definitions

Load models LM1 and LM2 are the most commonly used for bridge design, while the bridge owner may introduce special vehicles to supplement those given for LM3 where they consider it necessary, or for special usage of the bridges.

Traffic lanes

The full carriageway width, w , between kerbs or the inside face of the barriers is considered when assigning the notional lanes for design purposes. The notional lane width is generally taken as 3 m, and the number of notional lanes as $\text{Int}(w/3)$, as indicated in Figure 3.11. For carriageway widths between 5.4 m and 6.0 m, two notional lanes are taken with a width of $w/2$ each. Otherwise, only 3 m wide notional lanes are taken and all areas other than those covered by notional lanes are referred to as remaining areas. Any road markings delineating the physical traffic lanes are ignored when considering the position of the notional lanes which are located to give the most adverse effects from the analysis carried out.

The lane numbering is used to allocate the loading to give the most adverse effects, usually with one lane more heavily loaded than the other lanes. Where a single deck carries two

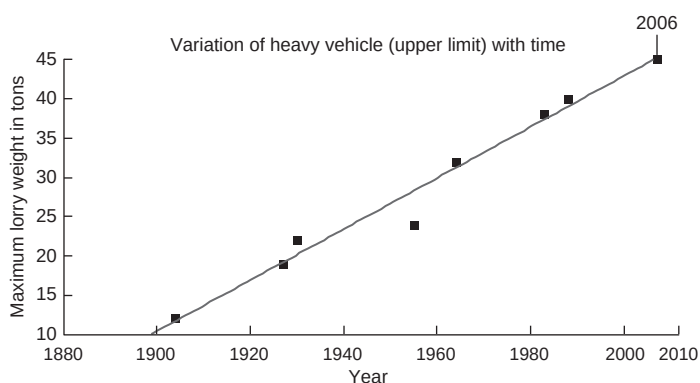


Figure 3.10 Variation of heavy vehicle load with time

or more separate parts of carriageway, only one numbering sequence is used across all parts. Where carriageways are on separate independent decks, each part of the carriageway is numbered separately.

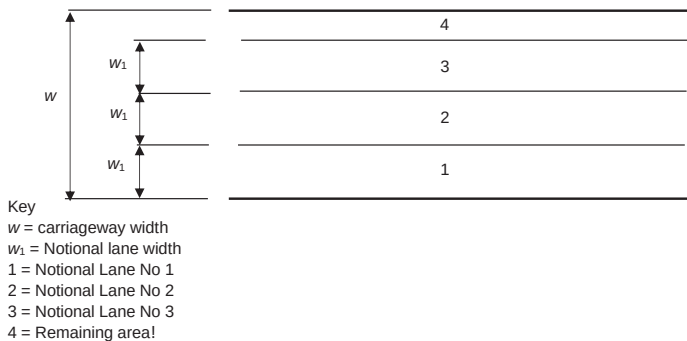


Figure 3.11 Eurocode notional lane layout

General loading

The general LM1 loading comprises a UDL (in kN/m^2) plus a double-axle tandem per lane, as shown in Figure 3.12, which includes the dynamic amplification that may occur with traffic loading. The tandem is dispensed with on the fourth lane and above, on carriageways of four lanes or more. Adjustment factors, α , can be used to reflect expected traffic and different classes of routes, with a wide range of values of α adopted throughout Europe.

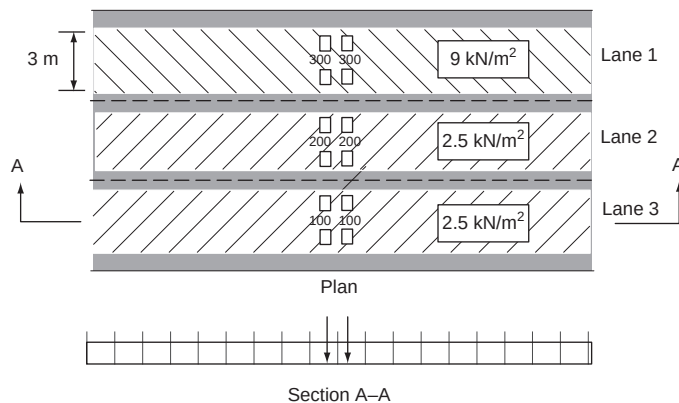


Figure 3.12 General loading model (LM1) to European Code EC1

For the standard Eurocode loading, the first lane is the most heavily loaded, with a UDL of 9 kN/m^2 (equivalent to a lane loading of 27 kN/m for a 3 m wide notional lane) plus a single tandem with axle loads of 300 kN each. The loads on remaining lanes and in the remaining area reduce, as indicated in Figure 3.12. In the UK, the UDLs adopted are 5.5 kN/m^2 for the first lane and all subsequent lanes, with the tandem loads remaining the same.

The LM2 loading is used to study local effects when the use of a 400 kN tandem axle is recommended, as shown in Figure 3.13. In certain circumstances, this can be replaced by a single

wheel load of 200 kN . The UK National Annex adopts a wheel contact area of 0.4 m^2 . The load is assumed to be distributed at 45° through the surfacing and concrete down to the support element's neutral axis.

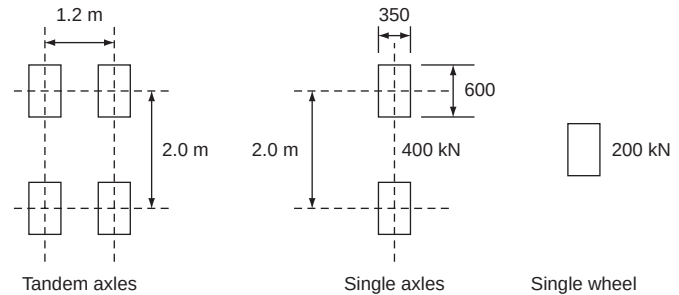


Figure 3.13 Tandem axle used in LM1 and single axle or wheel load used in LM2

The Eurocode load models are developed for use with loaded lengths less than 200 m , although the UK National Annex increases this to 1500 m , which covers most bridge designs.

When used with characteristic values, these LM1 and LM2 loads are based on a 1000 year return period, and when used with frequent values they are based on a 1 week return period.

Eurocodes provide separate load models for calculation of fatigue lives of bridges, including a reduced LM1 loading and a range of specific truck configurations with indicative frequencies.

Abnormal loads

Abnormal loads are considered in a similar manner to that developed in the UK, with a special abnormal load (model LM3) placed in one lane (or straddling two lanes), with a 25 m clear space front and back and normal LM1 loading placed in the other lanes. The vehicle may be specified by the particular road authority involved; alternatively, it may be as defined in EC1, which specifies eight load configurations with varying numbers of axles and loads from 600 kN to 3600 kN . Wheel areas are assumed to merge to form long areas of $1.2 \text{ m} \times 0.15 \text{ m}$. Axle lines are spaced at 1.5 m and may consist of two or three merged areas. A typical configuration for an 1800 kN vehicle is shown in Figure 3.14.

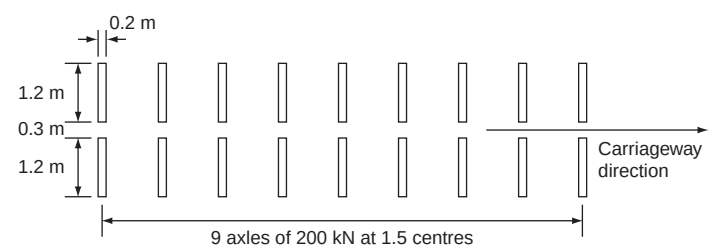


Figure 3.14 Typical LM3 vehicle (in this case 1800 kN)

The UK National Annex presents three basic LM3 special vehicle models, the SV80, SV100 and SV196 with total

weights of 80t, 100t and 196t, respectively, and with axle loads range from 100kN to 180kN. All three load configurations are considered for motorways and trunk roads, while SV80 and SV100 are considered for principal roads and only SV80 for minor roads. A number of special order vehicles are also given in the UK National Annex, ranging from 250t to 600t total weight, with axle loads up to 225kN, and are used on bridges where required by the bridge owner for special routes. These represent some of the abnormal loads that can be transported from major ports to industrial plants. These axle loads are increased for dynamic effects depending on the axle weights and varying from 1.2 to 1.07 for axle loads between 100kN and 225kN, respectively.

Secondary traffic loads

Secondary traffic loading emanates from the tendency of traffic to change speed or direction and results in horizontal forces applied either at deck level, from braking, traction skidding or centrifugal effects.

Braking and acceleration

This is considered as a group effect as far as traffic UDLs are concerned; it is assumed that the traffic in one lane brakes or accelerates simultaneously over the entire loaded length. The effect is considered as a longitudinal force applied at the road surface. There is considerable evidence to suggest that braking and forces are dissipated to a considerable extent in plan and, for most concrete and shallow deck structures, it is reasonable to consider the load spread over the entire width of the deck.

The longitudinal braking and acceleration loads, Q_{lk} are considered to be of the same magnitude but opposite in direction and related to the lane UDL vertical loads, Q_{lk} and q_{lk} , and the length of load, L , up to a maximum load of 900kN:

$$Q_{lk} = 0.6 \alpha_{Ql} (2 Q_{lk}) + 0.1 \alpha_{q_l} q_{lk}, \quad w_1 L \leq 900 \text{ kN} \quad (3.1)$$

For the LM3 load model, the braking and traction are specified in the UK National Annex as follows.

- Braking = braking factor $\times$ axle load, with the braking factor being 0.5, 0.4, 0.25 and 0.2 for the SC80, SV100, SV196 and other SOV vehicles, respectively.
- Acceleration = 10% of gross weight, distributed in the same proportion to vertical loads.

The significance of the braking load on the structure is mainly in the design of the bearings, piers and abutment walls.

Skidding or skew braking load

This is an accidental load, consisting of a transverse force equal to 25% of the braking or acceleration acting simultaneously with the longitudinal loads. The UK National Annex increases this to 50% for loaded lengths up to 120m and 280kN above that.

Centrifugal loads

These loads are important only on elevated curved superstructures with a radius less than 1500m. Above that they are usually ignored. They generate transverse forces on the structure as the traffic travels around a horizontal curve and are based on the centrifugal acceleration (α = velocity²/radius of curve) which, when substituted in Newton's second law gives

$$F = mv^2/r \quad (3.2)$$

which acts at the centre of mass of the vehicle in an outward horizontal direction.

If the weight of the vehicle is W , then

$$F = Wv^2/gr \quad (3.3)$$

Eurocodes simplify this by defining the centrifugal forces, Q_{lk} , as shown in Table 3.5, given in terms of the total maximum weight of the vertical loads of the LM1 tandem system, Q_v , and the radius of curvature, r . The centrifugal force is applied as a point load at any position along the deck.

Highway curvature radius, r : m	Centrifugal force, Q_{lk} : kN
<200	$0.2Q_v$
$200 \leq r \leq 1500$	$40Q_v/r$
1500	0

Table 3.5 Eurocode centrifugal forces

The UK National Annex adopts this formula when considering the LM3 model loads, with the velocity dependent on the special vehicle weight and the centrifugal force distributed between the axles and wheels in the same proportion as the vertical loads.

BS 5400 and BD 37/01 highway loading

Although BS 5400 (BSI, 2006a) and BD 37/01 (DoT, 1988) are not used for new bridge designs in the UK, they are still commonly used in other countries around the world and can be used for assessing existing bridges in the UK. The history behind the development of these codes is explained earlier in this chapter, with the latest highway loading adopted as follows.

Normal load application

The normal load consists of a lane UDL plus a lane KEL. The UDL (HAU) is based on the loaded length and is defined by a two-part curve, as shown in Figure 3.15, each defined by a particular equation, one up to 50m loaded length and the other for the remainder up to 1600m. The KEL (HAK) has a value of 120kN per lane.

The application and intensity of the traffic loads on the bridge deck depends on:

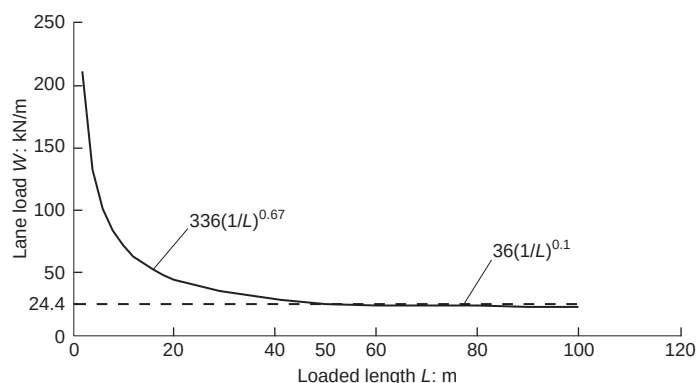


Figure 3.15 British Standard normal loading curve

- the carriageway width
- the loaded length
- the number of loaded lanes.

The carriageway width is essentially the distance between kerb lines and is described in Figure 1 of BD 37/01 (DoT, 1988). It includes the hard strips, hard shoulders and the traffic lanes marked on the road surface.

The attenuation of the curve in Figure 3.15 takes account of vehicle bunching along the length of a bridge. Lateral bunching is taken account of by applying lane factors β to the load in each lane (both the UDL and the KEL). Generally, this amounts to $\beta=1.0$ for the first two lanes and $\beta=0.6$ for the remainder. Thus, nominal lane load = $\beta\text{HAU} + \beta\text{HAK}$.

The number of lanes (called notional lanes, and not necessarily the same as the actual traffic lanes defined by the carriageway marking) is based on the total width (b) of the carriageway (the distance between kerbs in metres) and is given by $\text{Int}[(b/3.65) + 1]$ where 3.65 is the standard lane width in metres. Notional lanes are numbered from a free edge.

The two most prominent load applications are defined as HA only, and HA + HB; HA is applied as described previously to every (notional) lane across the carriageway attenuated, as defined in Table 14 of BD 37/01 (DoT, 1988).

Local effects

For parts of a bridge deck under the carriageway that are susceptible to the local effects of traffic loading, a wheel load is applied equivalent to either 45 units of HB or 30 units of HB, as appropriate to the bridge being considered. Alternatively, an accidental wheel load of 100 kN is applied away from the carriageway on such areas as verges and footpaths. The wheel load is assumed to exert a pressure of 1.1 N/mm² to the surfacing and is generally considered as a square of 320, 260 or 300 mm side for 45 units of HB, 30 units of HB or the accidental wheel load, respectively. Allowance can also be made for dispersal of the load through the surfacing and the structural concrete if desired.

Abnormal HB loading

The loading for the abnormal vehicle is concentrated on 16 wheels arranged on four axles, as shown in Figure 3.16. Its weight is measured in units per axle, where 1 unit = 10 kN. The maximum number of units applied to all motorways and trunk roads is 45 (equivalent to a total vehicle weight of 1800 kN), and the minimum number is 30 units applied to all other public roads. The inner axle spacing can vary to give the worst effect, but the most common value taken is 6 m. It is worth noting that vehicles with this configuration are not considered in the Construction and Use Regulations because it is a hypothetical vehicle and used only as a device for rating a bridge in terms of the number of HB units it can support. Each wheel area is based on a contact pressure of 1.1 N/mm².

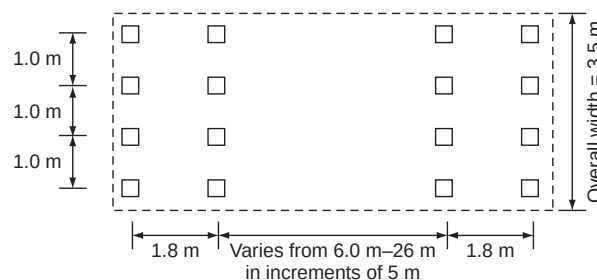


Figure 3.16 Abnormal HB vehicle

Load application

All bridges are designed for HA loading and checked for a combination of HA + HB loading; HA and HB are applied according to Figure 13 of BD 37/01 (DoT, 1988) with the HB vehicle placed in one lane or straddled over two lanes (depending on the width of the notional lane). Since such a load would normally be escorted by police, an unloaded length of 25 m in front and behind is specified, with HA loading occupying the remainder of the lane. The other lanes are loaded with an intensity of HA appropriate to the loaded length and the lane factor.

Exceptional loads

Road hauliers are often called on to transport very heavy items of equipment, such as transformers or parts for power stations or chemical plants, which can weigh up to 750 t (7500 kN), or even more. Special flatbed trailers are used with a number of axles and many wheels to spread the load, so that the overall effect is generally no more than that of HA loading, and contact pressures are no more than 1.1 N/mm², but where this is not possible, any bridges crossed en route have to be strengthened. The loads on the axles can be relieved by the use of a central air cushion, which raises the axles slightly and redistributes some of the load to the cushion. Heavy diesel traction engines placed in front and to the rear are used to pull and push the trailer. Some typical dimensions are shown in Figure 3.17.

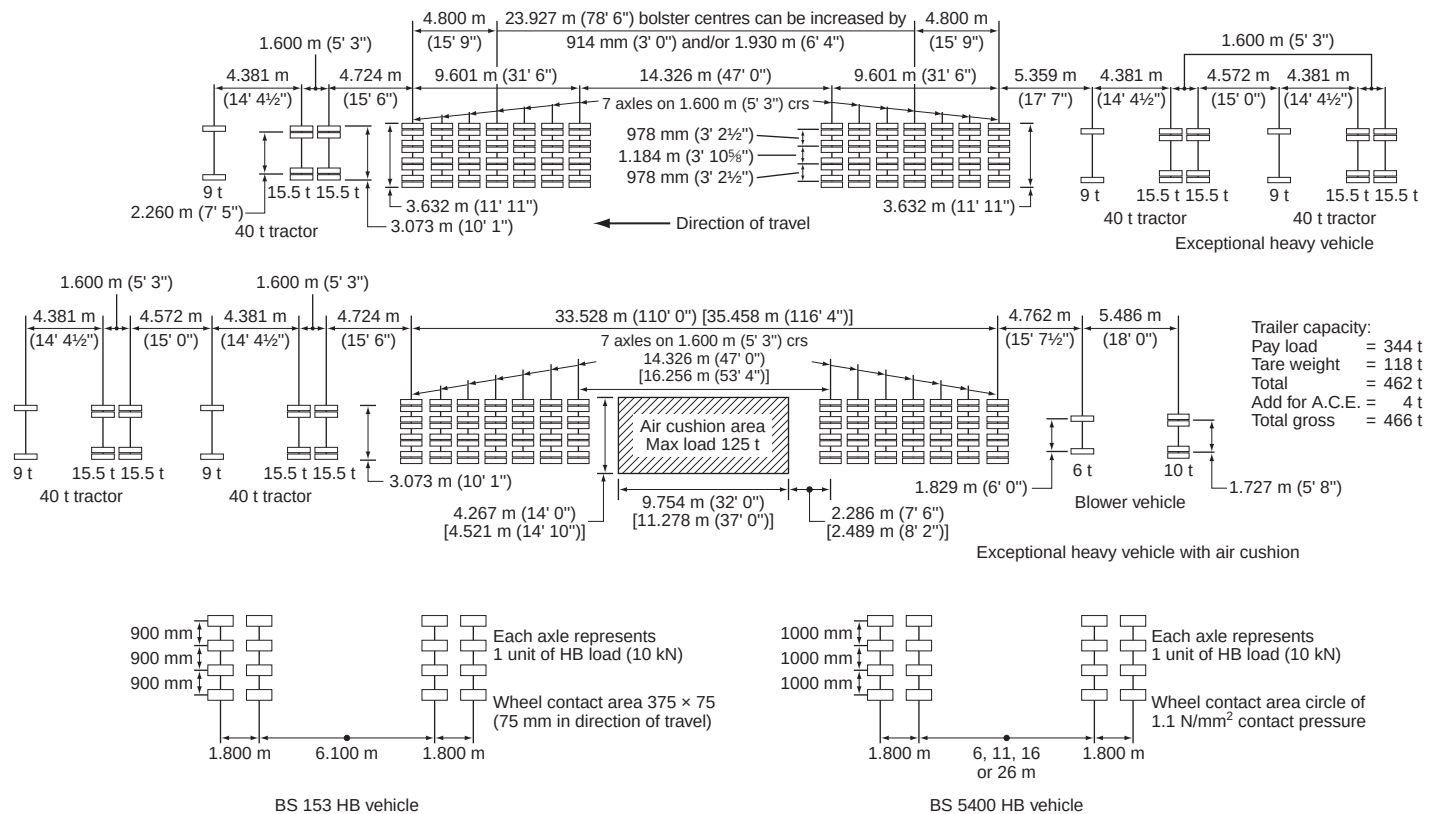


Figure 3.17 Typical vehicles used to transport exceptional loads (after Pennells, 1978)

Braking and traction

Similar to Eurocodes, BS 5400 (BSI, 2006a) and BD 37/01 (DoT, 2001) define the braking and traction loads to be applied in a bridge design. For HA loads, this is applied over the loaded length, and for HB at the axle locations, with the loads as follows:

- HA = 8 kN/m of loaded length + 250 kN but not greater than 750 kN
- HB = nominal HB load $\times 0.25$.

Skidding load

This accidental load consists of a single point load of 300kN acting horizontally in any direction at the road surface in a single notional lane. It is considered to act with the primary HA loading in Combination 4 only.

Centrifugal loads

The centrifugal forces are discussed further in the section under Eurocodes, while in BS 5400 (BSI, 2006a) and BD 37/01 (DoT, 2001) they are taken as a nominal load of

$$F_c = 400000 / (r + 150) \quad (3.4)$$

which approximates to a 40t (400kN) vehicle travelling at 70 miles per hour.

Each centrifugal force acts as a point load in a radial direction at the surface of the carriageway and parallel to it and should be applied at 50m centres in each of two nominal lanes, each in conjunction with a vertical live load component of 400kN (Figure 3.18).

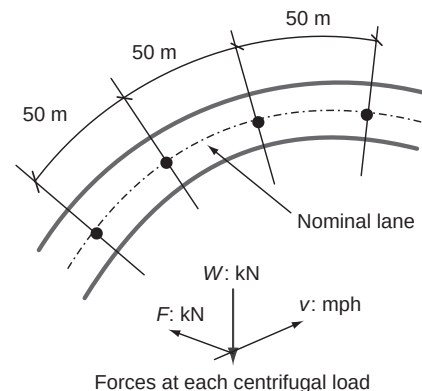


Figure 3.18 Centrifugal forces

US highway specifications

AASHTO highway loading

The US highway loads are based on American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2020) or, previously, the Standard Specification for Highway Bridges (AASHTO, 1996), which are similar. These specify the standard lane and truck loads to be adopted in the bridge design.

Lane loading

The commonly applied lane loading consists of a UDL plus a KEL on 'design lanes' typically 3.6 m wide placed centrally on the 'traffic lanes' marked on the road surface. The number of 'design lanes' is the integer component of the carriageway width/3.6. Traffic lanes less than 3.6 m wide are considered as design lanes with the same width as the traffic lanes. Carriageways between 6 m and 7.3 m are assumed to have two design lanes.

The lane load is constant, regardless of the loaded length, and is equal to 9.3 kN/m, which occupies a region of 3 m transversely, as indicated in Figure 3.19. Frequently, the lane load is increased by a factor of between 1.3 and 2.0 to reflect the heavier loads that can occur in some regions.

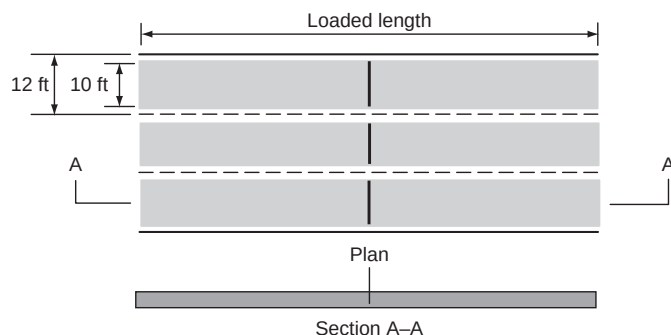


Figure 3.19 AASHTO simple lane loading

Truck loading

Acting with the lane loading, there are two different design truck loadings, namely

- tandem
- truck.

The new (AASHTO, 2020) tandem and truck loadings are shown in Figure 3.20 compared with the old (AASHTO, 1977) standard H and HS trucks.

To account for the fact that trucks will be present in more than one lane, the loading is further modified by a multiple presence factor, m , according to the number of design lanes,

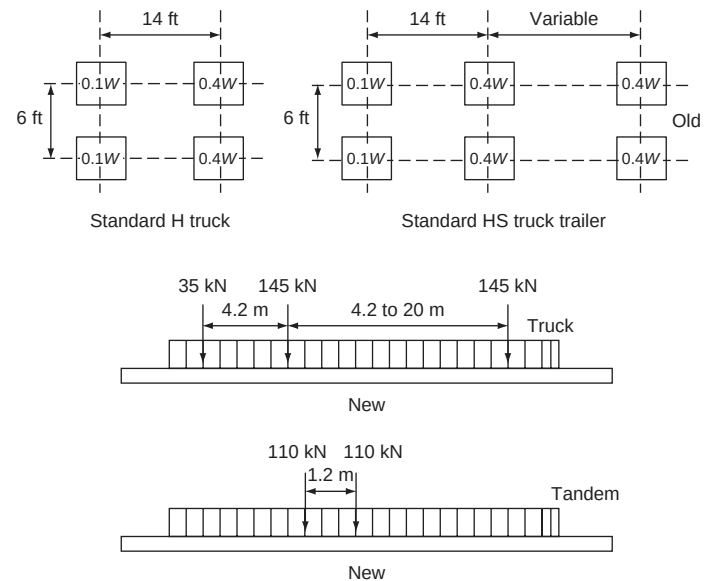


Figure 3.20 New (LRFD) and old (standard) AASHTO truck loadings

and ranges from 1.2 for one lane to 0.65 for more than three lanes (AASHTO, 2020).

The actual intensity of loading depends on the class of loading, as indicated in Table 3.6. The prefix H refers to a standard two-axle truck followed by a number that indicates the gross weight of the truck in tons, and the affix refers to the year the loading was specified. The prefix HS refers to a three-axle tractor (or semi-trailer) truck. The dimensions and wheel loadings of the two types of truck are shown in Figure 3.20 where W is the gross weight in tons.

Class of loading, 1944	Class of loading, 1993
H 20-44	HL 20-93
H 15-44	HL 15-93
HS 20-44	HLS 20-93
HS 15-44	HLS 15-93

Table 3.6 Class of loading – USA

Dynamic effects

Dynamic effects caused by irregularities in the road surface and different suspension systems magnify the static effects from the vehicles; this is accounted for by an impact factor called a dynamic load allowance (DLA) defined as:

$$DLA = D_{\text{dyn}}/D_{\text{sta}} \quad (3.5)$$

where D_{sta} is the static deflection under live loads and D_{dyn} is the additional dynamic deflection under live loads. This is applied to the static live load effect using the following equation:

$$\text{Dynamic live load effect} = (\text{static live load effect}) \times (1 + DLA) \quad (3.6)$$

Values of the DLA are given in AASHTO (2020) for individual components of the bridge, such as deck joints, beams and bearings, and global effects are not considered at all. This is a departure from the old practice, where the basic static live load was multiplied by an impact factor:

$$I = 50/(L + 125) \quad (3.7)$$

where L is the loaded length in feet and the maximum value of I allowed is 0.3.

The variable spacing of the trailer axles in the truck trailer is to allow for the actual values of the more common tractor trailers now in use.

Braking

The braking loads to be applied in a bridge design are defined in AASHTO (2020) as the greater of

- 25% of axle weight of design truck or tandem
- 5% of design truck or tandem with the lane load.

Centrifugal loads

Centrifugal forces are discussed further in the section under Eurocodes, while in AASHTO (2020) they are taken as a nominal lateral load equal to the axle weight of the design truck or tandem multiplied by the factor C , defined for normal loading as

$$C = 0.133v^2/R \quad (3.8)$$

where v is the design speed (in m/s) and R is the radius of curvature (in m).

The centrifugal force is applied as a horizontal load along the radius of the curve and a distance of 1.8m above the road surface.

Modern trends

The modern trend towards traffic loading is to try to model the movement, distribution and intensity of loading in a probability-based manner (Bez and Hirt, 1991). Stopped traffic is considered, representing a traffic jam situation consisting of semi-trailers, tractor trailers and trucks, and is related to the response of the bridge structure in a random manner. From this it is possible to determine the mean value and standard deviation of the maximum bending moment in the bridge. Different models are considered at both the ultimate limit state (ULS) and serviceability limit state (SLS) conditions. Vrouwenvelder and Waarts (1993) have conducted similar research in order to construct a probabilistic traffic flow model for the design of bridges at the ultimate limit state, in both the long and the short term. The loading that they arrived at can be transformed into a uniform load in combination with one or more movable

truck loads. Bailey and Bez (1996) studied the effect of traffic actions on existing load bridges with the idea of developing the concept of site-specific traffic loads. They considered the random nature of the traffic and the simulation of maximum traffic action effects and developed correction factors for application to the Swiss design traffic loads. Studies have also been carried out in the UK (Cooper, 1997; Page, 1997), through the collection of traffic data and the application of reliability methods for both assessment and design, but, for the foreseeable future, the simple lane loading of a UDL plus a KEL is set to continue to be the model adopted in practice.

Highway live load distribution in decks

Traffic loads on bridge decks are distributed according to the stiffness, geometry and boundary conditions of the deck. The deflection of a typical beam-and-slab deck under an axle load is shown in Figure 3.21.

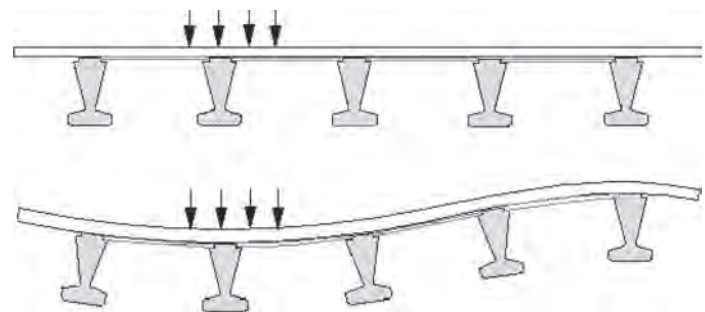


Figure 3.21 Typical transverse bending due to eccentric traffic load

For a single-span right deck on simple supports with different stiffnesses in two orthogonal directions, it is possible, using classical plate theory, to determine the load distributed to each member. If the amount of load carried by the most heavily loaded member can be found, the bending moment can be easily calculated.

The very first attempts at analysing bridge decks pioneered by Guyon (1946) and Massonet (1950) were aimed at simplifying the process for practising engineers using the method of distribution coefficients, that is, the calculation of the distribution of live loads to a particular beam (or portion of slab) as a fraction of the total. The method was developed in the UK by Morice and Little (1956), Rowe (1962) and Cusens and Pama (1975). It was later refined by Bakht and Jaeger (1985) of Canada and has actually been codified in the USA (AASHTO, 1977) and Canada (Ministry of Transportation and Communications, 1983).

The basic assumption of the distribution coefficient (or D-type) method is that the distribution pattern of longitudinal moments, shears and deflections across a transverse section is independent of the longitudinal position of the load and the transverse section considered (Bakht and Jaeger, 1985; Ryall, 1992). This is illustrated in Figure 3.22.

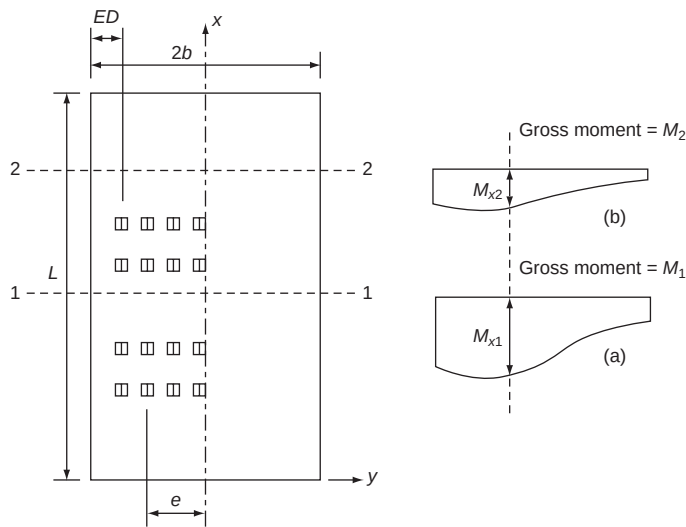


Figure 3.22 Transverse distribution of longitudinal moments at two sections due to traffic

The implication is that

$$M_{x1}/M_1 = M_{x2}/M_2 \quad (3.9)$$

where M_1 and M_2 are the gross moments at sections 1 and 2, respectively. For convenience, the maximum longitudinal bending moment M_{sw} from a single line of wheels of a standard vehicle is determined and is multiplied by a load fraction S/D to give the design moment for the bridge; thus, S/D is the proportion of the bending moment from a single line of wheels carried by a particular beam.

This can be seen by reference to Figure 3.23, where each coordinate of the distribution diagram represents the longitudinal moment per unit width of the deck.

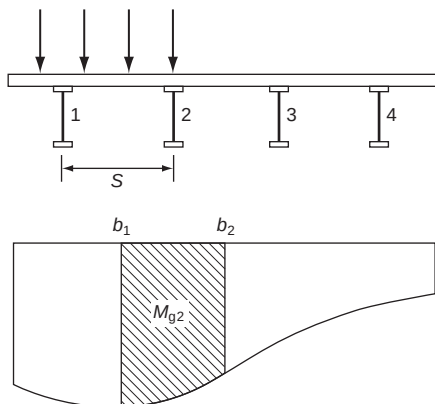


Figure 3.23 Proportion of total moment to be resisted by a given beam

If the total area under the curve represents the gross bending moment at the section due to the design vehicle, the total moment sustained by girder 2, for example, is represented by the hatched area, such that

$$M_{g2} = M_{x2} d_y \quad (3.10)$$

and

$$M_{av} = M_{g2}/S \quad (3.11)$$

If it is assumed that, for a particular bridge and design vehicle, a factor D (in terms of width) is known, such that

$$D = M_{sw}/M_{av} \quad (3.12)$$

then substituting for M_{av} gives

$$M_{g2} = SM_{sw}/D \quad (3.13)$$

The calculation of M_{sw} is trivial, so that if the value of D is known, it is a simple matter to determine M_{g2} . The distribution coefficients (D) are calculated by solving the well-known partial differential plate equation:

$$D_x + 2H + D_y = p(x, y) \quad (3.14)$$

where

$$2H = D_{xy} + D_{yx} + D_1 + D_2 \quad (3.15)$$

The solution is achieved numerically by satisfying the boundary conditions with the use of harmonic functions to represent the load and by assuming a sinusoidal deflection profile (Bakht and Jaeger, 1985). This is tantamount to idealising the deck as a continuum.

Apart from a concrete slab bridge deck, the continuum idealisation is, although better than finite elements, not strictly correct. Beam-and-slab decks, however, have physical characteristics that can be better represented in a semi-continuum way, that is to say that the transverse stiffnesses of the slab can be spread uniformly along the length of the bridge, while the longitudinal stiffnesses can be concentrated at locations across the width of the deck defined by the beam positions. Bakht and Jaeger (1985) have described this in detail.

Controlling parameters

Past research (Bakht and Jaeger, 1985) has shown that, apart from the pattern of live loads, the main factors affecting the transverse distribution of longitudinal bending moments are the flexural and torsional rigidities, the width of the deck ($2b$) and the edge distance (ED) of the standard vehicle. Furthermore, bridge decks in general can be defined by two non-dimensional characterising parameters:

$$\alpha = H/(D_x D_y)^{0.5} \quad (3.16)$$

and

$$\theta = b(D_x/D_y)^{0.25}/L \quad (3.17)$$

where D_x , D_y , D_1 , D_2 , D_{xy} and D_{yx} are the flexural and torsional rigidities of the deck per metre. In defining a particular bridge, all that is required are the values of α and θ . The usual range of α for concrete beam-and-slab, composite beam-and-slab and concrete slab decks is from 0.05 to 1.0, and the range of θ is

from 0 to 2.5. When analysing a bridge, the dimensions and rigidities are usually known; therefore, α and θ can be calculated. It should be evident that these calculations take very little time – far less than all the data preparation required to run a grillage analysis – and providing that the distribution coefficient D can be easily obtained from either pre-prepared tables or from a computer program, the critical bending moment is soon obtained. Tables of distribution coefficients for different types of live loading and ranges of characterising parameters can be prepared using suitable software.

The method can be utilised by using either predocumented tables of distribution coefficients, which can be incorporated into standard codes of practice, or a computer program to analyse a particular bridge. In either case, the data preparation is kept to a minimum, and only useful output data are generated.

The method can be used for design or assessment purposes to determine the value of critical moments under any load specification, such as the UK, AASHTO and EC1 loadings.

Example with a beam-and-slab deck

A reinforced concrete slab on prestressed concrete Y-beams will illustrate the method. The span is 11 m, the carriageway is 9 m and it is subject to 45 units of an HB vehicle. The deck is shown in Figure 3.24. The values of D_x , D_y , D_{xy} and D_{yx} are based on the beam and slab longitudinally, but on the slab only transversely, from which α and θ are calculated, as shown in Table 3.7.

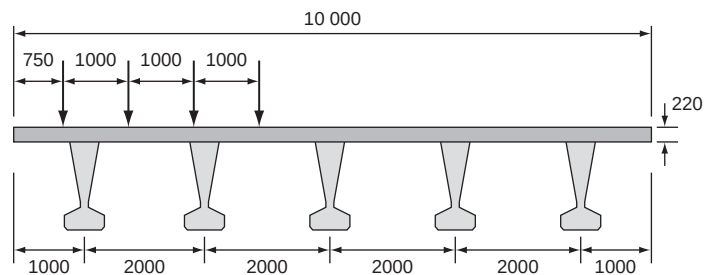


Figure 3.24 Prestressed concrete beam and reinforced concrete deck

Deck	D_x	D_y	D_1	D_2	D_{xy}	D_{yx}	H	α	θ
Prestressed concrete or concrete	4.69	0.03	0.01	0.01	0.13	0.02	0.08	0.23	0.82

Table 3.7 Deck properties ($\times 10^{12}$ kN mm²/m) for example

Tables of D can be generated very simply to account for the number of lanes, the type and intensity of loading and the edge distance of the loading; Table 3.7 is appropriate for this example.

The load in each case was that from 45 units of an HB vehicle taken from BD 37/01 (DoT, 2001), and placed in an outside lane so as to induce the worst possible longitudinal

bending moment. From Table 3.8, the distribution coefficients for the deck can be interpolated as 1.12. Then if the moment at the critical midspan section due to a single line of wheels from the reference vehicle (i.e. the HB vehicle) is calculated as 1644 kNm, then the moments in the most heavily loaded girder (the edge girder) are

$$M_{g1} = 2 \times 1644 / 1.12 = 2936 \text{ kNm} \quad (3.18)$$

Using the grillage method, the maximum moment was calculated as 3065 kNm.

9 m carriageway; 0.75 edge distance; 45 units of HB only								
α	θ							
	0.20	0.30	0.40	0.5	0.60	0.70	0.80	0.90
0.05	1.22	1.11	1.07	1.05	1.05	1.05	1.06	1.09
0.10	1.41	1.22	1.14	1.1	1.08	1.07	1.08	1.10
0.15	1.49	1.28	1.18	1.13	1.10	1.08	1.09	1.10
0.20	1.60	1.37	1.25	1.18	1.14	1.11	1.11	1.12
0.25	1.69	1.45	1.31	1.23	1.17	1.14	1.13	1.14
0.30	1.76	1.52	1.37	1.27	1.21	1.17	1.16	1.16

Table 3.8 Distribution coefficients D

The maximum difference between each of the methods is 4.2%, which by any standard is quite acceptable. It could be argued that the distribution method is more accurate, as it more closely models the deck as a continuum.

The main advantage of the D -type method over the traditional grillage and finite-element analysis (FEA) methods is its speed and simplicity. The initial data required are minimal, and if the computer option is utilised, the output data will not require more than a single A4 sheet of paper.

1. Find the overall width (and span) of the deck.
2. Determine the edge distance.
3. Calculate the rigidities D_x , D_y , D_{xy} , D_{yx} , D_1 and D_2 .
4. Calculate the characterising parameters α and θ (note that, for solid slabs, if it is assumed that $D_1 = D_2 = 0$ then $\alpha = 1.0$ and $\theta = b/L$).
5. Calculate the value of M_{sw} (the bending moment due to a single line of wheels of a standard vehicle).
6. Select the relevant distribution table (or use computer option) to determine D .
7. Calculate the maximum moment from $M_{max} = SM_{sw}D$. (Note: $S = 1$ m for a slab.)

The method can be utilised by using either predocumented tables of distribution coefficients, which can be incorporated into standard codes of practice, or a computer program to analyse a particular bridge. In either case, the data preparation is kept to a minimum and only useful output data are generated.

The method can be used for design or assessment purposes for determining the value of critical moments under any load specification, such as the UK, AASHTO and Eurocode loadings.

US practice

In the USA, the distribution method has been in use for many years and is a widely adopted tool for the global analysis of simply supported right bridges. The latest distribution factors (DFs) – referred to as mg – are presented in AASHTO (1994) and provide equations for calculating the DFs for slab-on-girder bridges for the maximum bending moment and shear force on an interior girder and an external girder. For example, for an interior girder:

$$M_{\text{int}} = mg_{\text{int}} \times \text{maximum moment from the AASHTO design truck} \quad (3.19)$$

(Note: The multiple presence factor (see the earlier section on truck loading) is implicitly included in mg .)

Typical examples are given in Barker and Puckett (1997).

Pedestrian loading

Crowd loading

The early pedestrian bridges were constructed more by trial and error than to any design loading applied. With the average weight of a person of around 0.6 kN, many early pedestrian bridges could only take a loading of a few kilonewtons per square metre of deck. There are many recorded examples of pedestrian bridge collapses over the centuries, caused by crowd loading exceeding the capacity of the structure. The Saale Bridge in Germany collapsed in 1825, when a procession to celebrate the opening of the bridge was crossing, as did the original Rialto Bridge in Venice in 1444.

Modern practice in most countries is to specify a nominal crowd loading of about 5 kN/m² (EC1 model LM4) to be placed on the footways of highway bridges or across pedestrian and cycle bridges. In some instances, reduction of loading is allowed for loaded lengths greater than 10 m.

Consideration also needs to be given to the dynamic loading that can occur on pedestrian bridges. This is rarely a problem for highway or rail bridges with pedestrian access, but with the contemporary approach of producing elegant lightweight pedestrian structures, their dynamic behaviour needs to be assessed and catered for in the design.

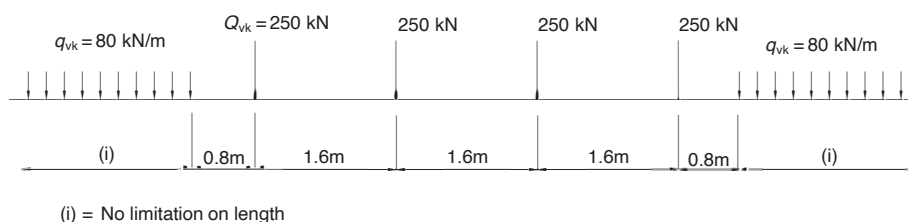


Figure 3.25 Eurocode LM 71 vertical loads

Rail traffic loading

Brief history of rail loading specifications in the UK

The Stockton and Darlington Railway opened the first public rail line in the UK in 1825 to connect the nearby coal mines to Darlington, using steam locomotives built by George Stephenson, who is considered the 'Father of Railways'. The track gauge, the distance between rails, was 4 ft 8 in (1.42 m), and the track passed over several bridges and high embankments. The early rail locomotives typically had four wheels on two axles and weighed between 4 and 8 tons. The UK railway network developed quickly over subsequent decades as passenger and freight lines multiplied, and by the 1870s the UK had over 13 500 miles of line. In this time, the locomotives had become longer, with more wheels and axles, and heavier, with weights up to 50 tons. In these early days, the main train weights were associated with the locomotives, pulling lighter passenger or freight wagons, often weighing 10 t or less.

Trains continued to get heavier until the 1930s, when steam locomotives weighing more than 100 t could be found on the numerous train lines around the UK. After this, the weights and axle load of the steam locomotives started to drop as the engineering improved and locomotives became more efficient.

Diesel and electric locomotives started to be seen in the 1940s, with weights up to 138 t by the 1960s, although the axle loads dropped after that, generally being 20 t or less. The 1960s also saw 100 t coal wagons, with axle loads up to 25 t, coming onto the scene, creating long trains of heavy loads to be catered for by the bridges.

In the 1970s, a typical passenger locomotive weighed around 70 t with axle loads of 17 t. Modern locomotives, such as the Hitachi Class 800 (2017), has axle loads of 15 t.

In the early days of the railways, bridge loading was generally considered as a UDL, derived by taking the locomotive weight divided by its length, with axle loads and dynamic or impact loading ignored. The mid 1800s saw rules develop for loading, but it was not until the 1920s that the British Standard train loading was introduced. This was followed in the 1970s with the introduction of the International Union of Railways (UIC or RU) loading, along with dynamic and rail–structure interaction effects, which have evolved into the modern design codes seen today.

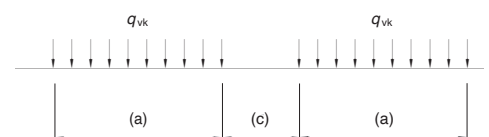


Figure 3.26 Eurocode load models SW/0 and SW/2

Eurocode rail loading, as applied in the UK

Vertical loads

The normal traffic loading on mainline railways is given by load models 71 and SW/0. LM71 is shown in Figure 3.25, with a series of four 250 kN axle loads and a UDL of 80 kN/m on either side. The SW/0 loading is indicated in Figure 3.26, with a load of 133 kN/m applied over a distance (a) of 15 m with a gap (c) of 5.3 m. These loads are multiplied by a factor varying between 0.75 and 1.46 for lines carrying lighter or heavier rail traffic.

A further load model SW/2 represents heavy loads and is also arranged as shown in Figure 3.26 with the load applied of 150 kN/m over a distance (a) of 25 m with a gap (c) of 7.0 m.

Light rail and mass rapid transit systems are normally located in urban settings with smaller locomotives and coaches. These usually run on bespoke bridges, purpose-built for the system and designed for the actual train configuration and axle load that will be used. Axle loads for these trains usually vary between 12 and 16 kN, depending on the type of system adopted.

High-speed trains, with speeds in excess of 200 km/h are becoming more commonplace in many countries around the world to link major cities and replace air travel. These are represented by load model HSLM, consisting of a range of different high-speed trains that are being used, with up to 18 coaches and 210 kN axle loads.

These loads are also subjected to dynamic effects with amplification factors up to 2.0 applicable, depending on the track maintenance regime, type of bridge and element of support being considered. Dynamic analysis of the bridges can also be conducted to determine the dynamic impact factor to be used; this is often done for light rail and high-speed rail systems.

Dispersal of concentrated loads

For ballasted track, the point loads given in Figure 3.25 are distributed with half applied to one sleeper and the remaining half applied equally to the adjacent sleepers on either side. With track-slab rail systems, this distribution is usually adjusted to be two-thirds and one-third.

The loads distribute out through the sleeper and are often taken to act over a distance of 800 mm at the underside of the sleeper and symmetrically about the rail. Where ballast is used, the dispersal of the load through the ballast is usually taken at 5° to the vertical.

Lurching loads

As trains travel forward, they can be affected by lurching from side to side, with a resulting redistribution of the vertical load between the rails. This effect is catered for by taking 0.56 of the axle load on one rail acting concurrently with 0.44 of the axle load on the other rail. Lurching is usually ignored on elements supporting loads from more than one track.

Nosing load

Trains are guided along the rails by a flanged wheel, which is set slightly narrower than the spacing of the rail to provide some tolerance. The wheels are kept in line and restrained by the rails when they try to wander. This restraint effect is referred to as hunting or nosing, with a small lateral horizontal load, 100 kN, applied horizontally at the rail level.

Centrifugal load

Where the track is curved, allowance is to be made for centrifugal forces from the rail traffic loads. The nominal centrifugal load, Q_{rk} (in kN), per track acting radially at a height of 1.8 m above rail level is calculated as

$$Q_{rk} = (V^2 \times f \times Q_{vk}) / 127r \quad \text{or} \quad q_{rk} = (V^2 \times f \times q_{vk}) / 127r \quad (3.20)$$

where: Q_{vk} or q_{vk} = vertical load from the train

r = radius of curvature

V = design speed of train on the curve

f = reduction factor

$$= 1 - [(V - 120)(814/V + 1.75)(1 - \sqrt{(2.88/L_f)})] / 1000$$

L_f = influence length of loaded part of curved track.

Longitudinal loads

As trains accelerate or slow, they create longitudinal tracking and braking loads, applied to the rails. These loads are considered as acting at rail level in a direction parallel to the tracks. No addition for dynamic effects is made for these loads. The longitudinal force arising from the traction force (30% of load on driving wheels) and from the braking (25% of load on braked wheels) is given in Table 3.9 for LM71, SW/0 and HSLM, with the values given adjusted for light rail or heavy trains accordingly.

Load arising from	Loaded length: m	Longitudinal load: kN
Traction (30% of load on driving wheels)	≤3	150
	3–5	225
	5–7	300
	7–25	24(L – 7) + 300
	750	500
Braking (25% of load on braked wheels)	≤3	125
	3–5	187
	5–7	250
	>7	24(L – 7) + 250

Table 3.9 Eurocode longitudinal loads for LM71, SW/0 and HSLM

For structures with more than one track, only a maximum of two tracks are considered for the simultaneous application of the traction and braking loads. Where the tracks carry traffic in opposite directions, the load due to braking is applied to one track and the load due to traction is applied to the other track.

BS 5400 rail loading

The loadings are very similar to the Eurocode loading described previously, with RU and SW/0 loading equivalent to LM71 and SW/0 in the Eurocode. In addition, an RL loading is given, as shown in Figure 3.27 to account for light rail or MRT type railways.

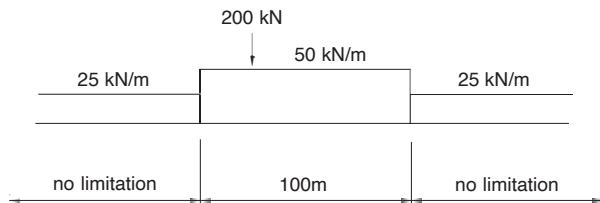


Figure 3.27 BS 5400 Type RL loading

Wind

Wind causes bridges – particularly long, relatively light bridges – to oscillate. It can also produce large wind forces in the transverse, longitudinal and vertical directions of all bridges. The estimation of wind loads on bridges is a complex problem because of the many variables involved, such as the size and shape of the bridge, the type of bridge construction, the angle of attack of the wind, the local topography of the land and the velocity–time relationship of the wind.

Although wind exerts a dynamic force, it may be considered as a static load if the time to reach peak pressure is equal to or greater than the natural frequency of the structure. This is the usual condition for a majority of bridges. Wind is not usually critical on most small- to medium-span bridges, but some long-span bridges or beam-type bridges on high piers are sensitive to wind forces. Wind can become more critical in high wind areas, such as where typhoons and hurricanes occur, or where wind becomes funnelled, such as in a steep-sided valley or between tall buildings, which need to be considered as special cases.

The greatest effects occur when the wind is blowing at right angles to the line of the bridge deck, and the nominal wind load can be defined as

$$P = qAC_D \quad (3.21)$$

where q is the dynamic pressure head, A is the solid projected area and C_D is the drag coefficient. Guidance is given in the various bridge codes on the calculation of these three quantities for different bridge types.

The velocity of the wind varies parabolically with height, as shown in Figure 3.28. Then

$$q = \rho v_c^2 / 2 = 0.613 v_c^2 \quad (3.22)$$

where ρ is the density of air normally taken as 1.226 N/m^3 and v_c is the maximum gust speed based on the mean hourly wind speed v and modified by a gust factor K_g (which increases with height above ground level but decreases with increased loaded length) and an hourly speed factor K_s (which increases with height above ground level) for particular loaded lengths;

thus

$$q = 0.613 v_c^2 / 10^3 [\text{kN/m}^2] \quad (3.23)$$

The value of v is normally obtained from local data in the form of isotachs (in m/s), and values of the gust factor (K_g) and the hourly speed factor (K_s) are quoted in the codes of practice; thus

$$v_c = v K_g K_s \quad (3.24)$$

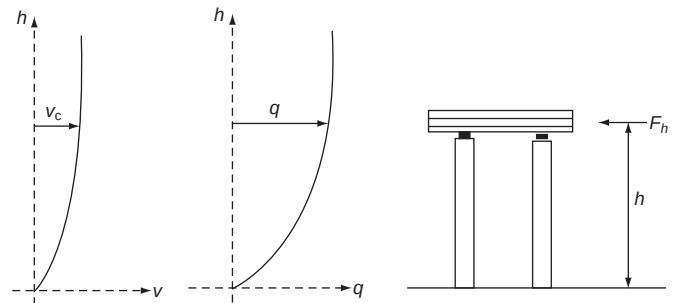


Figure 3.28 Variation of wind speed and pressure with height

The value of the force acting at deck level (and at various heights up the piers) can thus be determined for design purposes.

In the UK, both isotachs and drag coefficients for various cross-sectional shapes are given in BD 37/01. In the USA, wind pressures are found in AASHTO (2020).

Long-span and flexible bridges, such as cable-stayed and suspension bridges, can be subject to vibrations induced by varying wind loads on the bridge deck. The total wind load on the deck is given by Dyrbe and Hansen (1996) as

$$F_{\text{tot}} = F_q + F_t + F_m \quad (3.25)$$

where F_q is the time-averaged mean wind load, F_t is the fluctuating wind load due to air turbulence (buffeting) and F_m is the motion-induced wind load.

Aerodynamic effects

These are usually more significant on long, light bridges, such as cable-stayed or suspension bridges, and are generally classified as

- vortex excitation
- galloping and stall hysteresis
- classical flutter.

Random changes of speed and direction of incidence of the wind can also cause dynamic excitation.

Vortex excitation

Vortex shedding – from upper and lower surfaces, alternately – causes periodic fluctuations of the aerodynamic forces on the

structure. These are proportional to the wind speed; thus, a resonant response will occur at a specific speed. In extreme cases (witness the Tacoma Narrows Bridge in 1940), this can result in vertical and torsional deformations, leading to the failure of the bridge.

Structural damping can decrease the maximum amplitude and extent of wind speed range, but will not affect the critical speed.

Truss girder stiffened suspension bridges are generally free of vortex-excited oscillations, but plate girder and box girder stiffened bridges are prone to such oscillations. Appropriate modification of the size and shape of box girders can considerably reduce these effects; this is why wind tunnel tests are essential.

Wherever there is a surface of velocity discontinuity in flow, the presence of viscosity causes the particles of the fluid (wind) in the zone to spin. A vortex sheet is then produced, which is inherently unstable and cannot remain in place, so it rolls up to form vortices that increase in size until they are eventually 'washed' off and flow away. To replace each vortex, another vortex is generated; under steady-state conditions, it is reasonable to expect a periodic generation of vortices (see Figure 3.29).

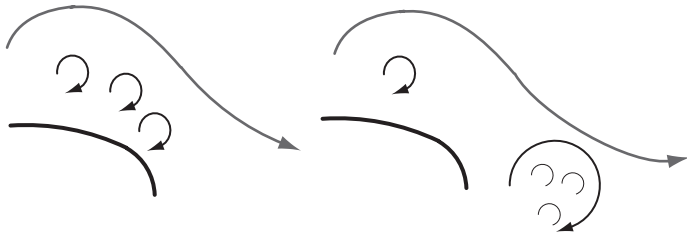


Figure 3.29 Vortex shedding

The most likely places for them to appear are at discontinuities, such as sharp edges, and they form above and below the body concerned. In the case of a bridge, the deck is subjected to this phenomenon.

The frequency of the shedding of the vortices can be related to the wind speed by means of a Strouhal number S , which is dimensionless:

$$S = n_v D / v \quad (3.26)$$

where n_v is the vortex shedding frequency, v is the wind speed (in m/s) and D is a reference dimension of the cross-section.

The worst situation occurs when the frequency of oscillation (f) is equal to the natural frequency (f_n) when the wind is at its critical speed V_{crit} . Thus,

$$V_{crit} = f_n D / S \quad (3.27)$$

At this point, the response amplitude is a maximum, as shown in Figure 3.30. If there is more than one mode of vibration (which is generally the case), there will be several critical wind speeds, each with a different corresponding amplitude.

The design must ensure that V_{crit} is kept outside of the normally expected wind speed range at the bridge site. Design

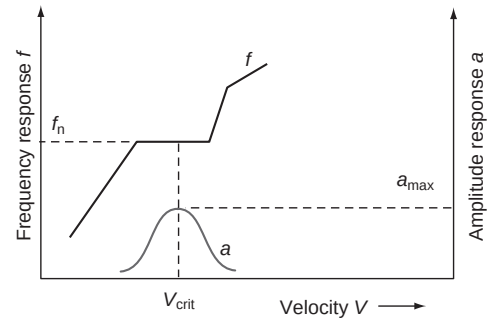


Figure 3.30 Relationship between response frequency and response amplitude

features that will minimise the depth of the wake (the turbulent air leeward of the deck) are found to reduce the power of vortex excitation, see Figure 3.31.

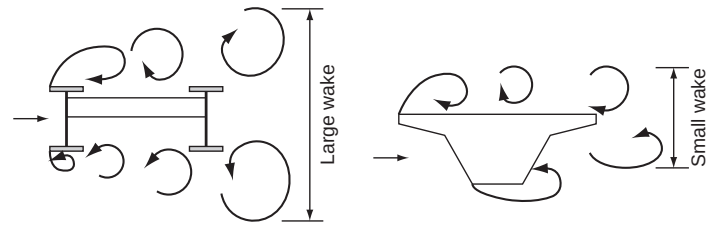


Figure 3.31 Effect of shape on depth of the wake

Some practical details that have been found to work are

- shallow sections (compared with width)
- perforation of beams to vent air into wake
- soffit plate to close off spaces between main girders
- tapered fairings or inclined web panels
- avoidance of high-solidity fixings and details, such as fascia beams near the deck edges
- the use of deflector flaps or vanes on deck edges to obviate vortices or promote reattachment of surfaces.

Gallop

This is large-amplitude, low-frequency oscillation of a long linear structure in transverse wind at the natural frequency f_n of the structure. It is a phenomenon that does not require high wind speeds when the cross-section has certain aerodynamic characteristics. Such was the case with the Tacoma Narrows Bridge, which began to gallop in wind speeds of only 40 miles per hour; this is why it got its nickname of 'Galloping Gertie'.

Once the critical wind speed has been reached, an oscillating motion begins at the natural frequency f_n , but the amplitude of vibration increases with increasing wind speed, apparently owing to changes in the direction of the wind due to the motion of the structure – essentially a constant changing of the angle of incidence (see Figure 3.32).

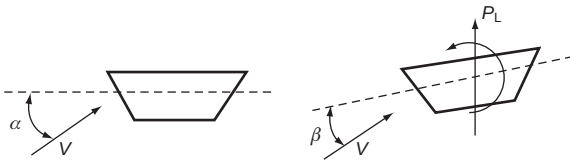


Figure 3.32 Apparent change of wind direction due to movement of deck

The lifting force is

$$P_L = [0.5\rho V^2][d^2][C_L] \quad (3.28)$$

where $0.5\rho V^2$ is the wind pressure, d^2 is a unit of deck area and C_L is the coefficient of alternating wind force, which depends on α .

Flutter

This is caused by a stalling air flow and causes an aeroelastic condition in which two degrees of freedom – rotation and vertical translation – couple together in a flow-driven oscillation. This was first observed on aerofoil sections used in the aeroplane industry and is usually confined to suspension bridges.

Buffeting

Buffeting is the effect of unsteady loading by velocity fluctuations in the oncoming (windward) flow. It can also occur in the wake and cause problems with an adjacent bridge or where stay cables are placed close together.

Wind loads using British standards

For most bridge designs, the real dynamic wind forces are converted to equivalent static forces acting transversely, longitudinally and vertically on short- to medium-span bridges, which comprise the majority of the UK's bridge stock. Cable-stayed and suspension bridges subject to dynamic forces and movements are not considered.

Detailed analysis requires knowledge of the wind speeds at the bridge location. This can be from an isotach map, if available, for the country or region where the bridge is to be constructed. For the UK, this is Figure 2 of BD 37/01 (DoT, 2001), reproduced here as Figure 3.33. This enables the determination of the maximum wind gust speed and mean hourly wind speed v from the equations:

$$V_d = S_g V_s \quad (3.29)$$

$$V_s = V_p S_p S_d \quad (3.30)$$

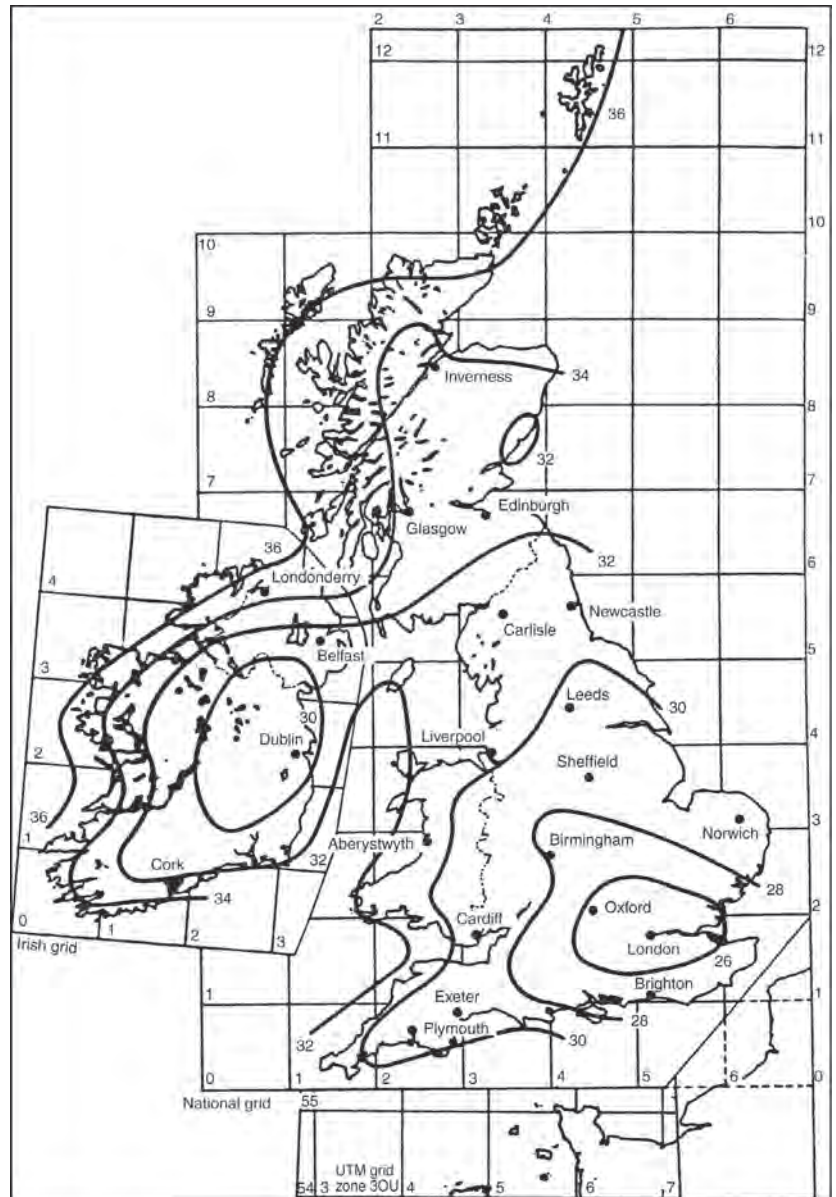


Figure 3.33 Isotach map of UK

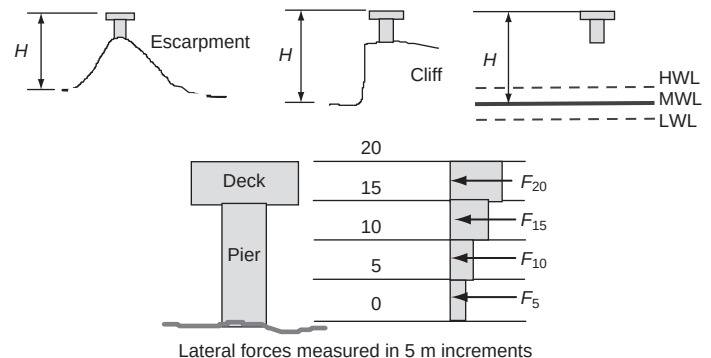


Figure 3.34 Definition of H

Values of S_g , S_p , S_a and S_d are given in BD 37/01, and are related to the height (H) of the bridge above sea level and the fetch, as given in Tables 3 and 4 of BD 37/01, and defined here in Figure 3.34.

Transverse wind load

The transverse wind forces acting on the bridge are then calculated as

$$P_t = qA_1C_D \quad (3.31)$$

where the dynamic pressure head is given as

$$q = 0.613V_c^2/10^3 \text{ [kN/m}^2\text{]} \quad (3.32)$$

$$A_1 = \text{projected unshielded area [m}^2\text{]} \quad (3.33)$$

$$C_D = \text{drag coefficient} \quad (3.34)$$

Solid bridges

For bridges presenting a solid elevation to the wind, A_1 is derived by determining the solid projected depth (d) from Figure 4 of BD 37/01 (reproduced as Figure 3.35):

$$A_1 = d_l \text{ per metre along the bridge} \quad (3.35)$$

Truss girder bridges

For truss girder bridges, A_1 is the solid (net) area presented to the wind by the girder members – that is, the sum of the projected areas of the truss:

$$A_1 = \Sigma \text{ member areas} \quad (3.36)$$

For windward girders, the value of C_D is taken from Table 6 of BD 37/01 according to the solidity of the truss defined by a solidity ratio:

$$\sigma = \text{net area of truss/overall area of truss} \quad (3.37)$$

For leeward girders, some shielding is inevitable from the windward girder, and this is taken into account by a shielding

factor η derived from Table 7 of BD 37/01 based on the spacing ratio SR:

$$\text{SR} = \text{spacing of trusses/depth of windward truss} \quad (3.38)$$

and the drag coefficient is given by ηC_D .

Note that, for both solid and truss bridges, two cases have to be considered:

- wind acting on the superstructure alone
- wind acting on the superstructure plus live loading from the traffic (maximum wind speed allowed is 35 m/s, above which it is assumed that traffic would be stopped from using the bridge).

The worst case is taken for design purposes.

Piers

The drag coefficients for piers are taken from Table 9 of BD 37/01, depending on the cross-sectional shape. Normally, no shielding is allowed for.

Longitudinal wind loads

Superstructure

As with transverse wind loads, the worst of the wind load on the superstructure alone (P_{LS}) and the wind on the superstructure plus live loading (P_{LL}) is taken for design purposes.

All structures with a solid elevation:

$$P_{LS} = 0.25qA_1C_D \quad (3.39)$$

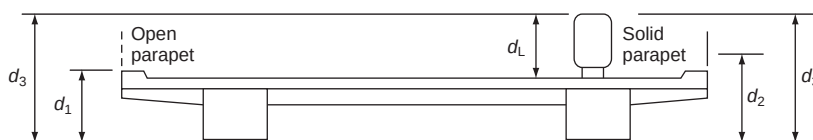
All truss girder structures:

$$P_{LS} = 0.5qA_1C_D \quad (3.40)$$

Live load on all structures:

$$P_{LL} = 0.5qA_1C_D \quad (3.41)$$

where $C_D = 1.45$.



Parapet	Unloaded bridge	Live loaded bridge
Open	$d = d_1$	$d = d_3$
Solid	$d = d_2$	$d = d_2 \text{ or } d_3$ whichever is greater

$d_L = 2.5 \text{ m above the highway carriageway, or}$
 $3.7 \text{ m above the rail level, or}$
 $1.25 \text{ m above footway or cycle track}$

Parapets and safety fences

$$\text{With vertical infill, } P_L = 0.8P_t. \quad (3.42)$$

$$\text{With two or three rails, } P_L = 0.4P_t. \quad (3.43)$$

$$\text{With mesh, } P_L = 0.6P_t. \quad (3.44)$$

Piers

The longitudinal wind load is given by

$$P_L = qA_2C_D \quad (3.45)$$

where A_2 is the transverse solid area and C_D is taken from Table 9 of BD 37/01, with b and d interchanged.

Figure 3.35 Depth d to be used for calculating A_1

Uplift

The vertical uplift wind force on the deck (P_v) is given by

$$P_v = qA_3C_L \quad (3.46)$$

where A_3 is the plan area of the deck and C_L (the lift coefficient) is taken from Figure 6 of BD 37/01 if the angle of elevation is less than 1° . For angles between 1° and 5° , C_L is taken as ± 0.75 . For angles $>5^\circ$, tests must be carried out.

Load combinations

There are four wind load cases to consider in load combination 2:

- P_t
- $P_t \pm P_v$
- P_L
- $0.5P_t + P_L \pm 0.5P_v$

Overturning effects

It is necessary to check the stability of the structure when subject to wind in combination with heavy vehicles on the outer extremities of the deck. This is illustrated in Figure 3.36.

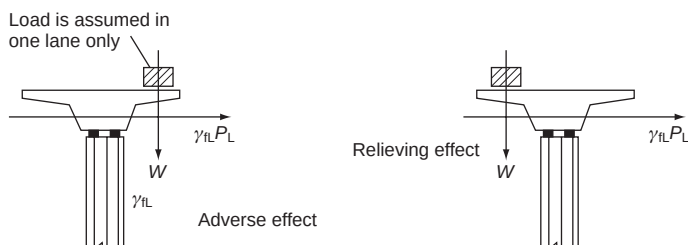


Figure 3.36 Stability requirements

Concluding remarks

Wind loads affect bridges in a twofold way: globally or locally.

Global wind forces induce overall bending, shear and twisting forces, and these loads are transferred to the tops of piers and abutments through the bearings and expansion joints; they are also transferred to the foundations. Stability is also a factor to be considered, especially in the case of continuous bridges with long spans and a degree of curvature.

Local wind forces are resisted by parapets and safety fences, and may have fatigue consequences in steel bridges and in the cables and hangers of tension structures.

For small- to medium-span bridges, wind loads are not normally critical, but in the case of long-span suspended structures, wind forces can dominate and can cause collapse.

Temperature

The temperature of both the bridge structure and its environment changes on a daily and seasonal basis and influences both the overall movement of the bridge deck and the stresses within it. The former has implications for the design of the bridge bearings and expansion joints, and the latter on the amount and disposition of the structural materials.

The daily effects give rise to temperature variations within the depth of the superstructure; these vary depending on whether the effect is heating or cooling, and guidance is normally given in the form of idealised linear temperature gradients to be expected when the bridge is heating or cooling for various forms of construction (concrete slab, composite deck, etc.) and blacktop surface thickness. The temperature gradients result in self-equilibrating internal stresses. Two types of stress are induced, namely primary and secondary, the former due to the temperature differences throughout the superstructure (whether simply supported or continuous) and the latter due to continuity. Both must be assessed and catered for in the design.

The temperature of a bridge deck varies throughout its mass. The variation is caused by

- the position of the sun
- the intensity of the sun's rays
- the thermal conductivity of the concrete and surfacing
- wind
- the cross-sectional make-up of the structure.

The effects are complex and have been investigated in the UK by the Transport Research Laboratory (TRL). Changes occur on a daily (short-term) and annual (long-term) basis. Daily, there is heat gain by day and heat loss by night. Annually, there is a variation of the ambient (surrounding) temperature.

On a daily basis, temperatures near the top of the bridge are controlled by incident solar radiation, and temperatures near the bottom are controlled by shade temperature. The general distribution is indicated in Figure 3.37. A positive temperature flow represents a rapid rise in temperature of the deck slab due to direct sunlight (solar radiation). A negative temperature flow represents a falling ambient temperature due to heat loss (re-radiation) from the structure.

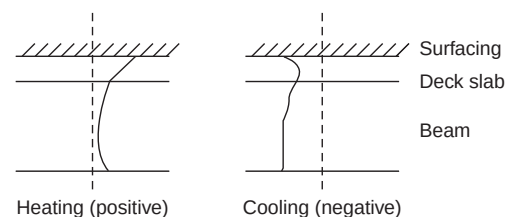


Figure 3.37 Typical temperature distributions

Research has indicated that, for the purposes of analysis, the distributions (or thermal gradients) can be idealised for different 'groups' of structure, as defined in Figure 9 of BD 37/01 Clause 5.4 (DoT, 2001). The critical parameters are the thickness of the surfacing, the thickness of the deck slab and the nature of the beam. Concrete construction falls within Group 4. Temperature differences cause curvature of the deck and result in internal primary and secondary stresses within the structure.

Primary stresses

Primary stresses occur in both simply supported and continuous bridges and are manifested as a variation of stress with depth. They develop as a result of the redistribution of restrained temperature stresses, which is a self-equilibrating process. They are determined by balancing the restrained stresses with an equivalent system of a couple and a direct force acting at the neutral axis position. The section is divided into slices, and the restraint force in each slice is determined. The sum of the moments of each force about the neutral axis and the sum of the forces give the couple and the direct force, respectively. These are shown in Figure 3.38.

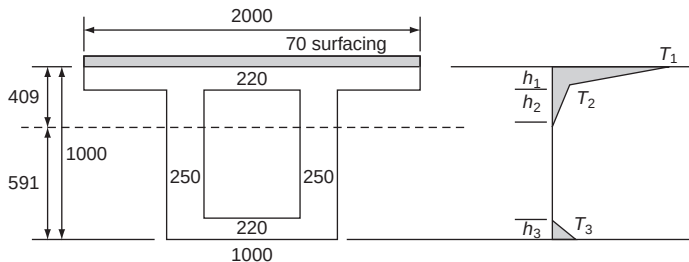


Figure 3.38 Primary stresses due to temperature gradient through bridge

Secondary stresses

Secondary stresses occur in continuous bridges or bridges with restraints and result from a change in the global reactions and bending moments as the bridge's ambient temperature changes and as the temperature gradient acts. They are determined by applying the built-up forces and couples at each free end of the bridge and determining the resulting reactions and moments. These are then added to the reactions and moments from the other loading.

Primary stresses are not necessarily larger than secondary stresses. Both can be significant and depend on a whole range of variables. Once calculated, they are included in Combination 3, defined in BD 37/88.

Annual variations

Annual (or seasonal) changes result in a change in length of the bridge and therefore affect the design of both bearings and

expansion joints. Movement is related to the minimum and maximum expected ambient temperatures. This information is normally available in the form of isotherms for a particular geographic region. The total expected movement (Δ) takes place from a fixed point, called the thermal centre or stagnant point, and is given by

$$\Delta = \text{thermal strain} \times \text{span} = \beta_L T L \quad (3.47)$$

where β_L is the coefficient of thermal expansion and T is the temperature change and is based on a total possible range of movement given by the difference of the maximum and minimum shade temperatures and specified in the code for a given bridge location as isotherms in Figures 7 and 8 of BD 37/01. These are further modified to take account of the bridge construction in Tables 10 and 11 of BD 37/01 to give the effective bridge temperature.

The bearings and expansion joints are set in position to account for actual movements, which will depend on the time of year and bridge ambient temperature at the time they are installed. This is shown in Figure 3.39 in relation to the 'setting' of an expansion joint.

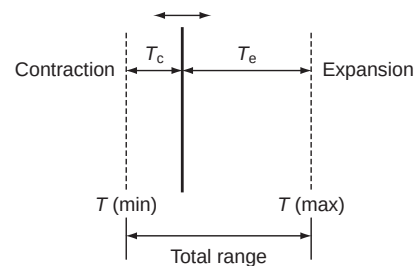


Figure 3.39 Setting of an expansion joint

Bearing friction

For mechanical bearings with sliding surfaces, a frictional force can be generated as the bearing movement occurs. This can be generated by the change in ambient temperature of the deck or when horizontal loads are applied to the deck. The magnitude of the frictional force varies depending on the vertical load applied and the sliding surface configuration. Typically, a maximum force of 5% of the vertical load is taken on horizontal surfaces or 2% of the transient lateral loads on vertical surfaces, such as in guided bearings. Transient loading, such as traffic or wind can cause a reduction in the friction force, owing to vibrations that occur.

Earthquakes

In the past, the effect of earthquakes on buildings has received more attention than that on bridges, probably because the social and economic consequences of earthquake damage in buildings has proved to be greater than that resulting from damage to bridges. However, there have been some notable collapses of bridges when major earthquakes have occurred.

In a study of seismic shock, Albon (1998) observes that:

Bridges should be designed to absorb seismic forces without collapse to ensure that main arterial routes remain open after major seismic events. This helps the movement of aid and rescue services in the first instance and underpins the ability of the local community to recover in the long term.

Observations over many years indicate that bridge failures due to earthquake forces on bridges are not caused by the collapse of any single element of the superstructure but rather by three effects:

- the superstructure being shaken off the bearings and falling to the ground
- structural failure due to the loss of strength of the soil under the superstructure as a result of the vibrations induced in the ground
- plastic hinges forming in the piers.

The effect of an earthquake depends on the dynamic characteristics and distribution of the self-weight of the bridge. A simplified procedure is to consider that the earthquake produces lateral forces acting in any direction at the centre of gravity of the structure and having a magnitude equal to a percentage of the weight of the structure or any part of the structure under consideration. These loads are then treated as static for the analysis of the structure.

The design lateral force applied at deck level is given by

$$F = C_D \gamma_i W_i \quad (3.48)$$

where C_D is the seismic design coefficient, which depends on the soil conditions, the risk against collapse, the ductility of the structure and an amplification factor; γ_i is a distribution factor depending on the height of the deck from foundation level; and W_i is the permanent load plus a given percentage of the live load.

Modern codes, such as the current Eurocode EC8 (BSI, 2013), allow three different methods of analysis, namely

- fundamental mode method (static analysis)
- response spectrum method
- time history representation.

The first two are linearly elastic analyses and the last is non-linear. More detailed explanation of the different methods is given in Chapter 5.

The high-profile earthquakes in Northridge, Los Angeles, in January 1994 and Kobe, Japan, in January 1995 proved invaluable to the understanding of the behaviour of bridge structures under earthquake loading and have helped refine the design procedures in the modern codes.

Snow and ice

In certain parts of the world, snow and ice are in evidence for considerable periods and, in the case of cable-stayed and suspension bridges, can contribute significantly to the dead weight by forming around cables and parapets and on supporting towers. Complete icing of the parapets also means that lateral wind forces are increased, owing to the larger solid area exposed to the wind. Expansion joints and bearings can also become locked, resulting in large restraining forces to the deck and substructures.

Bridges with piers standing in water can also be affected by ice flows, which can exert considerable forces on the substructure and foundations. These can be mitigated by shaping the piers and pile caps, as adopted for the Confederation Bridge crossing the Northumberland Strait in Canada.

Water

Rivers in flood represent a serious threat to bridges both from the point of view of lateral forces on the abutments, piers and superstructures and that of the possible undermining of the foundations as a result of the scouring effect of the water.

The lateral hydrodynamic forces are calculated in a similar manner to those due to wind. Thus, from

$$q = \rho v_c^2 / 2 \quad (3.49)$$

(where v_c is the velocity of flow in m/s), if the density of water is taken as 1000 N/m^3 , the water pressure is

$$q = 500 v_c^2 / 10^3 \text{ [kN/m}^2\text{]} \quad (3.50)$$

and

$$P = q A C_D \text{ [kN]} \text{ (as for wind)} \quad (3.51)$$

Values of C_D for various shaped piers in the USA are given in AASHTO LRFD (AASHTO, 2020) and for piers in the UK are found in BA 59 (Highways Agency, 1994).

The degree of scour depends on many factors, such as the geometry of the pier, the speed of flow and the type of soil (Hamill, 1998; Kirby, 2015)

The total depth of bridge scour is due to a combination of general scour due to the constriction of the waterway area leading to an increase in the flow velocity, and local scour adjacent to a pier or abutment from turbulence in the water. Many models are available for dealing with these phenomena (Federal Highways Administration, 1991a; Hamill, 1998; Melville and Sutherland, 1988), all with particular points of merit. Scour is one of the major causes of bridge failure (Smith, 1976) and proper design and protection is essential to guard against such catastrophic events.

There is also the danger from fast-moving debris hitting the piers or the deck, and the possibility of accumulated debris blocking the bridge opening; both need to be considered in design.

Normal flow

Bridge piers are designed to resist lateral forces from water in normal flow conditions. The forces induced are calculated using the same formulae as for moving air, but with the density of air replaced by that for water:

$$\rho_w = 1000 \text{ kg/m}^3 = 10 \text{ kN/m}^3 \quad (3.52)$$

so the dynamic pressure head is

$$q = 500 v_c^2 / 10^3 \text{ [kN/m}^2\text{]} \quad (3.53)$$

$$P_{tw} = q A_1 C_D \quad (3.54)$$

Values of C_D can be determined from Table 9 of BD 37/01.

In the UK some guidance is given in Departmental Memorandum BA 59/94, *The Design of Highway Bridges for Hydraulic Action* (Highways Agency, 1994), which also considers forces due to ice, debris and ship collision.

Flood conditions

Flood waters exert forces many times those exerted under normal conditions. Very often the waters top the bridge (negative freeboard) and both the deck and piers are subject to the full force of water and debris. Areas of turbulence cause high local forces and scour of the riverbed around the piers. Estimation of the forces involved is complex and unreliable (foreexample, estimating the speed and height of the flood waters), and most countries have their own procedures, which take into account local topography and experience from previous floods.

Scour

Scour is not classed as a load, but it is caused by erosion of the riverbed around the piers and foundations and can cause undermining of the foundations and eventual collapse.

Three types of scour – general, local and combined – are considered in BA 59/94 (Highways Agency, 1994), which leans heavily on US reports FHWA-IP-90-017 (Federal Highways Administration, 1991a) and FHWA-IP-90-014 (Federal Highways Administration, 1991b). An example is shown to illustrate scour effects and is reproduced in Figure 3.40.

Impact and collision loads

These accidental loads can arise from a number of different situations, with the main ones considered in the design of bridges being vehicular impact on parapets or piers, or ship impact where piers are in navigational waterways.

Parapet collision loads

As can be seen by the damage or marks on many highway bridges, it is not uncommon for an out-of-control vehicle to



Figure 3.40 Example of scour (courtesy of Department of Transport)

collide with the parapets or barriers down the sides of the carriageway.

The traditional approach has been to design the parapets for the specified load capacities, such as those given in BD52/93 (DoT, 1993) or AASHTO (2020) Section 13. The decks supporting the parapets are then designed for a greater load, either simply factored up or based on the actual strength of the parapets, to ensure that the parapets fail without significantly damaging the bridge deck itself.

Rail bridges have a different approach, with most train systems restraining errant locomotives or coaches through the track-slab or ballast arrangement, or with derailment kerbs running alongside the rails. In this case, the parapets are not designed to restrain the trains and are mainly designed to ensure the safety of maintenance workers or disembarked passengers if trains break down.

Vehicular impact on piers

An out-of-control vehicle on a road passing under a bridge may collide with the bridge supports, and result in severe impact loads. These usually occur at bumper or fender level, but in some cases on high vehicles a secondary impact occurs at higher levels. The impact forces depend on the mass of the vehicle and speed of travel; typical equivalent static design forces to be considered for different classifications of highways are given in Eurocode 1 (BSI, 2006b) and the associated UK National Annex, as shown in Table 3.10.

With highway bridges, all too often ‘bridge bashing’ occurs, where vehicles passing underneath make contact with the bridge superstructure. This can simply be from a high vehicle hitting a low-headroom bridge, although a bridge built to current standards of headroom can also be impacted by a tipper truck with its back raised or a low-loader transporter carrying an excavator with its arm raised. To cater for this type of

	Force F_{dx} in the direction of normal travel: kN	Force F_{dy} perpendicular to the direction of normal travel: kN	Point of application on bridge support
Bridges over motorways, trunk and principal roads			
Main component	1500	750	At the most severe point between 0.75 m and 1.5 m above carriageway level
Residual component	750	375	At the most severe point between 1 m and 3 m above carriageway level
Bridges over other roads where speed limit ≥ 45 miles per hour (72 km/h), e.g. other rural roads			
Main component	1125	565	At the most severe point between 0.75 m and 1.5 m above carriageway level
Residual component	565	265	At the most severe point between 1 m and 3 m above carriageway level
Bridges over other roads where speed limit < 45 miles per hour (72 km/h): e.g. other urban roads			
Main component	750	375	At the most severe point between 0.75 m and 1.5 m above carriageway level
Residual component	375	185	At the most severe point between 1 m and 3 m above carriageway level
Bridges over roads: minimum forces for robustness			
Main component	225	225	At the most severe point between 0.75 m and 1.5 m above carriageway level
Residual component	150	150	At the most severe point between 1 m and 3 m above carriageway level

Table 3.10 Impact loading on substructure

impact, an equivalent static force is applied to the deck soffit, which can be up to 500 kN for bridges over major highways.

Piers adjacent to rail tracks also need to be designed for a derailed train, usually travelling at speed, resulting in much greater impact forces than those associated with highway vehicles. It is usually preferable to place the bridge supports away from the tracks where possible.

Ship impact

A constant danger for any bridge with piers in a navigational waterway, ship impact forces can often govern the design of piers and foundations. The magnitude of any collision force is governed by the mass and speed of the vessel, but also by the vessel type and the angle of impact. The water flow and depth can add to the impact forces or limit the size of vessels, respectively, while a protection island, dolphins and fenders can be used to mitigate the risks and reduce any damage.

The size and type of ship varies greatly, depending on whether it is seagoing, plying an inland waterway, or a pleasure craft along a canal. With seagoing vessels, the layout of the navigational channels and vessel control systems also play a significant part in the risk of impact occurring, while near harbours and ports the presence of pilots on board and tugs alongside can minimise the risks.

Modern seagoing vessels can have a deadweight tonnage (DWT) up to 500 000 t, potentially causing impact forces approaching 1000 MN. These often lead to an acceptable risk type analysis, with a ship impact risk assessment (SIRA) made to quantify the probability of a collision, with the consequences that may occur, to allow the bridge and its supports

to be designed with sufficient robustness to reduce the risks to an acceptable level. This may be set as the annual frequency of a bridge collapse to be less than a return period of 1 in 10 000 years.

On rivers, the critical vessel is often a barge, with the larger vessels having displacement tonnages of as much as 6000 t, creating impact forces up to 15 MN. The risk of collisions on rivers is often governed by the river alignment, with bends or obstructions increasing the likelihood of an impact occurring. Fenders, either independent or fixed to the bridge pier, are commonly used to protect the structure from errant vessels.

Details of ship characteristics and the interaction between the vessels and bridge structures have been described by Larsen (1993), who gives an insight into this complex topic.

Construction loads

Temporary forces occur on the bridge at each stage of construction, owing to the self-weight of plant and equipment and the method of construction. Generally, these forces are more significant in bridges built using serial construction methods, such as post-tensioned concrete box girders where long unsupported cantilever sections induce forces that are substantially different from those in the completed bridge, in both magnitude and distribution. Cable-stayed and suspension bridges are also susceptible to the method of erection where the deck sections are built up piecemeal from the towers or supporting pylons. In all cases, construction loads and the method of erection should be closely examined to ensure that accidents do not happen, and that the serviceability condition of the final structure is not impeded.

For so long ignored by many international design codes, the importance of considering temporary loads on structures was brought in to focus by the many bridges that have collapsed or suffered damage during the construction phase. One of the earliest publications to start quantifying the construction process and temporary loading was the *Guide Specification for Design and Construction of Segmental Concrete Bridges* (AASHTO, 1989), which has been updated several times since. In more recent times, guidance has been included in Eurocode 1, Part 1-6 (BSI, 2005) for most common types of loading during construction stages.

Load combinations

Just as important as the loads themselves, the way in which they are combined and used creates the basis on which any bridge design is carried out. Bridge loads tend to fall into three broad categories: permanent, transient and accidental load combinations.

In the British Standards, five combinations of loading are considered for the purposes of design: three principal and two secondary. These are defined in Clause 4.4 and Table 1 of BD 37/01 (DoT, 2001) and are reproduced as Table 3.11. It is usual in practice to design for Combination 1 and check other combinations if necessary. Different load factors are applied, depending on the combination being considered and whether serviceability or ultimate conditions are being considered. Similarly, AASHTO has a simple series of combinations when considering service, strength and extreme loading conditions. A basic load case, combining permanent loads, live loads, long-term effects, temperature and settlement, is used with strength load factors, while wind is added for the basic service combination, albeit with the factors generally reduced to 1.0. Additional load combinations are created for including wind at strength, and earthquake, impact and collision loads are added for extreme combinations.

The Eurocodes approach is more probabilistic, including numerous load combinations to cover all possible combinations of the different design situations and loading conditions. Each load type also has a range of factors to be applied, depending on whether the load combination being considered is a persistent,

transient, accidental or seismic design situation and whether the loading is a leading or accompanying component within the combination.

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Combination UK (see BD 37/01 Clause 3.2 for definitions)

1	Permanent + primary live
2	Permanent + primary live + wind + (temporary erection loads)
3	Permanent + primary live + temperature restraint + (temporary erection loads)
4	Permanent + secondary live + associated primary live
5	Permanent + bearing restraint

Table 3.11 UK load combinations

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Chapter 4

Structural analysis

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This chapter has been developed from that authored by NE Shanmugam and R Narayanan in the 2nd edition, published in 2008.

A description of the analytical tools used in modelling the response of typical bridge elements to specified actions (loads, displacements, vibrations, etc.) is given. The models are invariably simplified so that essential computations are made efficiently and rapidly; their design implications are discussed. The theoretical analysis of such bridge systems as beams and trusses is summarised and simplified tables, charts and graphs are provided to assist in rapid calculations. The use of influence lines to assess the effects of moving loads on typical bridge elements is described in detail. The analysis of plated structural components and their bracing systems by conventional methods and techniques using computer codes (finite-element method and grillage analysis) is presented and simplifications used in codes of practice are discussed. Soil–structure interaction is covered in sufficient depth to make an adequate assessment of this vital component of the analysis. The inclusion of structural dynamics provides an introduction to this analytical process; fuller details of this topic are covered in Chapter 5. The goal is to provide background information for engineers in the early part of their careers and to assist senior engineers in conducting rapid reviews of analytical calculations. Extensive reference has been made to several books and research papers relevant to the sections covered herein. For detailed methods of analysis, however, engineers are advised to refer to the references included at the end of the chapter.

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Fundamental concepts

General introduction

Structural analysis essentially consists of mathematical modelling of the response of a structure to the applied loading. Such models are based on idealisations of the structural behaviour of the material and are, therefore, imperfect to some extent, depending on the simplifying assumptions in modelling. Consequently, the assessment of structural responses is the best estimate that can be obtained in view of the assumptions implicit in the modelling of the system. Some of these assumptions are necessary in the light of inadequate data: others are introduced to simplify the calculation procedure to economic levels. Examples of idealisations introduced in the modelling process are as follows.

- The physical dimensions of the structural components are idealised. For example, skeletal structures are represented by a series of line elements and the joints are assumed to be of negligible size. The imperfections in the member straightness are ignored or, at best, idealised.
- Material behaviour is simplified. For example, the stress–strain characteristic of steel is assumed to be linearly elastic, and then perfectly plastic. No account is taken of the variation of yield stress along or across the member.

- The implications of actions that are included in the analytical process itself are frequently ignored. For example, the possible effects of change of geometry causing local instability are rarely, if ever, accounted for in the analysis.

However, it must be recognised that the design loads employed in assessing structural response are themselves approximate. The analysis chosen should be adequate for the purpose and capable of providing solutions at an economical cost.

Structural design involves the arrangement and proportioning of structures and their components in such a way that the assembled structure is capable of supporting the designed loads over a designed lifespan within the allowable limit states.

In practice, all structures are three-dimensional. Structures that have regular layout and are rectangular in shape can be idealised into two-dimensional frames. *Joints* in a structure are locations where two or more members are connected. A structural system in which joints are capable of transferring end moments is called a *frame*. Members in this system are assumed to be capable of resisting bending moments, axial forces and shear forces. A structure is said to be two-dimensional or planar if all the members lie in the same plane.

Beams are members that are subjected to bending and shear. *Ties* are members that are subjected to axial tension only, while *struts* are members subjected to axial compression only.

A *hinge* represents a pin connection to a structural assembly; it does not allow translational movements (Figure 4.1(a)). It is assumed to be frictionless and to allow rotation of one member with respect to the others. A *roller* represents a support that permits the attached structural part to rotate freely with respect to the foundation or base and to translate freely in the direction parallel to the base (Figure 4.1(b)). No translational movement in any other direction is allowed. A *fixed support* (Figure 4.1(c)) does not allow rotation or translation in any direction. A *rotational spring* represents a support that provides some rotational restraint but does not provide any translational restraint (Figure 4.1(d)). A *translational spring* can provide partial restraints along the direction of deformation (Figure 4.1(e)).

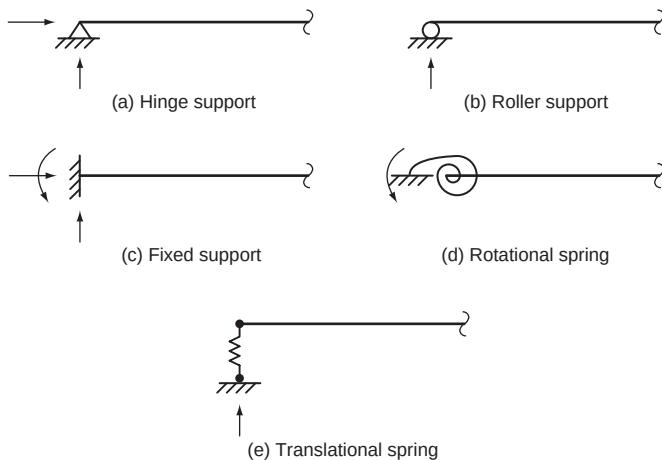


Figure 4.1 Various boundary conditions

Loads and reactions

Loads may be broadly classified as *permanent loads* that are constant in magnitude and remain in one position and *variable loads* that may change in position and magnitude. Permanent loads are also referred to as *dead loads*, and include the self-weight of the structure, and other loads permanently attached to the structure. Variable loads are also referred to as live (or imposed) loads, and include those caused by construction operations, wind, rain, earthquakes, snow, blasts and temperature changes, in addition to those loads that move (vehicles, trains, etc.).

Wind loads act as pressures on windward surfaces and pressures or suctions on leeward surfaces. *Impact loads* are caused by suddenly applied loads or by the vibration of moving or movable loads. *Earthquake loads* are forces caused by the acceleration of the ground surface during an earthquake.

A structure that is initially at rest and remains at rest when acted on by applied loads is said to be in a state of *equilibrium*. The resultant of the external loads on the body and the supporting forces or *reactions* is zero. If a structure, or part thereof, is to be in equilibrium under the action of a system of loads, it must satisfy the six equations of static equilibrium described next.

Equations of static equilibrium

When a body (or structure) remains in a state of static equilibrium, the following two conditions must be satisfied.

- The sum of the components of all forces acting on the body, resolved along any arbitrary direction, is equal to zero. This condition is completely satisfied if the components of all forces resolved along the x , y and z directions individually add up to zero. (This can be represented by $\Sigma P_x = 0$, $\Sigma P_y = 0$, $\Sigma P_z = 0$, where P_x , P_y and P_z represent forces resolved in the x , y and z directions.) These three equations represent the condition of zero translation.
- The sum of the moments of all forces resolved in any arbitrarily chosen plane about any point in that plane is zero. This condition is completely satisfied when all the moments resolved into xy , yz and zx planes all individually add up to zero. ($\Sigma M_{xy} = 0$, $\Sigma M_{yz} = 0$ and $\Sigma M_{zx} = 0$.) These three equations provide for zero rotation about the three axes.

In general, therefore, there are, in total, *six* equations of static equilibrium. If a structure is planar and is subjected to a system of coplanar forces, the conditions of equilibrium can be simplified to the following *three* equations.

- The components of all forces resolved along the x and y directions will individually add up to zero ($\Sigma P_x = 0$ and $\Sigma P_y = 0$).
- The sum of the moments of all the forces about any arbitrarily chosen point in the plane is zero (i.e. $\Sigma M = 0$).

The principle of superposition

This principle is only applicable when the displacements are linear functions of applied loads. For structures subjected to multiple loading, the total effect of several loads can be computed as the sum of the individual effects calculated by applying the loads separately. This principle is a very useful tool in computing the combined effects of many load effects (e.g. moment or deflection). These can be calculated separately for each load and then summed.

Element analysis

Any complex structure can be looked on as being built up of simpler units or components, termed 'members' or 'elements'. Broadly speaking, these can be classified into three categories.

- *Skeletal structures* consisting of members with one dimension (say, length) much larger than the other two (namely, breadth and height). Such a line element is variously termed a bar, beam, column or tie. A variety of structures are obtained by connecting such members together using rigid or hinged joints. Should all the axes of the members be situated in one plane, the structures so produced are termed *plane structures*. Where all members are not in one plane, the structures are termed *space structures*.

- Structures consisting of members with two dimensions (i.e. length and breadth) of the same order but much greater than the thickness, fall into the second category. Such structural elements are called *plated structures*. These structures are further classified as plates or shells, depending on whether they are plane or curved. In practice, these units are used in combination with beams or bars.
- The third category consists of structures composed of members with all three dimensions (i.e. length, breadth and depth) of the same order. The analysis of such structures is extremely complex, even when several simplifying assumptions are made.

For the most part, the structural engineer is concerned with skeletal structures. Increasing sophistication in available techniques of analysis has enabled the economic design of plated structures in recent years. Recent advances in finite-element methods and the availability of fast and powerful computers have enabled the economic analysis of complex structures. Three-dimensional analysis of structures is only rarely carried out. Under incremental loading, the initial deformation or displacement response of a member is largely elastic. Continued application of load results in large displacements with yield spreading through the cross-section before ultimate collapse occurs.

Line elements

The deformation response of a line element depends on a number of cross-sectional properties, such as area, A , second moment of area ($I_{xx} = \int y^2 dA$; $I_{yy} = \int x^2 dA$) and the product moment of area ($I_{xy} = \int xy dA$). The two axes xx and yy are orthogonal. For doubly symmetrical sections, the axes of symmetry are those for which $\int xy dA = 0$. These are known as principal axes. For a plane area, the principal axes may be defined as a pair of rectangular axes in its plane and passing through its centroid, such that the product moment of area $\int xy dA = 0$, the coordinates referring to the principal axes. If the plane area has an axis of symmetry, it is obviously a principal axis (by symmetry $\int xy dA = 0$). The other axis is at right angles to it, through the centroid of the area.

Tables of properties of sections (including the centroid and shear centre of each section) are available as published data.

If a section has no axis of symmetry (e.g. an angle section), the principal axes must be determined. Referring to Figure 4.2, if uu and vv are the principal axes, the angle α between the uu and xx axes is given by

$$\tan 2\alpha = \frac{-2I_{xy}}{I_{xx} - I_{yy}} \quad (4.1)$$

$$I_{uu} = \frac{I_{xx} + I_{yy}}{2} + \frac{I_{xx} - I_{yy}}{2} \cos 2\alpha - I_{xy} \sin 2\alpha \quad (4.2)$$

$$I_{vv} = \frac{I_{xx} + I_{yy}}{2} - \frac{I_{xx} - I_{yy}}{2} \cos 2\alpha + I_{xy} \sin 2\alpha \quad (4.3)$$

The values of α , I_{uu} and I_{vv} are available in published design guides (e.g. SCI, 2007).

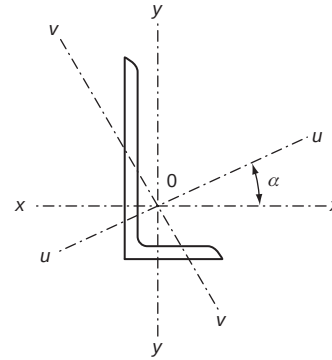


Figure 4.2 Angle section (no axis of symmetry)

Elastic analysis of line elements under axial loading

When a cross-section is subjected to a compressive or tensile axial load, P , the resulting stress is given by the load divided by the area of the section, i.e. P/A . An axial load is defined as one acting at the centroid of the section. When loads are introduced to a section in a uniform manner (e.g. through a heavy end plate), this represents the state of stress throughout the section. Conversely, when a tensile load is introduced via a bolted connection, there will be regions of the member where stress concentrations occur and plastic behaviour may be evident locally, even though the mean stress across the section is well below yield.

If the force P is not applied at the centroid, the longitudinal direct stress distribution will no longer be uniform. If the force is offset by eccentricities e_x and e_y , measured from the centroidal axes in the x and y directions, the equivalent set of actions are (a) an axial force P , (b) a bending moment $M_x = Pe_x$ in the yz plane and (c) a bending moment $M_y = Pe_y$ in the xz plane (see Figure 4.3). The method of evaluating the stress distribution resulting from an applied moment is given in a later section. The total stress at any section can be obtained as the algebraic sum of the stresses resulting from P , M_x and M_y .

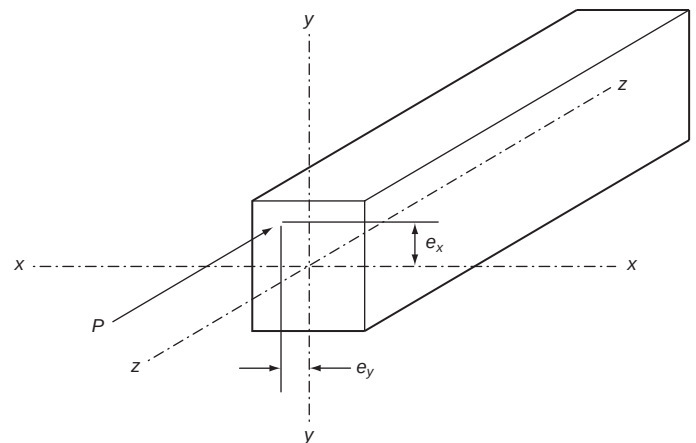


Figure 4.3 Compressive force applied eccentrically with reference to the centroidal axis

Elastic analysis of line elements in pure bending

For a section with at least one axis of symmetry and acted on by a bending moment in the plane of symmetry, the Bernoulli equation of bending may be used as the basis to determine both stresses and deflections within the elastic range. The following assumptions form the basis of the theory.

- The beam is subjected to a pure moment (i.e. shear is absent). (Generally the deflections due to shear are small compared with those due to flexure; this is not true of deep beams.)
- Plane sections before bending remain plane after bending.
- The material has a constant value of modulus of elasticity (E) and is linearly elastic.

The following equation results (see Figure 4.4):

$$\frac{M}{I} = \frac{f}{y} = \frac{E}{R} \quad (4.4)$$

where M is the applied moment, I is the second moment of area about the neutral axis, f is the longitudinal direct stress at any point within the cross-section, y is the distance of the point from the neutral axis, E is the modulus of elasticity and R is the radius of curvature of the beam at the neutral axis.

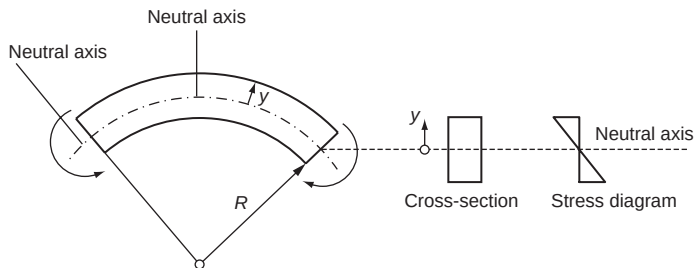


Figure 4.4 Pure bending

The stress at any section can then be obtained as

$$f = \frac{My}{I} \quad (4.5)$$

For a given section (with a known value of I) the stress varies linearly from zero at the neutral axis to a maximum at extreme fibres on either side of the neutral axis:

$$f_{\max} = \frac{My_{\max}}{I} = \frac{M}{Z} \quad (4.6)$$

where

$$Z = \frac{I}{y_{\max}} \quad (4.7)$$

The term Z is known as the elastic section modulus and is tabulated in section tables for steel members (SCI, 2007). The elastic moment capacity of a given section may be found directly as the product of the elastic section modulus, Z , and the maximum allowable stress.

If the section is doubly symmetrical, the neutral axis is midway between the two extreme fibres. Hence, the maximum tensile and compressive stresses will be equal. For an asymmetrical section, this will not be the case, as the value of y for the two extreme fibres will be different.

For a monosymmetrical section, such as the T-section shown in Figure 4.5, subjected to a moment acting in the plane of symmetry, the elastic neutral axis will be the centroidal axis. These equations are still valid. The values of $y_{\max}$ for the two extreme fibres (one in compression and the other in tension) are different. For an applied sagging (positive) moment, shown in Figure 4.5, the extreme fibre stress in the flange will be compressive and that in the stalk will be tensile. The numerical values of the maximum tensile and compressive stresses will differ. In the case sketched in Figure 4.5, the magnitude of the tensile stress will be greater, as $y_{\max}$ in tension is greater than that in compression.

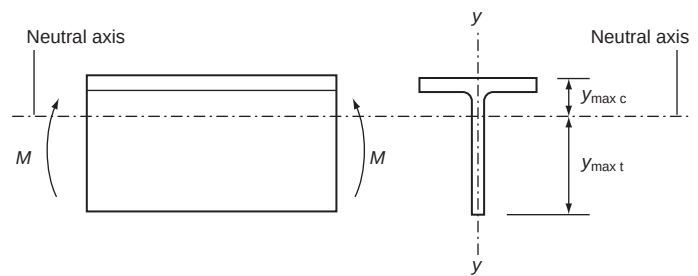


Figure 4.5 Monosymmetrical section subjected to bending

Caution has to be exercised in extending the pure bending theory to asymmetrical sections. There are two special cases where no twisting occurs.

- Bending about a principal axis in which no displacement perpendicular to the plane of the applied moment results.
- The plane of the applied moment passes through the shear centre of the cross-section.

When a cross-section is subjected to an axial load and a moment such that no twisting occurs, the stresses may be determined by resolving the moment into components M_{uu} and M_{vv} about the principal axes uu and vv and combining the resulting longitudinal stresses with those resulting from axial loading:

$$f_{u,v} = \pm \frac{P}{A} \pm \frac{M_{uu}v}{I_{uu}} \pm \frac{M_{vv}u}{I_{vv}} \quad (4.8)$$

For a section with two axes of symmetry (see Figure 4.3), this simplifies to

$$f_{x,y} = \pm \frac{P}{A} \pm \frac{M_{xx}y}{I_{xx}} \pm \frac{M_{yy}x}{I_{yy}} \quad (4.9)$$

Pure bending does not cause the section to twist. When the shear force is applied eccentrically in relation to the shear centre of the cross-section, the section twists and initially plane sections no longer remain plane. The response is complex and

consists of a twist and a deflection with components in and perpendicular to the plane of the applied moment. This is not discussed in this chapter. A simplified method of calculating the elastic response of cross-sections subjected to twisting moments is given in an SCI publication by Nethercot *et al.* (1998).

Elastic analysis of line elements subject to shear

Pure bending, discussed in the preceding section, implies that the shear force applied to the section is zero. Application of transverse loads on a line element will, in general, cause a bending moment that varies along its length, and hence a shear force that also varies along the length is generated.

If the member remains elastic and is subjected to bending in a plane of symmetry (such as the vertical plane in a doubly symmetrical or monosymmetrical beam), the shear stresses caused vary with the distance from the neutral axis.

For a narrow rectangular cross-section of breadth b and depth d , subjected to a shear force V and bent in its strong direction (see Figure 4.6(a)), the shear stress varies parabolically from zero at the lower and upper surfaces to a maximum value, $q_{\max}$, at the neutral axis given by

$$q_{\max} = \frac{3V}{2bd} \quad (4.10)$$

that is, 50% higher than the average value.

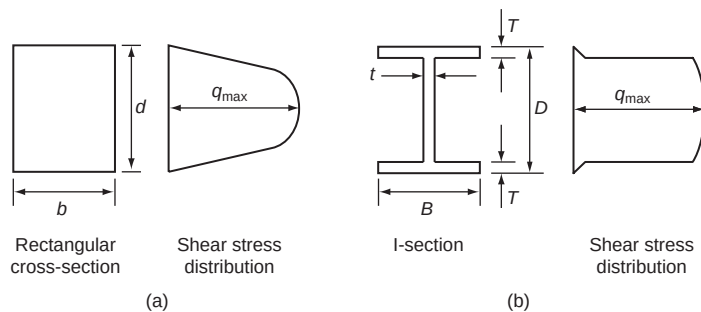


Figure 4.6 Shear stress distribution: (a) rectangular cross-section; (b) I-section

For an I-section (Figure 4.6(b)), the shear distribution can be evaluated from

$$q = \frac{V}{IB} \int_{y=h}^{y=h_{\max}} b y \, dy \quad (4.11)$$

where B is the breadth of the section at which shear stress is evaluated. The integration is performed over that part of the section remote from the neutral axis, i.e. from $y=h$ to $y=h_{\max}$, with a general variable width of b .

Clearly, for the I- (or T-) section, at the web–flange interface, the value of the integral will remain constant. As the section just inside the web becomes the section just inside the flange, the value of the vertical shear abruptly changes, as B changes from web thickness to flange width.

Flexural members

One of the most common structural elements is a *beam*; it bends when subjected to loads acting transversely to its centroidal axis or sometimes by loads acting both transversely and parallel to this axis. The discussions given in the following subsections are limited to idealised straight beams in which the centroidal axis is a straight line with shear centre coinciding with the centroid of the cross-section. The loads and reactions are assumed to lie in a plane that also contains the centroidal axis of the flexural member and the principal axis of every cross-section. If these conditions are satisfied, the beam will only bend in the plane of loading, without twisting.

Axial force, shear force and bending moment

The *axial force* at any transverse cross-section of a straight beam is the algebraic sum of the components acting parallel to the axis of the beam of all loads and reactions applied to the portion of the beam on either side of that cross-section. The *shear force* at any transverse cross-section of a straight beam is the algebraic sum of the components acting transverse to the axis of the beam of all the loads and reactions applied to the portion of the beam on either side of the cross-section. The *bending moment* at any transverse cross-section of a straight beam is the algebraic sum of the moments on either side of cross-section taken about an axis passing through the centroid of the cross-section. The axis about which the moments are taken is, of course, normal to the plane of loading.

Relation between load, shear, bending moment and deflection

When a beam is subjected to transverse loads, there exist certain relationships between load, shear and bending moment. Let us consider, for example, the beam shown in Figure 4.7 subjected to some arbitrary loading, p .

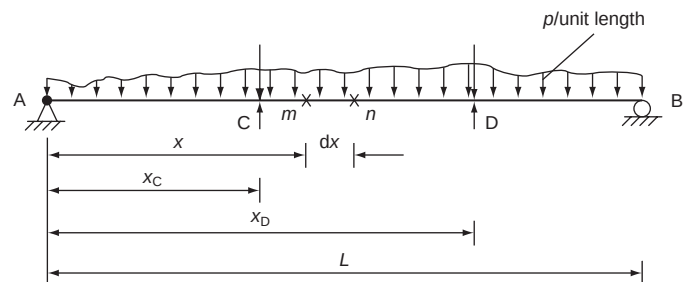


Figure 4.7 A beam under arbitrary loading

Let S and M be the shear and bending moment, respectively for any point m at a distance x , which is measured from A, being positive when measured to the right. The corresponding values of shear and bending moment at point n , at a small distance dx to the right of m , are $S+dS$ and $M+dM$, respectively. It can be shown, neglecting the second-order quantities, that

$$p = \frac{dS}{dx} \quad (4.12)$$

$$S = \frac{dM}{dx} \quad (4.13)$$

$$m = EI \frac{d^2 y}{dx^2} \quad (4.14)$$

and

$$p = EI \frac{d^4 y}{dx^4} \quad (4.15)$$

Beam deflection

There are several methods for determining beam deflections: (a) moment area method; (b) conjugate-beam method; (c) virtual work and (d) Castigliano's second theorem, among others. The first two methods are described next.

The elastic curve of a member is the shape taken by the neutral axis when the member deflects under load. The inverse of the radius of curvature at any point of this curve is obtained as

$$\frac{1}{R} = \frac{M}{EI} \quad (4.16)$$

in which M is the bending moment at the point and EI is the flexural rigidity of the beam section. Since the deflection is small, $1/R$ is approximately taken as $d^2 y/dx^2$, and Equation 4.16 may be rewritten as

$$M = EI \frac{d^2 y}{dx^2} \quad (4.17)$$

In Equation 4.17, y is the deflection of the beam at distance x measured from the origin of coordinate. The change in slope in a distance dx can be expressed as $M dx/EI$; hence, the slope in a beam is obtained as

$$\theta_B - \theta_A = \int_A^B \frac{M}{EI} dx \quad (4.18)$$

where θ_A and θ_B are the slopes at A and B of the deflected beam.

Equation 4.18 may be stated as follows: 'The change in slope between the tangents to the elastic curve at two points is equal to the area of the M/EI diagram between the two points.'

Once the change in slope between tangents to the elastic curve is determined, the deflection can be obtained by further integrating the slope equation. In a distance dx , the neutral axis changes in direction by an amount $d\theta$. The deflection of one point on the beam with respect to the tangent at another point due to this angle change is equal to $d\delta = x d\theta$, where x is the distance from the point at which deflection is desired to the particular differential distance.

To determine the total deflection from the tangent at one point A to the tangent at another point B on the beam, it is necessary to obtain a summation of the products of each angle $d\theta$ (from A to B) multiplied by the distance to the point where deflection is desired, or

$$\delta_B - \delta_A = \int_A^B \frac{Mx}{EI} dx \quad (4.19)$$

This equation can be stated as follows: 'The deflection of a tangent to the elastic curve of a beam with respect to a tangent at another point is equal to the moment of the M/EI diagram between the two points, taken about the point at which deflection is desired.'

Moment area method

The moment area method is most conveniently used for determining slopes and deflections for beams in which the direction of the tangent to the elastic curve at one or more points is known, such as cantilever beams, where the tangent at the fixed end does not change in slope. The method is easily applied to beams loaded with concentrated loads, because the moment diagrams consist of straight lines. These diagrams can be broken down into single triangles and rectangles. Beams supporting uniform loads or uniformly varying loads may be handled by integration.

It should be noted that the slopes and deflections that are obtained using the moment area theorems are with respect to tangents to the elastic curve at the points being considered. The theorems do not directly give the slope or deflection at a point in the beam as compared with the horizontal axis (except in one or two special cases); they give the change in slope of the elastic curve from one point to another or the deflection of the

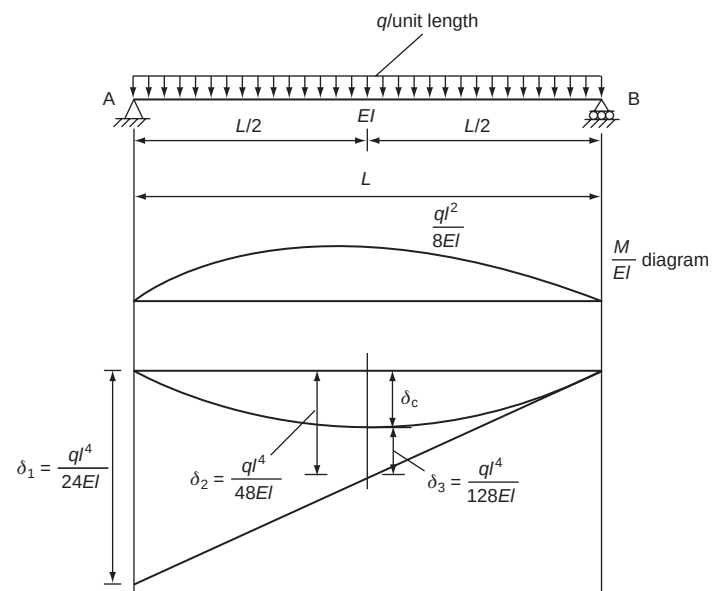


Figure 4.8 Deflection – simply supported beam under a uniformly distributed load

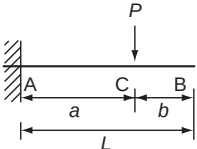
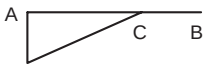
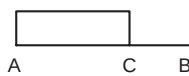
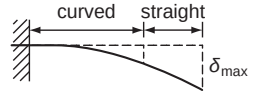
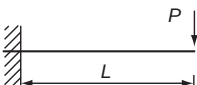
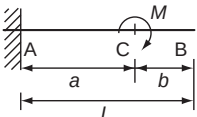
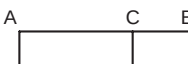
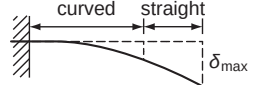
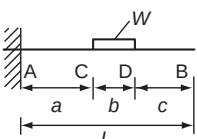
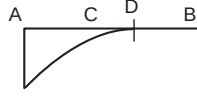
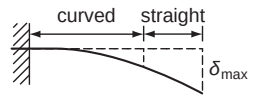
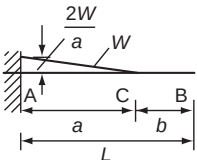
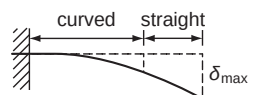
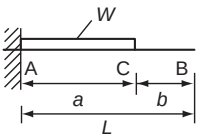
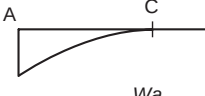
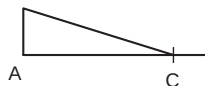
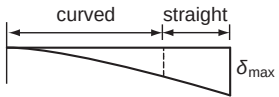
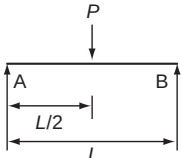
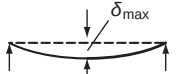
Loading	Bending moment	Shear force	Deflection
	 $M_{\max} = Pa$ at A 0 at C and B	 P from A to C	 curved straight $\delta_{\max} = \frac{Pa^3}{3EI} \left[1 + \frac{3}{2} \left(\frac{L-a}{a} \right) \right]$ $\delta_c = \frac{Pa^3}{3EI}$
	Substitute $a = L$ in case (1)		
		Zero	 curved straight $\delta_c = \frac{Ma^2}{2EI}$ $\delta_{\max} = \frac{Ma^2}{2EI} \left(1 + \frac{2b}{a} \right)$
	 $M_{\max} = W \left(a + \frac{b}{2} \right)$	 $SF_{\max} = W$	 curved straight $\delta_{\max} = \frac{W}{24EI} \times$ $(8a^3 + 18a^2b + 12ab^2 + 3b^3 + 12a^2c + 12abc + 4b^2c)$
	 $M_{\max} = \frac{Wa}{3}$	 $SF_{\max} = W$	 curved straight $\delta_{\max} = \frac{Wa^3}{15EI} \left(1 + \frac{5b}{4a} \right)$
	 $M_{\max} = \frac{Wa^2}{2}$	 $SF_{\max} = W$	 curved straight $\delta_{\max} = \frac{Wa^3}{8EI} \left(1 + \frac{4b}{3a} \right)$
	 $M_{\max} = \frac{PL}{4}$	 $(SF)_{\max} = 0.5P$	 $\delta_{\max} = \frac{5}{384} \frac{WL^3}{EI}$

Figure 4.9 Bending moment, shear force and deflected shape diagrams

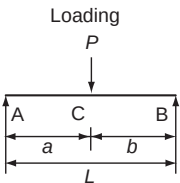
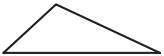
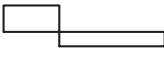
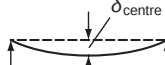
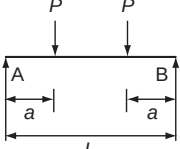
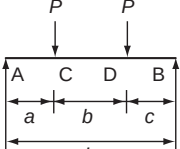
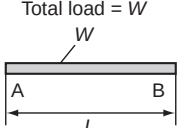
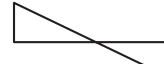
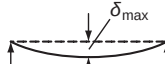
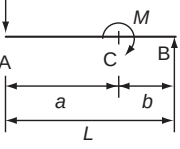
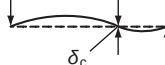
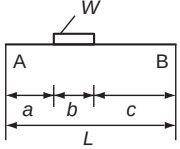
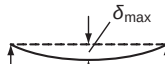
	 $M_{\max} = \frac{Pab}{L} \text{ at } C$	 On A, C, $(SF)_{\max} = \frac{Pb}{L}$ On C, B, $(SF)_{\max} = \frac{Pa}{L}$	 $\delta_{\text{centre}} = \frac{PL^3}{48EI} \left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3 \right]$ This value is always within 2½% of max deflection given by $Pb(L^2 - b^2)^{3/2}/(9+3LEI)$
	 $M_{\max} = Pa$	 $(SF)_{\max} = P$	 $\delta_{\max} = \frac{PL^3}{6EI} \left[\frac{3a}{4L} - \left(\frac{a}{L}\right)^3 \right]$
	 $M_C = \frac{Pa(b+2c)}{L}$ $M_D = \frac{Pc(b+2a)}{L}$	 $R_A = P \left(\frac{b+2c}{L} \right)$ $R_B = P \left(\frac{b+2a}{L} \right)$	Add the values of deflection, calculated for each P, using case 2
	 $M_{\max} = \frac{WL}{8}$	 $(SF)_{\max} = 0.5W$	 $\delta_{\max} = \frac{5}{384} \frac{Wl^3}{EI}$
	 $M_{CA} = \frac{Ma}{L}$ $M_{CB} = \frac{Mb}{L}$	 $(SF) = M/L$	 $\delta_c = \frac{Mab}{3MI} \left(\frac{a}{L} - \frac{b}{L} \right)$
	 $M_{\max} = \frac{Wa}{2} \left[\left(a + \frac{R_A^b}{W} \right) - a^2 \right]$	 $R_A = \frac{W}{L} \left(\frac{b}{2} + c \right)$ $R_B = \frac{W}{L} \left(\frac{b}{2} + a \right)$	 $\delta_{\max} = \frac{W}{384EI} (8L^3 - 4Lb^2 + b^3)$ when $a = c$

Figure 4.9 Continued

tangent at one point with respect to the tangent at another point. There are some cases in which beams are subjected to several concentrated loads or the combined action of concentrated and uniformly distributed loads (UDLs). In such cases, it is advisable to separate the concentrated loads and uniformly distributed loads; the moment area method can be applied separately to each of these loads. The final responses are obtained by the principle of superposition.

The use of the moment area method is illustrated by considering a simply supported beam subjected to uniformly distributed load q , as shown in Figure 4.8. The tangent to the elastic curve at each end of the beam is inclined. The deflection δ_1 of the tangent at the left end from the tangent at the right end is found as $ql^4/24EI$. The distance from the original chord between the supports and the tangent at the right end, δ_2 , can be computed as $ql^4/48EI$. The deflection of a tangent at the centre from a tangent at right end, δ_3 , is determined in this step as $ql^4/128EI$. The difference between δ_2 and δ_3 gives the centre line deflection as $5ql^4/384EI$.

In general, the bending moment, shear force and deflected shape diagrams can be drawn for a given loading by employing Equations 4.12 to 4.19. Examples of applications to cantilevers

and simply supported beams for some of the commonly encountered loading cases are given in Figure 4.9.

Conjugate-beam method

The conjugate-beam method (first developed by Otto Mohr in 1860) requires the same amount of computation as the moment area theorems to determine the slope or deflection of a beam. It relies on the similarity between several computations.

- The equation for the shear (V) compares with that for the slope (θ).
- The equation for the moment (M) compares with that for the displacement.
- The application of the external load (w) to compute the moment (M) compares with that for the application of the moment (M) to compute displacement.

To make use of this comparison, we employ a 'conjugate beam' with the same length as the 'real' beam; the computation follows the application of these two theorems:

- The slope at a point in the 'real' beam is equal to the shear at the corresponding point in the 'conjugate' beam.

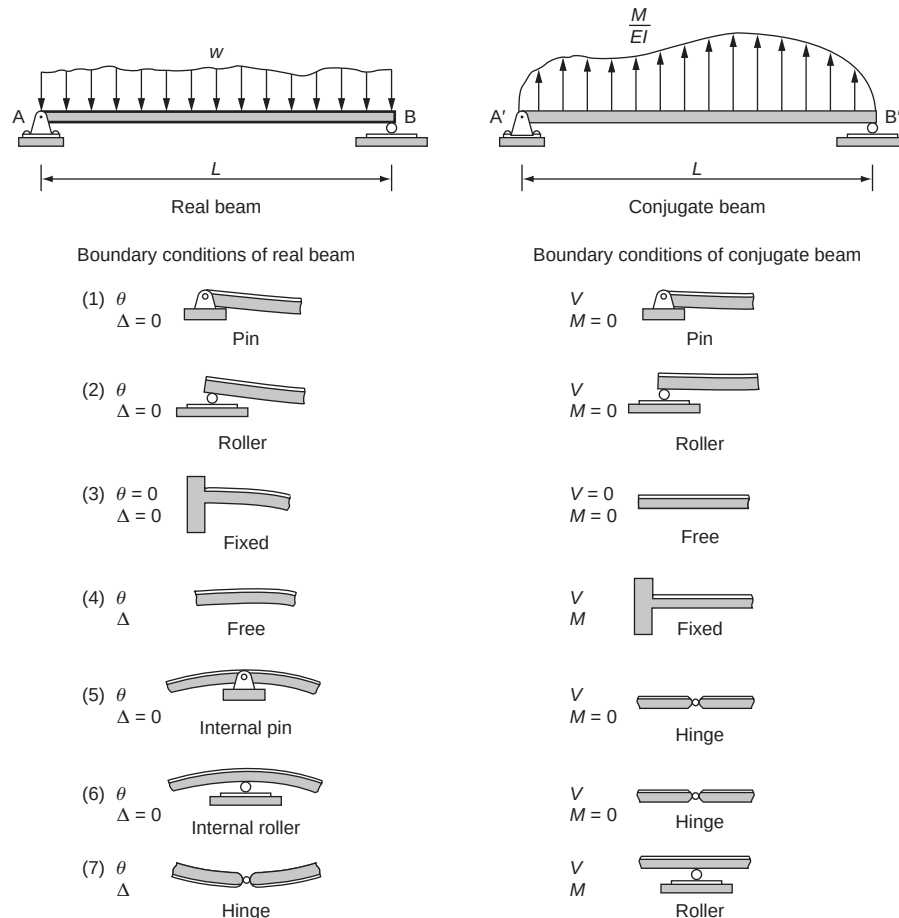


Figure 4.10 Real and conjugate beams

- The displacement of a point in the 'real' beam is equal to the moment at the corresponding point in the 'conjugate' beam.

This method is valuable for evaluating deflections of more complex cases, such as fixed beams. When employing the conjugate-beam method, it is important that proper boundary conditions be used. These are given in Figure 4.10.

Fixed-ended beams

When the ends of a beam are held firmly and are not free to rotate under the action of applied loads, the beam is known as a built-in or encasté beam and is statically indeterminate. The bending moment diagram for such a beam can be considered to consist of two parts, namely the free (bending) moment diagram obtained by treating the beam as if the ends are simply supported and the fixing (moment) diagram resulting from the restraints imposed at the ends of the beam. It can be shown that:

1. The area of the fixing (bending) moment diagram is equal to that of the free (bending) moment diagram.
2. The centres of gravity of the two diagrams lie in the same vertical line, that is, they are equidistant from either end of the beam.

The bending moment diagram for a fixed beam can be obtained as illustrated in the example shown in Figure 4.11, where PQUT is the free moment diagram, M_s , and PQRS is the fixing moment diagram, M_i . The net bending moment diagram, M is shown shaded. If A_s is the area of the free bending moment diagram and A_i the area of the fixing moment diagram then, from item (1), $A_s = A_i$ and

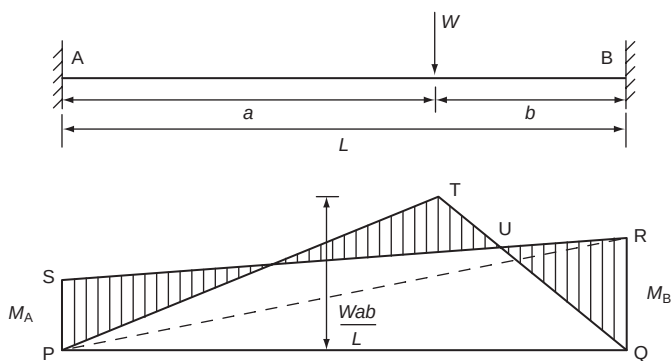


Figure 4.11 Fixed beam

$$\frac{1}{2} \times \frac{Wab}{L} \times L = \frac{1}{2} (M_A + M_B) \times L \quad (4.20)$$

$$M_A + M_B = \frac{Wab}{L} \quad (4.21)$$

Equating the moment about A of A_s and A_i , we have

$$M_A + 2M_B = \frac{Wab}{L^3} (2a^2 + 3ab + b^2) \quad (4.22)$$

Solving these two equations for M_A and M_B , we obtain

$$\left. \begin{aligned} M_A &= \frac{Wab^2}{L^2} \\ M_B &= \frac{Wa^2b}{L^2} \end{aligned} \right\} \quad (4.23)$$

The shear force can be determined once the bending moment is known. The shear forces at the ends of the beam, A and B, are

$$S_A = \frac{M_A - M_B}{L} + \frac{Wb}{L} \quad (4.24)$$

$$S_B = \frac{M_B - M_A}{L} + \frac{Wa}{L} \quad (4.25)$$

The bending moment and shear force diagrams for some typical loading cases are shown in Figure 4.12.

Continuous beams are statically indeterminate. Bending moments in these beams are functions of the geometry, moments of inertia and modulus of elasticity of individual members, besides the load and span. They may be determined by Clapeyron's theorem of three moments, the moment distribution method or the slope deflection method.

The theorem of three moments is applied to two adjacent spans at a time and the resulting equations in terms of unknown support moments are solved. The theorem states that

$$M_A L_1 + 2M_B (L_1 + L_2) + M_C L_2 = 6 \left(\frac{A_1 x_1}{L_1} + \frac{A_2 x_2}{L_2} \right) \quad (4.26)$$

where M_A , M_B and M_C , are the hogging moment at the supports A, B and C, respectively, of two adjacent spans of uniform cross-section and lengths L_1 and L_2 (Figure 4.13); A_1 and A_2 are the areas of bending moment diagrams produced by the vertical loads on the simple spans AB and BC, respectively; x_1 is the centroid of A_1 from A and x_2 is the distance of the centroid of A_2 from C. If the beam section is constant within a span but remains different for each of the spans, Equation 4.26 is written as

$$M_A \frac{L_1}{I_1} + 2M_B \left(\frac{L_1}{I_1} + \frac{L_2}{I_2} \right) + M_C \frac{L_2}{I_2} = 6 \left(\frac{A_1 x_1}{L_1 I_1} + \frac{A_2 x_2}{L_2 I_2} \right) \quad (4.27)$$

in which I_1 and I_2 are the moments of inertia of the beam section in spans L_1 and L_2 , respectively.

Loading	Bending moment	Shear force	Deflection
	 $BM_{\max} = \pm \frac{PL}{8}$	 $(SF)_{\max} = P/2$	 $\delta_{\max} = \frac{PL^3}{192EI}$
	 $\frac{2Pa^2b^2}{L^3}$ $-\frac{Pab^2}{L^2}$ $-\frac{Pba^2}{L^2}$	 $(SF)_A = P \left(\frac{b}{L} \right)^2 \left(1 + \frac{2a}{L} \right)$ $(SF)_B = P \left(\frac{a}{L} \right)^2 \left(1 + \frac{2b}{L} \right)$	 $\delta_{\max} = \frac{2Pa^2b^3}{3E(3L-2a)^2}$ when $x = L^2/(3L-2a)$
	 $-\frac{Pa(L-a)}{L}$ $-\frac{Pa(L-a)}{L}$	 $(SF)_A = (SF)_B = P$	 $\delta_{\max} = \frac{PL^3}{6EI} \left[\frac{3a^2}{4L^2} - \left(\frac{a}{L} \right)^3 \right]$
	 $M_A = M_B = \frac{-WL}{12}$ $M_C = \frac{+WL}{24}$	 $(SF)_A = (SF)_B = \frac{W}{2}$	 $\delta_{\max} = \frac{WL^3}{384EI}$
	 $M_{AC} = M \frac{b}{L^2} (3a-L)$ $M_{BC} = -M \frac{a}{L^2} (3b-L)$ $M_{CA} = -M \left(1 - \frac{a}{L} \right) \left[1 - \frac{3a}{L} + 6 \left(\frac{a}{L} \right)^2 \right]$	 $(SF)_A = \frac{M_{AC} + M_{CA}}{a}$ $(SF)_B = \frac{M_{CB} + M_{BC}}{b}$	 $\delta_c = \frac{Ma^2}{2EI} \left(1 - \frac{a}{L} \right)^2 \left(1 - \frac{b}{L} \right)$
	 $M_A = \frac{-W}{12L^2b} \left[\frac{(b+c)^3}{(4L-3b-3c)} - c^3(4L-3c) \right]$ $M_B = \frac{-W}{12L^2b} \left[\frac{(a+b)^3}{(4L-3a-3b)} - a^3(4L-3a) \right]$	 $(SF)_A = \left(\frac{M_A - M_B}{L} \right) \frac{+W}{L} \left(\frac{b}{2} + c \right)$ $(SF)_B = \left(\frac{M_B - M_A}{L} \right) \frac{+W}{L} \left(\frac{b}{2} + a \right)$	 $\delta_{\max} = \frac{W}{384EI} (L^3 + 2L^2a + 4La^2 - 8a^3)$ when $a = c$

Figure 4.12 Bending moment, shear force and deflection diagrams for typical loading in fixed beams

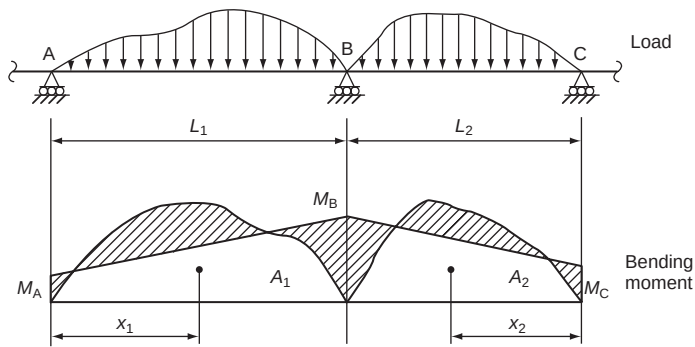


Figure 4.13 Continuous beams

Similar equations for different pairs of adjacent spans are set up at a time, until all the spans have been covered, two at a time. These provide sufficient numbers of additional equations, which, together with the equations of statics, enable the calculation of bending moments at all sections.

Values of bending moments and reactions at supports for two-, three- and four-span continuous beams are presented for several simple cases of loading in Figure 4.14. These will be of value to designers for carrying out rapid calculations and checks.

Slope deflection method

This method is a special case of the stiffness method of analysis, and is convenient for hand analysis of small structures. Members are assumed to be of constant section between each pair of supports. It is further assumed that the joints in a structure may rotate or deflect, but that the angles between the members meeting at a joint remain unchanged. Moments at the ends of frame members are expressed in terms of the rotations and deflections of the joints.

The member force-displacement equations that are needed for the slope deflection method are written for a member AB in a frame. This member, which has its undeformed position along

(a)

Loading	Max. +ve BM	Max. -ve BM	Reactions at supports		
			1	2	3
	$+0.156WL$	$-0.188WL$	$0.313W$	$1.375W$	$0.313W$
	$+0.203WL$	$-0.094WL$	$0.406W$	$0.688W$	$-0.094W$ (downwards)
	$+0.070WL$	$-0.125WL$	$0.375W$	$1.25W$	$0.375W$
	$+0.063WL$	$-0.063WL$	$0.438W$	$0.625W$	$-0.063W$ (downwards)

Figure 4.14 Bending moments and reactions of continuous beams: (a) two-span continuous beams; (b) three-span continuous beams; (c) four-span continuous beams

(b)

Loading	Max. +ve BM	Max. -ve BM	Reactions at supports			
			1	2	3	4
	$0.175WL$ at (5)	$-0.150WL$ at (2)	$0.35W$	$1.15W$	$1.15W$	$0.35W$
	$0.200WL$ at (5)	$-0.100WL$ at (2)	$0.4W$	$0.725W$	$-0.15W$ (downwards)	$-0.025W$ (downwards)
	$0.175WL$ at (6)	$-0.075WL$ at (2) & (3)	$-0.075W$	$0.575W$	$0.575W$	$-0.075W$ (downwards)
	$0.213WL$ at (5) & (7)	$-0.075WL$ at (2) & (3)	$0.425W$	$0.575W$	$0.575W$	$0.425W$
	$0.163WL$ at (5)	$-0.175WL$ at (2)	$0.325W$	$1.3W$	$0.425W$	$-0.5W$ (downwards)
	$0.080WL$ at (5)	$-0.100WL$ at (2) & (3)	$0.40W$	$1.10W$	$1.10W$	$0.40W$
	$0.094WL$ at (5)	$-0.067WL$ at (2)	$0.433W$	$0.65W$	$-0.10W$ (downwards)	$0.017W$
	$0.075WL$ at (6)	$-0.05WL$ at (2) & (3)	$-0.05W$	$0.55W$	$0.55W$	$-0.05W$ (downwards)
	$0.073WL$ at (5)	$-0.117WL$ at (2)	$0.383W$	$1.2W$	$0.45W$	$-0.033W$ (downwards)
	$0.101WL$	$-0.05WL$	$0.45W$	$0.55W$	$0.55W$	$0.45W$

Figure 4.14 Continued

(c)

Loading	Max. +ve BM	Max. -ve BM	Reactions at support				
			1	2	3	4	5
	$0.17WL$ at (6)	$-0.161WL$ at (2)	$0.339W$	$1.214W$	$0.893W$	$1.214W$	$0.339W$
	$0.20WL$ at (6)	$-0.10WL$ at (2)	$0.4W$	$0.728W$	$-0.161W$	$0.04W$	$-0.07W$
	$0.173WL$ at (7)	$-0.08WL$ at (3)	$-0.074W$	$0.567W$	$0.607W$	$-0.121W$	$0.02W$
	$0.21WL$ at (6)	$-0.08WL$ at (2)	$0.42W$	$0.607W$	$0.446W$	$0.607W$	$-0.08W$
	$0.16WL$ at (6)	$-0.181WL$ at (2)	$0.319W$	$0.335W$	$0.286W$	$0.647W$	$0.413W$
	$0.143WL$ at (7)	$-0.161WL$ at (3)	$-0.054W$ (downwards)	$0.446W$	$1.214W$	$0.446W$	$-0.054W$ (downwards)
	$0.077WL$	$-0.107WL$	$0.393W$	$1.143W$	$0.929W$	$1.143W$	$0.393W$
	$0.094WL$	—	$0.433W$	$0.652W$	$-0.107W$	$0.027W$	$-0.05W$
	$0.074WL$ at (7)	$-0.054WL$ at (3)	$-0.049W$	$0.545W$	$0.571W$	$-0.08W$	$0.013W$
	$0.098WL$ at (9)	$-0.058WL$ at (4)	$0.38W$	$1.223W$	$0.357W$	$0.598W$	$0.442W$
	$0.10WL$ at (6)	$-0.054WL$ at (2) & (4)	$0.446W$	$0.572W$	$0.464W$	$0.572W$	$-0.054W$
	$0.056WL$ at (7)	$-0.107WL$ at (3)	$-0.036W$	$0.464W$	$1.143W$	$0.464W$	$-0.036W$

Figure 4.14 Continued

the x axis is deformed into the configuration shown in Figure 4.15. The positive axes, along with the positive member end force components and displacement components, are shown in the figure (M_{AB} is the end moment at A on AB and M_{BA} is the end moment at B on BA).

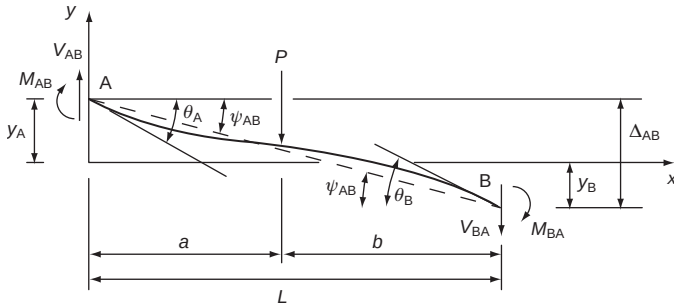


Figure 4.15 Deformed configuration of a beam

The equations for end moments are written as

$$M_{AB} = \frac{2EI}{\ell}(2\theta_A + \theta_B - 3\psi_{AB}) + M_{FAB} \quad (4.28)$$

$$M_{BA} = \frac{2EI}{\ell}(2\theta_B + \theta_A - 3\psi_{AB}) + M_{FBA} \quad (4.29)$$

in which M_{FAB} and M_{FBA} are fixed-end moments at supports A and B, respectively, owing to the applied load (see Figure 4.13) and ψ_{AB} is the rotation as a result of the relative displacement between the member ends A and B, given as

$$\psi_{AB} = \frac{\Delta_{AB}}{\ell} = \frac{y_A + y_B}{\ell} \quad (4.30)$$

where Δ_{AB} is the relative deflection of the beam ends and y_A and y_B are the vertical displacements at ends A and B, respectively. The slope deflection equations (Equations 4.28 and 4.29) show that the moment at the end of a member depends on the member properties EI , dimension P and displacement quantity. The fixed-end moments reflect the transverse loading on the member.

Application of slope deflection method to frames

The slope deflection equations may be applied to statically indeterminate frames with or without sidesway. A frame may be subjected to sidesway if the loads, member properties and dimensions of the frame are not symmetrical about the centre line. Application of slope deflection method can be illustrated by the following example.

Example

Consider the frame shown in Figure 4.16. The support at D is assumed to sink downwards by 12 mm and rotate in the clockwise direction by 0.002 radians. Equations 4.28 and 4.29 can be applied to each of the members of the frame and the member end moments may be obtained as follows.

Member AB:

$$M_{AB} = \frac{2EI}{8} \left(2\theta_A + \theta_B - \frac{3\Delta}{8} \right) + M_{FAB} \quad (4.31)$$

$$M_{BA} = \frac{2EI}{8} \left(2\theta_B + \theta_A - \frac{3\Delta}{8} \right) + M_{FBA} \quad (4.32)$$

and

$$\theta_A = 0, \quad M_{FAB} = M_{FBA} = 0 \quad (4.33)$$

as there are no externally applied loads.

Hence

$$M_{AB} = \frac{2EI}{8} (\theta_B - 3\psi) \quad (4.34)$$

$$M_{BA} = \frac{2EI}{8} (2\theta_B - 3\psi) \quad (4.35)$$

in which

$$\psi = \frac{\Delta}{8} \quad (4.36)$$

Member BC:

$$M_{BC} = \frac{2EI}{6} \left(2\theta_B + \theta_C - 3 \times \frac{\Delta'}{6} \right) + M_{FBC} \quad (4.37)$$

$$M_{CB} = \frac{2EI}{6} \left(2\theta_C + \theta_B - 3 \times \frac{\Delta'}{6} \right) + M_{FCB} \quad (4.38)$$

and

$$\Delta' = 0.012 \text{ m}, \quad M_{FBC} = M_{FCB} = 0 \quad (4.39)$$

as before.

Hence

$$M_{BC} = \frac{2EI}{6} (2\theta_B + \theta_C - 0.006) \quad (4.40)$$

$$M_{CB} = \frac{2EI}{6} (2\theta_C + \theta_B - 0.006) \quad (4.41)$$

Member CD:

$$M_{CD} = \frac{2EI}{6} \left(2\theta_C + \theta_D - \frac{3\Delta}{6} \right) + M_{FCD} \quad (4.42)$$

$$M_{DC} = \frac{2EI}{6} \left(2\theta_D + \theta_C - \frac{3\Delta}{6} \right) + M_{FDC} \quad (4.43)$$

and

$$\theta_D = 0.002 \text{ radians}, \quad M_{FCD} = M_{FDC} = 0 \quad (4.44)$$

as before.

Hence

$$M_{CD} = \frac{2EI}{6} \left(2\theta_C + 0.002 - \frac{\Delta}{2} \right) \quad (4.45)$$

$$M_{DC} = \frac{2EI}{6} \left(\theta_C + 0.004 - \frac{\Delta}{2} \right) \quad (4.46)$$

The joint moment conditions are

$$M_{BA} + M_{BC} = 0 \quad (4.47)$$

$$M_{CB} + M_{CD} = 0 \quad (4.48)$$

The shear condition is

$$\frac{M_{AB} + M_{BA}}{8} + \frac{M_{CD} + M_{DC}}{6} = 0 \quad (4.49)$$

Substituting for moments M_{BA} , M_{BC} , M_{CB} , M_{CD} , M_{BA} , M_{AB} , M_{BA} , M_{CD} and M_{DC} , and solving for θ_B , θ_C and Δ

$$\theta_B = 2.0441 \times 10^{-3} \text{ rad} \quad (4.50)$$

$$\theta_C = 1.8676 \times 10^{-3} \text{ rad} \quad (4.51)$$

$$\Delta = 10.587 \times 10^{-3} \text{ m} \quad (4.52)$$

Substituting these values of joint rotations and translation back into the slope deflection equations gives

$$M_{AB} = -38.52 \text{ kNm} \quad (4.53)$$

$$M_{BA} = +2.36 \text{ kNm} \quad (4.54)$$

$$M_{BC} = -2.36 \text{ kNm} \quad (4.55)$$

$$M_{CB} = -11.76 \text{ kNm} \quad (4.56)$$

$$M_{CD} = +11.76 \text{ kNm} \quad (4.57)$$

$$M_{DC} = +15.36 \text{ kNm} \quad (4.58)$$

This example illustrates that, even without applying a load, the frame can develop substantial moments, owing to the settlement of supports.

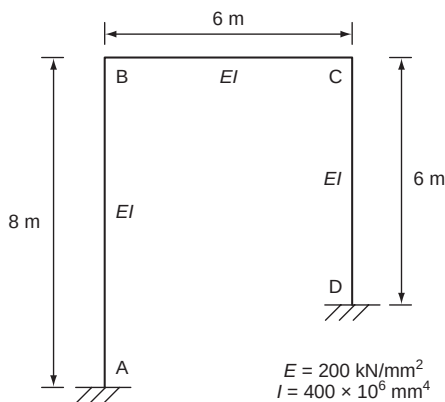


Figure 4.16 Portal frame example

Torsion

When a structural element is loaded such that the loading axis passes through the shear centre of the cross-section, no torsion will result. When the loading axis and the centroidal axis of a beam do not coincide, torsion will inevitably result. Asymmetrically loaded slabs supported on beams, interconnected girders, off-axis loading on beams and loading away from the shear centre are all instances where torsion is a significant factor in design.

When a circular cross-section is subjected to torsion (Figure 4.17), plane sections remain plane and the torsional behaviour is described by an equation similar to the bending equation, taking a form similar to Equation 4.4:

$$\frac{T}{J} = \frac{\tau}{r} = \frac{N\theta}{l} \quad (4.59)$$

where T is the applied torque, J is the polar moment of inertia, τ is the shear stress at a radius r , N is the modulus of rigidity and θ/l is the angle of rotation per unit length of the bar.

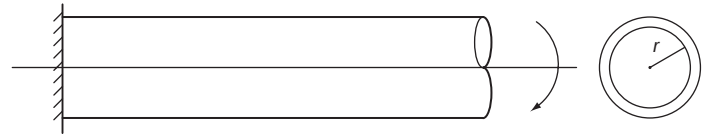


Figure 4.17 Torsion of a circular section

In this case, the shear strain is directly proportional to the distance from the neutral axis.

Conversely, when the cross-section is rectangular, the originally plane sections warp during twisting. The simple torsion equation (Equation 4.59) no longer applies, as both axial and circumferential shear stresses are produced. The maximum shear stresses occur on the periphery, at the middle of the longer sides of the rectangle. The torsional shear stress is given by

$$\tau = \gamma a N \theta \quad (4.60)$$

where γ is a function of the ratio of the longer side of the rectangle to its shorter side (a/b), a is the length of the shorter side and θ is the angle of twist in radians per unit length.

The twisting moment resistance is obtained by

$$T = \beta a^3 \gamma N \theta \quad (4.61)$$

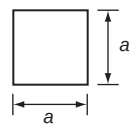
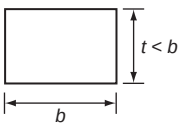
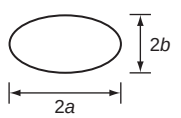
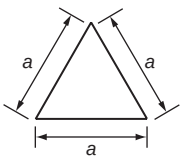
where β is a function of b/a .

The torsional resistance of a section composed of a number of rectangles (e.g. T-, L- or I-sections) is obtained approximately as the sum of torsional resistances of the component rectangles, i.e.

$$T = N \theta \sum \frac{1}{3} a^3 b \quad (4.62)$$

The summation will cover all the separate rectangles which, when combined, will result in the composite cross-section. Torsional properties of solid and open cross-sections are given in Figure 4.18. A detailed treatment of torsion in non-circular sections may be found in Timoshenko and Goodier (1970).

In general, a structural component subjected to uniform pure torsion (or St Venant torsion) develops only shear stresses. This implies that the section has no warping restraint. When there is restraint to longitudinal warping, (e.g. by providing a fixed end), there will be both shear and longitudinal stresses. Generally, non-uniform torsion can be computed as a combination of uniform torsion and warping restraint stresses.

Cross-section	Shear stress	Torsional constant J
	$\frac{2T}{\pi r^3}$	$\frac{1}{2} \pi r^4$
	$\frac{T}{0.208a^2}$ at midpoint each side	$0.141a^4$
	$\frac{3T}{bt^2} \left(1 + 0.6 \frac{t}{b}\right)$ at midpoint each long side	$\frac{bt^3}{3} \left[1 - 0.63 \frac{t}{b} + 0.052 \left(\frac{t}{b}\right)^2\right]$
	$\frac{2T}{\pi ab^2}$ at ends of minor axis	$\frac{\pi a^3 b^3}{a^2 + b^2}$
	$\frac{20T}{a^3}$ at midpoint each side	$\frac{a^4 + 3}{80}$

* $T = GJ\theta$, where T = torque, G = shearing modulus of elasticity, J = torsional stiffness, θ = angle of twist, radians per unit length

Figure 4.18 Torsional properties of solid and open cross-sections

More detailed treatment of torsion is provided later in this chapter in the section on plate and box girder analysis.

Trusses

A structure that is composed of a number of bars pin-connected at their ends to form a stable framework is called a truss. If all the bars lie in a plane, the structure is a planar truss. It is generally assumed that loads and reactions are applied to the truss only at the joints. The centroidal axis of each member is straight, coincides with the line connecting the joint centres at each end of the member, and lies in a plane that also contains the lines of action of all the loads and reactions.

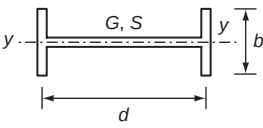
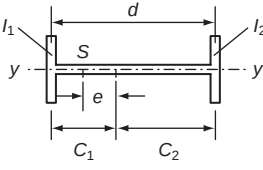
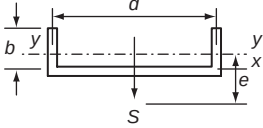
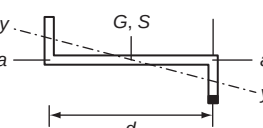
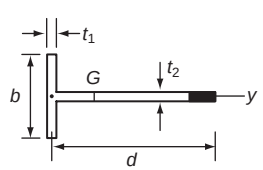
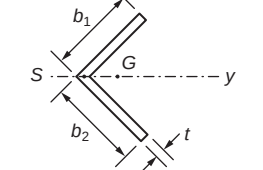
Many truss structures are three-dimensional and a complete analysis would require consideration of the full spatial interconnection of the members. However, in many bridge structures, the three-dimensional framework can be subdivided into planar components for analysis as planar trusses without seriously compromising the accuracy of the results. Figure 4.19 shows some typical idealised planar truss structures.

There exists a relation between the number of members, m , the number of joints, j , and reaction components, r :

$$m = 2j - r \quad (4.63)$$

which must be satisfied if the truss is to be statically determinate internally. The smallest number of reaction components required for external stability is r . If m exceeds $(2j - r)$, the excess members are called redundant members and the truss is said to be statically indeterminate.

Truss analysis gives the bar forces in a truss; for a statically determinate truss, these bar forces can be found by employing the laws of statics to assure internal equilibrium of the structure. The process requires repeated use of free-body diagrams, from which individual bar forces are determined. The *method of joints* is a technique of truss analysis in which the bar forces are determined by the sequential isolation of joints – the unknown bar forces at one joint are calculated and become known bar forces at subsequent joints. The other method is known as the *method of sections*, in which equilibrium of a part of the truss is considered.

Cross-section	Warping constant C	Location e of shear centre S
	$\frac{d^2 l_y}{4}$	
	$\frac{d^2 I_1 I_2}{I_y}$	$\frac{C_1 I_1 - C_2 I_2}{I_y}$
	$\frac{d^2 l_y}{4} \left[1 - \frac{\bar{x}(e - \bar{x})}{r_y^2} \right]$	$\frac{\bar{x}}{4} \left(\frac{d}{r_x} \right)^2$
	$\frac{d^2}{4} I_a$	
	$\frac{t_1^3 b^3}{144} + \frac{t_2^3 b^3}{36}$	
	$(b_1^3 + b_2^3) \frac{t^3}{36}$	

Note: The torsional stiffness J for cross-sections in this table can be determined sufficiently accurately for most applications by $J = \Sigma b t^3/3$. The warping constant C is usually negligible for the angle and the T .

Figure 4.18 Continued

Method of joints

An imaginary section may be completely passed around a joint in a truss. The joint has become a free body in equilibrium under the forces applied to it. The equations $\Sigma H=0$ and $\Sigma V=0$ may be applied to the joint to determine the unknown forces in members meeting there. It is evident that no more than two unknowns can be determined at a joint with these two equations.

Method of sections

If only a few member forces of a truss are needed, the quickest way to find these forces is by the method of sections. In this method, an imaginary cut ('section') is drawn through a stable and determinate truss. Thus, a section subdivides the truss into two separate parts. Since the entire truss is in equilibrium, any part of it must also be in equilibrium. Either of the two parts of the truss can be considered and the three equations of

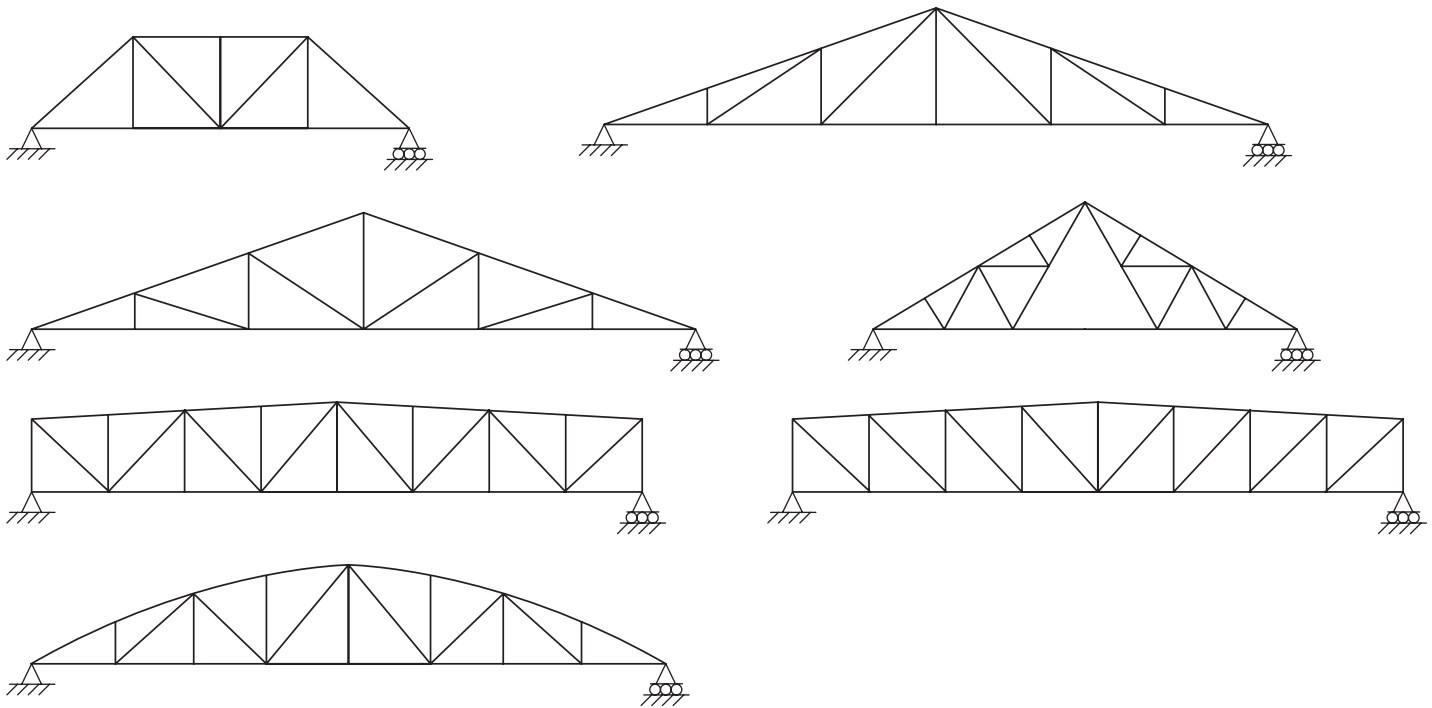


Figure 4.19 Typical planar trusses

equilibrium $\Sigma F_x = 0$, $\Sigma F_y = 0$ and $\Sigma M = 0$ can be applied to solve for member forces.

The example of the truss sketched in Figure 4.20(a) is considered. To calculate the force in member 3–5, F_{35} , a section A–A should be run to cut the member 3–5, as shown in the figure. It is only required to consider the equilibrium of one of the two parts of the truss. In this case, the portion of the truss on the left of the section is considered. The left portion of the truss, as shown in Figure 4.20(b), is in equilibrium under the action of the external and internal forces. Considering the equilibrium of forces in the vertical direction, one obtains

$$135 - 90 + F_{35} \sin 45^\circ = 0 \quad (4.64)$$

Therefore, F_{35} is obtained as

$$F_{35} = -45\sqrt{2} \text{ kN} \quad (4.65)$$

The negative sign indicates that the member force is compressive. The other member forces cut by the section can be obtained using the other equilibrium equations, namely $\Sigma M = 0$. More sections can be taken in the same way to solve for other member forces in the truss. The most important advantage of this method is that one can obtain the required member force without solving for the other member forces.

Compound trusses

A compound truss is formed by interconnecting two or more simple trusses. Examples of compound trusses are shown in Figure 4.21.

A typical compound roof truss is shown in Figure 4.21(a), in which two simple trusses are interconnected by means of a single member and a common joint. The compound truss shown in Figure 4.21(b) is commonly used in bridge construction; in this case, three members are used to interconnect two simple trusses at a common joint. In the cantilevered construction shown in Figure 4.21(c), three simple trusses are interconnected at their common joints.

The method of sections may be used to determine the member forces in the interconnecting members of compound trusses similar to those shown in Figures 4.21(a) and 4.21(b). However, in the case of a cantilevered truss (Figure 4.21(c)), the middle simple truss is isolated as a free-body diagram to find its reactions. These reactions are reversed and applied to the interconnecting joints of the other two simple trusses. After the interconnecting forces between the simple trusses are found, the simple trusses are analysed using the method of joints or the method of sections.

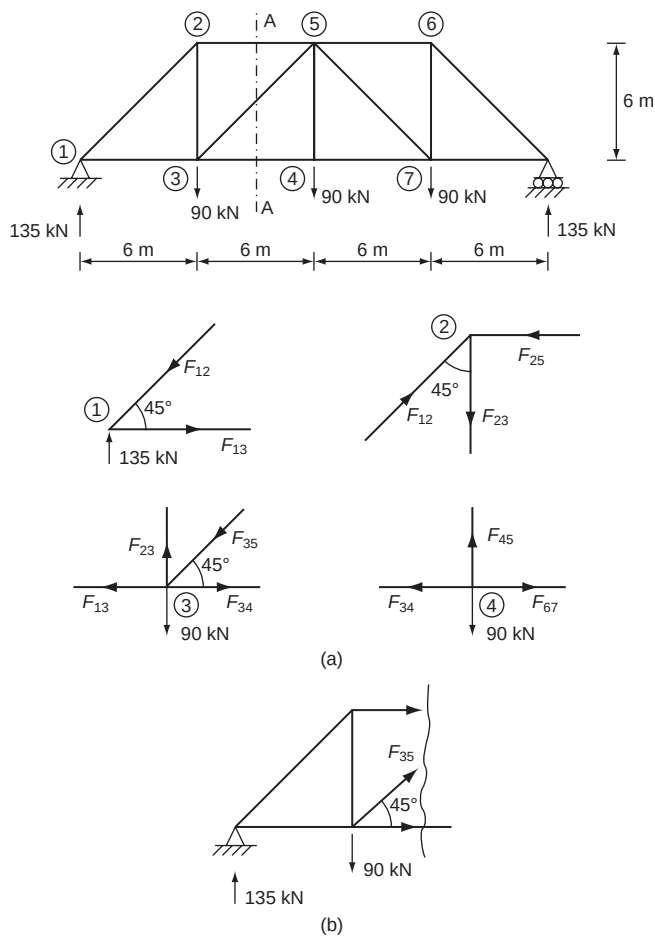


Figure 4.20 Example – method of sections, planar truss

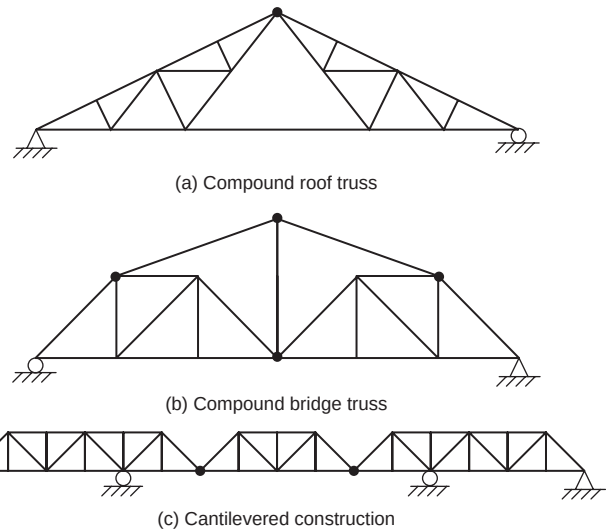


Figure 4.21 Compound trusses

whether the simple plane truss is stable. Simple plane trusses having $(2n - 3)$ members are not necessarily stable.

Influence lines

Bridge decks are required to support both static and moving loads. Each element of a bridge must be designed for the most severe conditions that can possibly be developed in that member. Live loads should be placed at the positions where they will produce severe conditions. The critical positions for placing live loads will not be the same for every member. A useful method of determining the most severe condition of loading is by using 'influence lines'.

An influence line for a particular response, such as reaction, shear force, bending moment or axial force, is defined as a diagram in which the ordinate at any point equals the value of that response attributable to a unit load acting at that point on the structure. Influence lines provide a systematic procedure for determining how the force (or a moment or shear) in a given part of a structure varies as the applied load moves about on the structure. Influence lines of responses of statically determinate structures consist only of straight lines, whereas this is not true of indeterminate structures. They are primarily used to determine where to place live loads to cause maximum effect.

Influence lines for shear in simple beams

Figure 4.22 shows influence lines for shear at two sections of a simply supported beam. It is assumed that positive shear occurs when the sum of the transverse forces to the left of a section is in the upwards direction or when the sum of the forces to the right of the section is downwards. A unit force is placed at various locations and the shear force at sections 1–1 and 2–2

Stability and determinacy

A stable and statically determinate plane truss should have at least three members, three joints and three reaction components. To form a stable and determinate plane truss of n joints, the three members of the original triangle plus two additional members for each of the remaining $(n - 3)$ joints are required. Thus, the minimum total number of members, m , required to form an internally stable plane truss is $m = (2n - 3)$. If a stable, simple, plane truss of n joints and $(2n - 3)$ members is supported by three independent reaction components, the structure is stable and determinate when subjected to a general loading. If the stable, simple, plane truss has more than three reaction components, the structure is externally indeterminate. This means that not all of the reaction components can be determined from the three available equations of statics. If the stable, simple, plane truss has more than $(2n - 3)$ members, the structure is internally indeterminate and hence all of the member forces cannot be determined from the $2n$ available equations of statics in the method of joints. The analyst must examine the arrangement of the truss members and the reaction components to determine

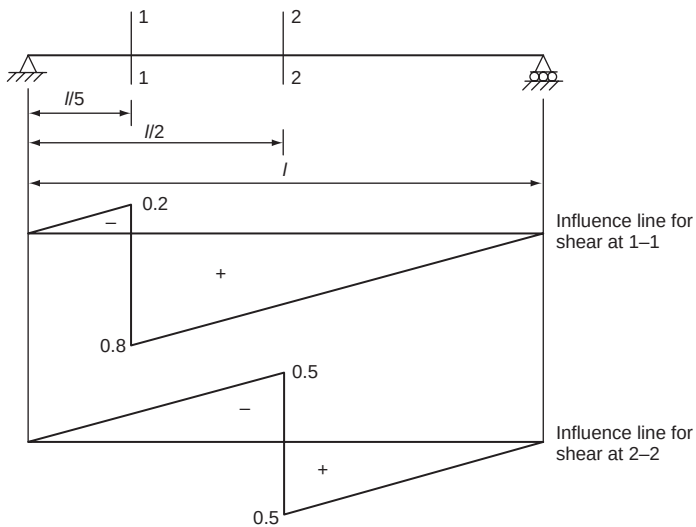


Figure 4.22 Influence lines for shear force

are obtained for each position of the unit load. These values give the ordinate of influence line with which the influence line diagrams for shear force at sections 1–1 and 2–2 can be constructed. Note that the slope of the influence line for shear on the left of the section is equal to the slope of the influence line on the right of the section. This information is useful in drawing shear force influence lines in other cases.

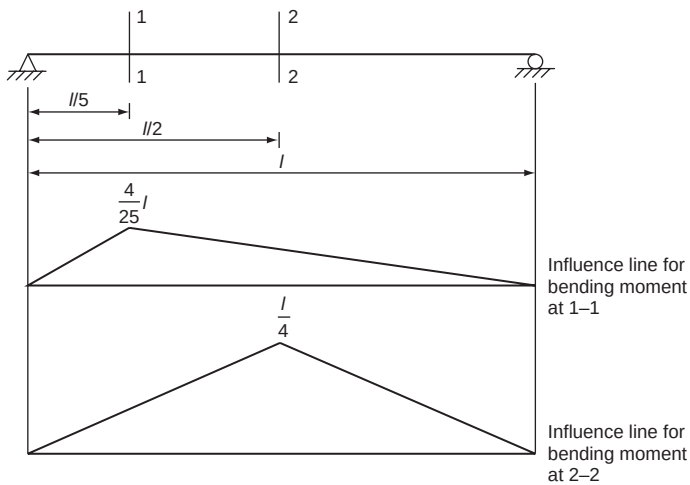


Figure 4.23 Influence lines for bending moment

It may be noted that a shear or bending moment diagram shows the variation of shear or moment across an entire structure *for loads fixed in one position*. Conversely, an influence line for shear or moment shows the variation of that response *at one particular section* in the structure caused by the *movement of a unit load* from one end of the structure to the other.

Influence lines can be used to obtain the value of a particular response for which it is drawn when the beam is subjected to any particular type of loading. If, for example, a uniform

load of intensity q_0 per unit length acts over the entire length of the simple beam shown in Figure 4.22, then the shear force at section 1–1 is given by the product of the load intensity, q_0 , and the net area under the influence line diagram. The net area is equal to $0.3P$ and the shear force at section 1–1 is, therefore, equal to $0.3q_0P$. In the same way, the bending moment at the section can be found as the area of the corresponding influence line diagram multiplied by the intensity of loading, q_0 . The bending moment at the section is, therefore, $(0.08P^2 \times q_0) = 0.08q_0P^2$.

Influence lines for trusses

Influence lines for support reactions and member forces may be constructed in the same manner as those for various beam functions. They are useful to determine the maximum load that can be applied to the truss. As the unit load moves across the truss, the ordinates for the responses under consideration may be computed for the load at each panel point. Member force, in most cases, need not be calculated for every panel point, because certain portions of influence lines can readily be seen to consist of straight lines for several panels. One method used for calculating the forces in a chord member of a truss is the method of sections discussed earlier.

The truss shown in Figure 4.24 is considered for illustrating the construction of influence lines for trusses.

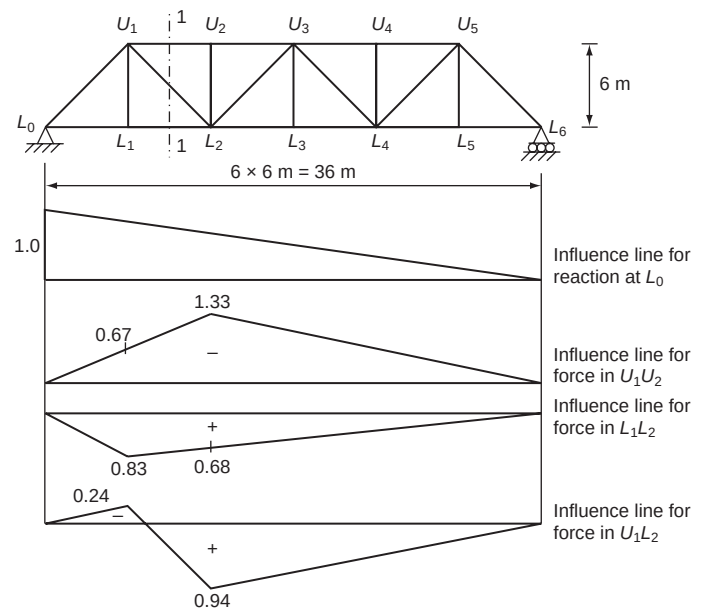


Figure 4.24 Influence lines for member force in a truss

The member forces in U_1U_2 , L_1L_2 and U_1L_2 are determined by passing a section 1–1 and considering the equilibrium of the free-body diagram of one of the truss segments. A unit load is placed at L_1 first and the force in U_1U_2 is obtained by taking moments about L_2 of all the forces acting on the right-hand

segment of the truss and dividing the resulting moment by the lever arm (the perpendicular distance of the force in U_1U_2 from L_2). The value thus obtained gives the ordinate of the influence diagram at L_1 in the truss. The ordinate at L_2 obtained similarly represents the force in U_1U_2 for a unit load placed at L_2 . The influence line can be completed with two other points, one at each of the supports. The force in the member L_1L_2 owing to the unit loads placed at L_1 and L_2 can be obtained in the same manner and the corresponding influence line diagram can be completed. By considering the horizontal component of force in the diagonal of the panel, the influence line for force in U_1L_2 can be constructed.

Figure 4.24 shows the respective influence diagram for member forces in U_1U_2 , L_1L_2 and U_1L_2 . Influence line ordinates for the force in a chord member of a 'curved-chord' truss may be determined by passing a vertical section through the panel and taking moments at the intersection of the diagonal and the other chord.

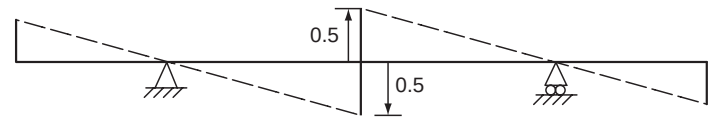


Figure 4.26 Influence line for midspan shear force

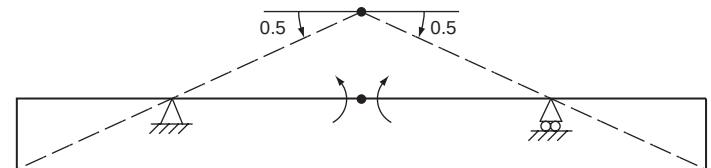


Figure 4.27 Influence line for midspan bending moment

Qualitative influence lines: Müller–Breslau principle

One of the most effective methods of obtaining influence lines involves using the Müller–Breslau principle, which states, 'The ordinates of the influence line for any response in a structure are equal to those of the deflection curve obtained by releasing the restraint corresponding to this response and introducing a corresponding unit displacement in the remaining structure.' In this way, the shape of the influence lines for both statically determinate and indeterminate structures can be easily obtained, especially for beams.

The following influence lines can be drawn:

- **Support reaction.** Remove the support and introduce a unit displacement in the direction of the corresponding reaction to the remaining structure, as shown in Figure 4.25 for a symmetrical overhang beam.
- **Shear.** Make a cut at the section and introduce a unit relative translation (in the direction of positive shear) without relative rotation of the two ends at the section, as shown in Figure 4.26.
- **Bending moment.** Introduce a hinge at the section (releasing the bending moment) and apply bending (in the direction corresponding to the positive moment) to produce a unit relative rotation of the two beam ends at the hinged section, as shown in Figure 4.27.

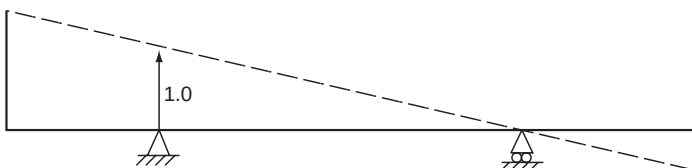


Figure 4.25 Influence line for support reaction

Influence lines for continuous beams and frames

Using the Müller–Breslau principle, the shape of the influence line of any response of a continuous beam or frame can be sketched easily. One of the methods for beam deflection can then be used to determine the ordinates of the influence line at critical points. Figures 4.28, 4.29 and 4.30 show the influence lines for bending moment at various points of two-, three- and four-span continuous beams, respectively.

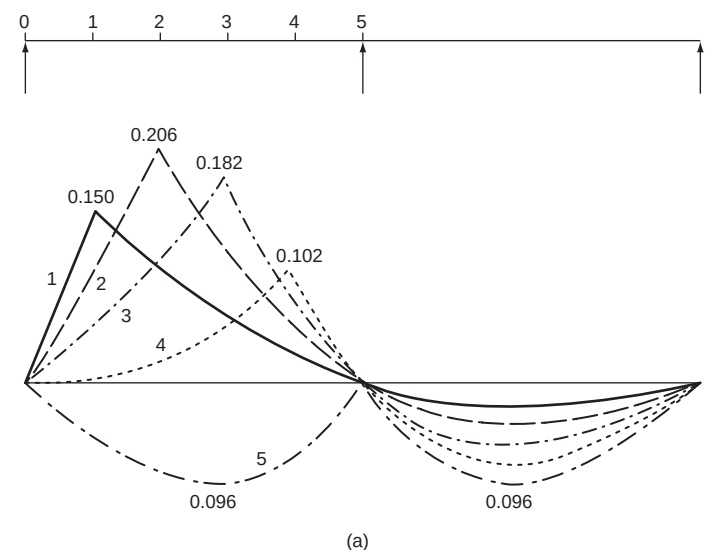
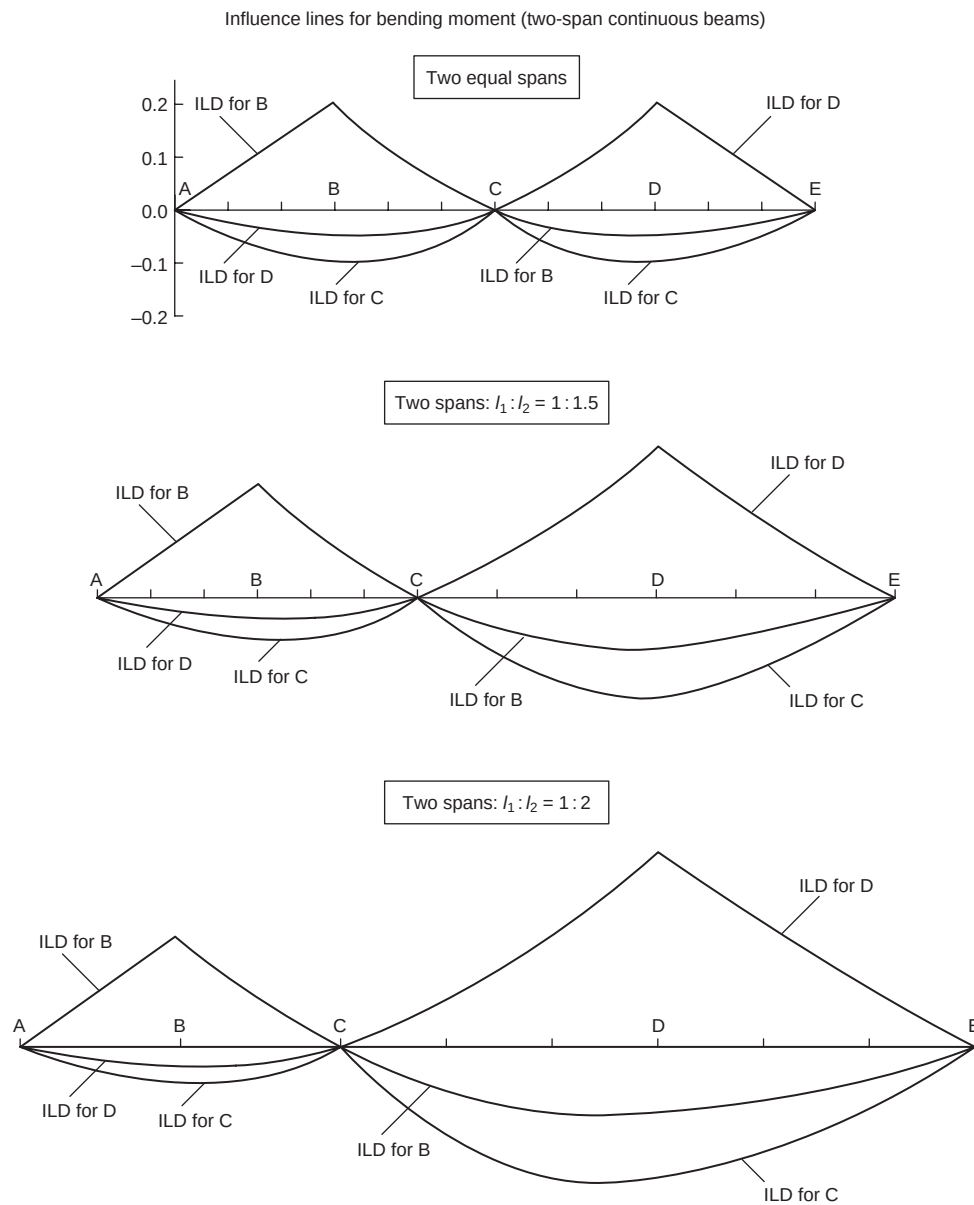


Figure 4.28 Influence lines for bending moment – two-span continuous beams (ILD, influence line diagram)



To obtain the maximum bending moment at any of the critical section due to a train of loads:

- (1) Plot on the diagram, the train of loads placed in the most adverse position
- (2) Bending moment due to a given load = Influence line ordinate $\times$ Load
- (3) Algebraically add all the values of the bending moment calculated as above

Max ordinate in ILD for	$l_1:l_2 = 1$		$l_1:l_2 = 1:1.5$		$l_1:l_2 = 1:2$	
	Span AC	Span CE	Span AC	Span CE	Span AC	Span CE
B	0.203	-0.047	0.213	-0.084	0.219	0.125
C	-0.094	-0.094	-0.075	-0.169	-0.063	-0.250
D	-0.047	0.203	-0.038	0.291	-0.031	-0.375

(b)

Figure 4.28 Continued

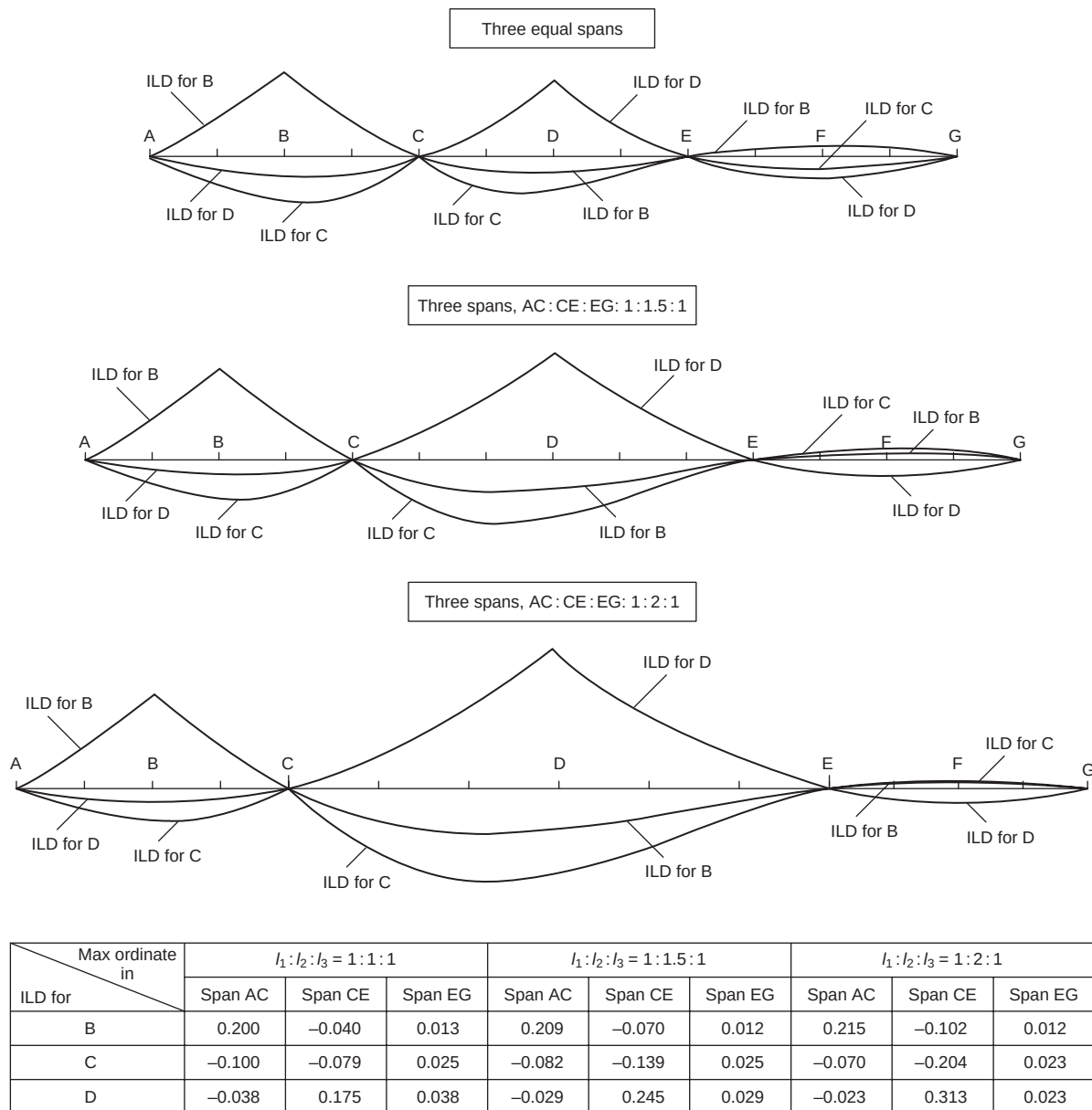


Figure 4.29 Influence lines for bending moment – three-span continuous beams (ILD, influence line diagram)

Plates and plated bridge structures

Bending of thin plates

Steel and steel–concrete composite plates are employed extensively in steel bridges. A *plate* is defined as a structural component having two of its dimensions significantly larger than the third (namely, its thickness). The structural analysis of a plate is carried out by considering the state of stress at the middle

plane of a plate. All the stress components are expressed in terms of the deflection w of the plate (where w is a function of the two coordinates (x, y) in the plane of the plate). This deflection function has to satisfy a linear partial differential equation, which, together with its boundary condition, completely defines w .

Figure 4.31 shows a plate element cut from a plate whose middle plane coincides with the x – y plane. The middle plane of the plate is subjected to a lateral load of intensity q . It can be shown, by considering the equilibrium of the plate element, that the stress resultants are given as

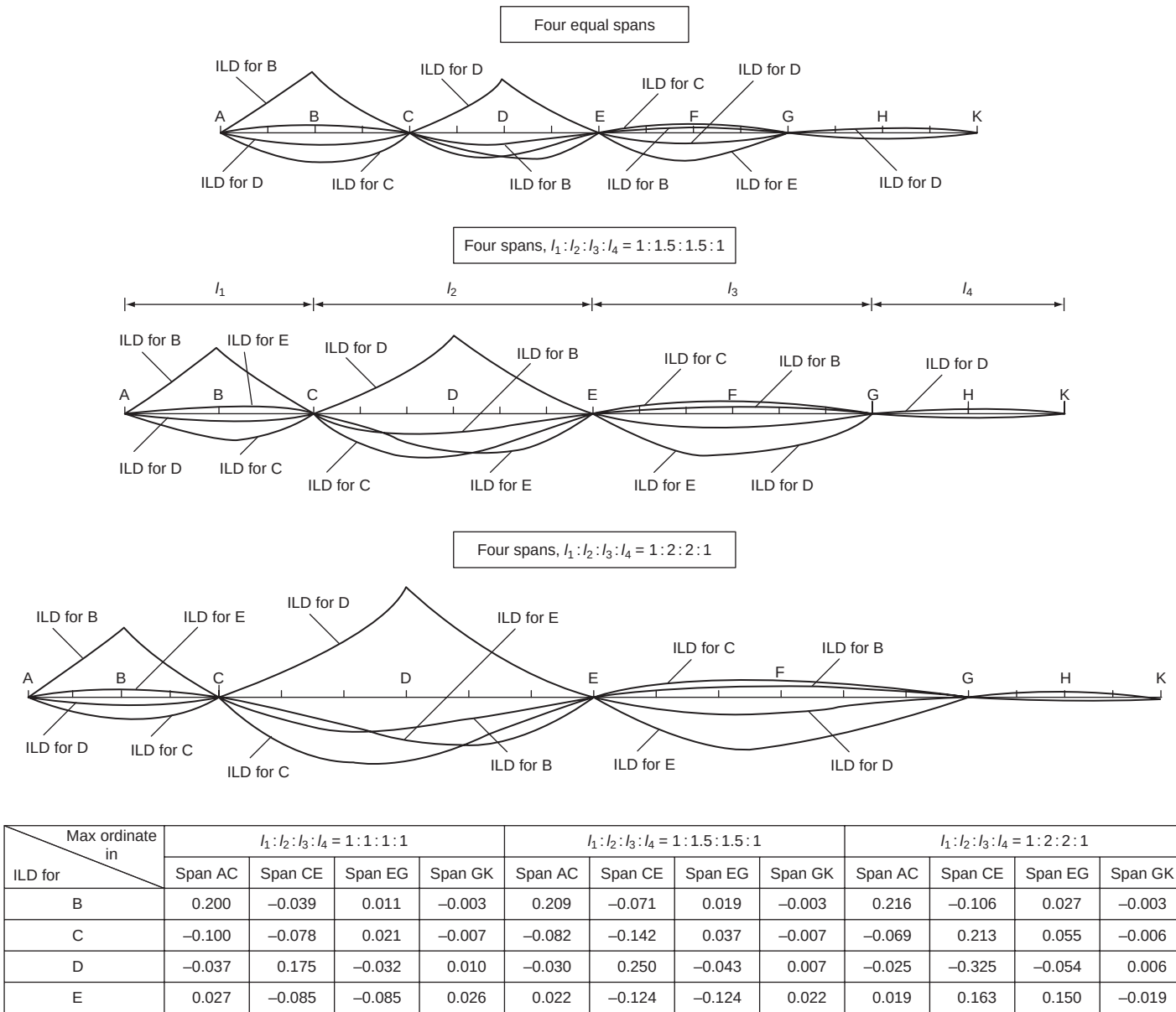


Figure 4.30 Influence lines for bending moment – four-span continuous beams (ILD, influence line diagram)

$$M_x = -D \left(\frac{\partial^2 w}{\partial x^2} + \nu \frac{\partial^2 w}{\partial y^2} \right) \quad (4.66)$$

$$M_y = -D \left(\frac{\partial^2 w}{\partial y^2} + \nu \frac{\partial^2 w}{\partial x^2} \right) \quad (4.67)$$

$$M_{xy} = -M_{yx} = D(1-\nu) \frac{\partial^2 w}{\partial x \partial y} \quad (4.68)$$

$$V_x = \frac{\partial^3 w}{\partial x^3} + (2-\nu) \frac{\partial^3 w}{\partial x \partial y^2} \quad (4.69)$$

$$V_y = \frac{\partial^3 w}{\partial y^3} + (2-\nu) \frac{\partial^3 w}{\partial y \partial x^2} \quad (4.70)$$

$$Q_x = -D \frac{\partial}{\partial x} \left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right) \quad (4.71)$$

$$Q_y = -D \frac{\partial}{\partial y} \left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right) \quad (4.72)$$

$$R = 2D(1-\nu) \frac{\partial^2 w}{\partial x \partial y} \quad (4.73)$$

where M_x and M_y are the bending moments per unit length in the x and y directions, respectively; M_{xy} and M_{yx} are the twisting moments per unit length; Q_x and Q_y are the shearing forces per unit length in the x and y directions, respectively; V_x and V_y are the supplementary shear forces in the x and y directions respectively, R is the corner force; D is the flexural rigidity of the plate per unit length; E is the modulus of elasticity; h is the thickness of the plate; ν is Poisson's ratio; and

$$D = \frac{Eh^3}{12(1-\nu^2)} \quad (4.74)$$

The governing equation for the plate is obtained as

$$\frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q}{D} \quad (4.75)$$

Any plate problem should satisfy this equation and the boundary conditions of the plate

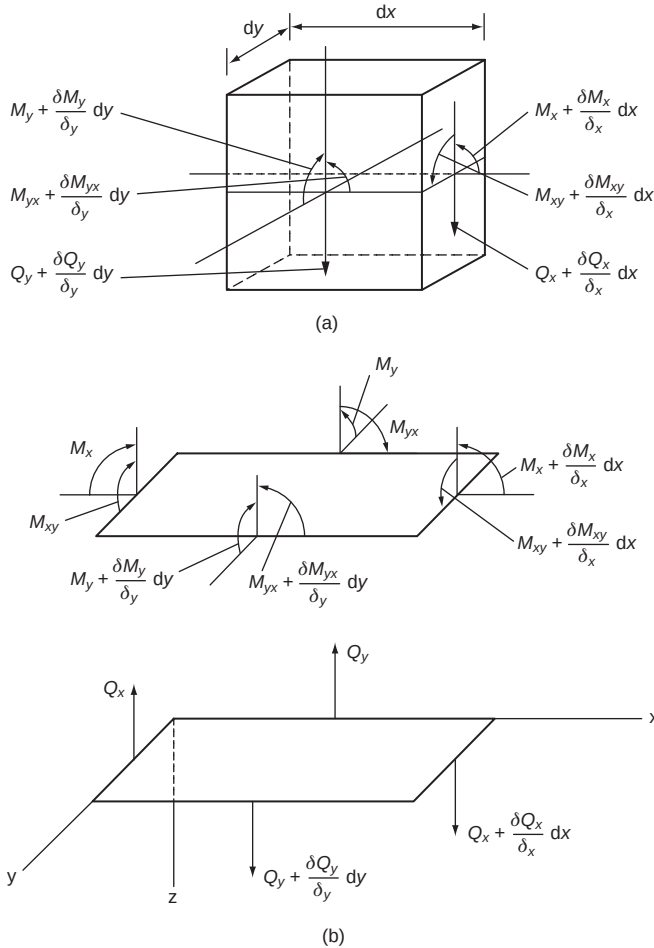


Figure 4.31 (a) Plate element; (b) stress resultants

Boundary conditions

There are three basic boundary conditions for plate problems. These are the clamped edge, simply supported edge and free edge.

Clamped edge

For this boundary condition, the edge is restrained such that the deflection and slope are zero along the edge. If the edge $x=a$ is clamped, then

$$(w)_{x=a}=0; \quad \left(\frac{\partial w}{\partial x}\right)_{x=a}=0 \quad (4.76)$$

Simply supported edge

If the edge $x=a$ of the plate is simply supported, the deflection w along this edge must be zero. At the same time, this edge can rotate freely with respect to the edge line. This means that

$$(w)_{x=a}=0; \quad \left(\frac{\partial^2 w}{\partial x^2}\right)_{x=a}=0 \quad (4.77)$$

Free edge

If the edge $x=a$ of the plate is entirely free, there are no bending and twisting moments or vertical shearing forces. This can be written in terms of w , the deflection, as

$$\left(\frac{\partial^2 w}{\partial x^2} + \nu \frac{\partial^2 w}{\partial y^2}\right)_{x=a}=0 \quad (4.78)$$

$$\left(\frac{\partial^3 w}{\partial x^3} + (2-\nu) \frac{\partial^3 w}{\partial x \partial y^2}\right)_{x=a}=0 \quad (4.79)$$

Bending of simply supported rectangular plates

A number of the plate-bending problems may be solved directly by solving the governing differential equation (Equation 4.75). The solution, however, depends on the loading and boundary conditions. Consider a simply supported plate subjected to sinusoidal loading (Figure 4.32) given by

$$q(x,y) = q_0 \sin \frac{\pi x}{a} \sin \frac{\pi y}{b} \quad (4.80)$$

The governing differential equation that satisfies the boundary conditions is

$$\frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q_0}{D} \sin \frac{\pi x}{a} \sin \frac{\pi y}{b} \quad (4.81)$$

The boundary conditions for the simply supported edges are

$$w = 0, \quad \frac{\partial^2 w}{\partial x^2} = 0 \quad \text{for } x = 0 \text{ and } x = a \quad (4.82)$$

$$w = 0, \quad \frac{\partial^2 w}{\partial y^2} = 0 \quad \text{for } y = 0 \text{ and } y = b \quad (4.83)$$

If the simply supported rectangular plate is subjected to any kind of loading given by

$$q = q(x,y) \quad (4.84)$$

the function $q(x,y)$ is represented in the form of a double trigonometric series in order to satisfy the boundary conditions:

$$q(x,y) = \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} q_{mn} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b} \quad (4.85)$$

in which q_{mn} is given by

$$q_{mn} = \frac{4}{ab} \int_0^a \int_0^b q(x,y) \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b} dx dy \quad (4.86)$$

This results in the expression for deflection:

$$w = \frac{1}{\pi^4 D} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \frac{q_{mn}}{\left(\frac{m^2}{a^2} + \frac{n^2}{b^2}\right)^2} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b} \quad (4.87)$$

If the applied load is uniformly distributed of intensity q_0 , then

$$q(x, y) = q_0 \quad (4.88)$$

and

$$q_{mn} = \frac{4q_0}{ab} \int_0^a \int_0^b \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b} dx dy = \frac{16q_0}{\pi^2 mn} \quad (4.89)$$

in which m and n are odd integers; $q_{mn} = 0$ if either or both of m and n is even. We can, therefore, write the expression for deflection of a simply supported plate subjected to uniformly distributed load as

$$w = \frac{16q_0}{\pi^6 D} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \frac{\sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}}{mn \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2} \quad (4.90)$$

where $m = 1, 3, 5, \dots$ and $n = 1, 3, 5, \dots$

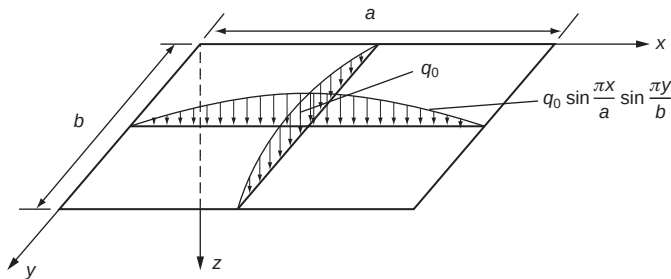


Figure 4.32 Rectangular plate under sinusoidal loading

The maximum deflection occurs at the centre and it can be written by substituting $x = a/2$ and $y = b/2$ in Equation 4.90 as

$$w_{\max} = \frac{16q_0}{\pi^6 D} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \frac{(-1)^{\frac{m+n}{2}} - 1}{mn \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2} \quad (4.91)$$

This equation is a rapidly converging series and a satisfactory approximation can be obtained by taking only the first term of the series; for example, in the case of a square plate:

$$w_{\max} = \frac{4q_0 a^4}{\pi^6 D} = 0.00416 \frac{q_0 a^4}{D} \quad (4.92)$$

Assuming $\nu = 0.3$, we get the maximum deflection as

$$w_{\max} = 0.0454 \frac{q_0 a^4}{Eh^3} \quad (4.93)$$

The expressions for bending and twisting moments can be obtained by substituting Equation 4.93 into Equations 4.66 to 4.73. Figure 4.33 shows some loading cases and the corresponding loading functions.

Solution for plates with arbitrary boundary conditions are complicated. It is possible to make some simplifying assumptions for plates with the same boundary conditions along two parallel edges in order to obtain the desired solution. Alternately, energy methods can be applied efficiently to solve problems for plates with complex boundary conditions. However, it should be noted that the accuracy of results depends on the deflection function chosen. These functions must be chosen so that they satisfy at least the kinematic boundary conditions.

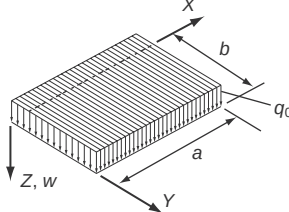
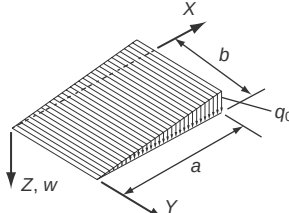
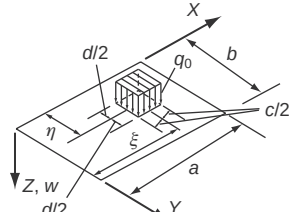
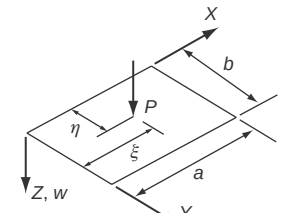
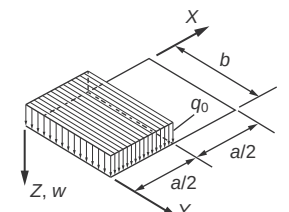
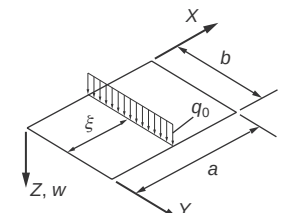
No.	Load	Expansion coefficients
	$q(x, y) = \sum m \sum n q_{mn} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}$	q_{mn}
1		$q_{mn} = \frac{16q_0}{\pi^2 mn}$ ($m, n = 1, 3, 5, \dots$)
2		$q_{mn} = \frac{-8q_0 \cos m\pi}{\pi^2 mn}$ ($m, n = 1, 3, 5, \dots$)
3		$q_{mn} = \frac{16q_0}{\pi^2 mn} \sin \frac{m\pi \xi}{a} \sin \frac{n\pi \eta}{b}$ $\times \sin \frac{m\pi c}{2a} \sin \frac{n\pi d}{2b}$ ($m, n = 1, 3, 5, \dots$)
4		$q_{mn} = \frac{4q_0}{ab} \sin \frac{m\pi \xi}{a} \sin \frac{n\pi \eta}{b}$ ($m, n = 1, 3, 5, \dots$)
5		$q_{mn} = \frac{8q_0}{\pi^2 mn}$ for $m, n = 1, 3, 5, \dots$ $q_{mn} = \frac{16q_0}{\pi^2 mn}$ for $\begin{cases} m = 2, 6, 10, \dots \\ n = 1, 3, 5, \dots \end{cases}$
6		$q_{mn} = \frac{4q_0}{\pi an} \sin \frac{m\pi \xi}{a}$ ($m, n = 1, 3, 5, \dots$)

Figure 4.33 Typical loading on plates and loading functions for rectangular plates

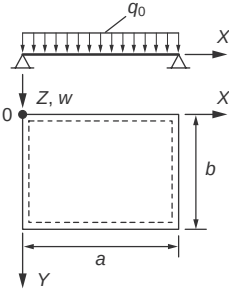
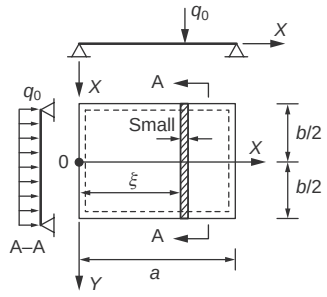
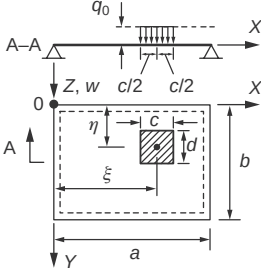
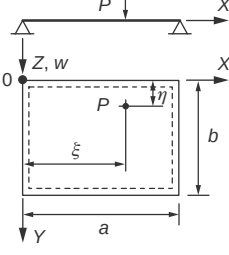
Case no	Structural system and static loading	Deflection and internal forces
1		$w = \frac{16q_0}{\pi^6 D} \sum_m \sum_n \frac{\sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}}{mn \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2}$ $m_x = \frac{16q_0 a^2}{\pi^4} \sum_m \sum_n \frac{\left(m^2 + \nu \frac{n^2}{\varepsilon^2} \right) \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}}{mn \left(m^2 + \frac{n^2}{\varepsilon^2} \right)^2}$ $m_y = \frac{16q_0 a^2}{\pi^4} \sum_m \sum_n \frac{\left(\frac{n^2}{\varepsilon^2} + \nu m^2 \right) \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}}{mn \left(m^2 + \frac{n^2}{\varepsilon^2} \right)^2}$ $\varepsilon = \frac{b}{a}, m = 1, 3, 5, \dots, \infty; n = 1, 3, 5, \dots, \infty$
2		$w = \frac{a^4}{D\pi^4} \sum_{m=1} \frac{P_m}{m^4} \left(1 - \frac{2 + \alpha_m \tanh \alpha_m}{2 \cosh \alpha_m} \cosh \lambda_m y + \frac{\lambda_m y \sinh \lambda_m y}{2 \cosh \alpha_m} \right) \sin \lambda_m x$ where $P_m = \frac{2q_0}{a} \sin \frac{m\pi \xi}{a} \quad \lambda_m = \frac{m\pi}{a} \quad \alpha_m = \frac{m\pi b}{2a}$ $m = 1, 2, 3, \dots$
3		$w = \frac{16q_0}{D\pi^6} \sum_m \sum_n \frac{\sin \frac{m\pi \xi}{a} \sin \frac{n\pi \eta}{b} \sin \frac{m\pi c}{a} \sin \frac{n\pi d}{2b}}{mn \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}$ $m = 1, 2, 3, \dots$ $n = 1, 2, 3, \dots$
4		$w = \frac{4P}{D\pi^4 ab} \sum_m \sum_n \frac{\sin \frac{m\pi \xi}{a} \sin \frac{n\pi \eta}{b} \sin \frac{m\pi x}{a} \sin \frac{n\pi y}{b}}{\left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2}$ $m = 1, 2, 3, \dots$ $n = 1, 2, 3, \dots$

Figure 4.34 Typical loading and boundary conditions for rectangular plates

Figure 4.34 gives formulae for deflection and bending moments of rectangular plates with typical boundary and load-conditions.

Strain energy of simple plates

The strain energy expression for a simple rectangular plate is

$$U = \frac{D}{2} \iint_{\text{area}} \left\{ \left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right)^2 - 2(1-\nu) \left[\frac{\partial^2 w}{\partial x^2} \frac{\partial^2 w}{\partial y^2} - \left(\frac{\partial^2 w}{\partial x \partial y} \right)^2 \right] \right\} dx dy \quad (4.94)$$

where

$$\left[\frac{\partial^2 w}{\partial x^2} \frac{\partial^2 w}{\partial y^2} - \left(\frac{\partial^2 w}{\partial x \partial y} \right)^2 \right] \quad (4.95)$$

is known as the Gaussian curvature.

A suitable deflection function $w(x,y)$ satisfying the boundary conditions of the given plate may be chosen. The strain energy, U , and the work done by the given load, $q(x,y)$

$$W = - \iint_{\text{area}} q(x,y) w(x,y) dx dy \quad (4.96)$$

can be calculated. The total potential energy is, therefore, given as $V = U + W$. Minimising the total potential energy, the plate problem can be solved.

If the function $w(x,y) = f(x)\phi(y)$ (that is, if it is the product of a function of x only and a function of y only) and $w=0$ at the boundary are assumed, then the integral of the Gaussian curvature over the entire plate equals zero. Under these conditions

$$U = \frac{D}{2} \iint_{\text{area}} \left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right)^2 dx dy \quad (4.97)$$

A detailed treatment of plate-bending problems can be found in Timoshenko and Krieger (1959).

Orthotropic plates

Plates of anisotropic materials have important applications, owing to their exceptionally high bending stiffness. A non-isotropic or anisotropic material displays direction-dependent properties. Simplest among them are those in which the material properties differ in two mutually perpendicular directions. A material so described, for example, wood, is orthotropic. A number of manufactured materials can be approximated as orthotropic. Examples include corrugated and rolled metal sheets, fillers in sandwich plate construction, plywood, fibre-reinforced composites, reinforced concrete and grid work. The latter consists of two systems of equally spaced parallel ribs (beams), mutually perpendicular, and attached rigidly at the points of intersection.

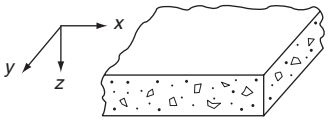
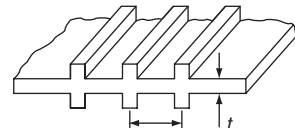
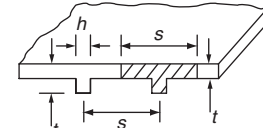
Geometry	Rigidities
Reinforced concrete slab with x and y directed reinforcement steel bars 	$D_s = \frac{E_c}{1-\nu_c^2} \left[I_{cx} + \left(\frac{E_s}{E_c} \right) I_{sx} \right] \quad D_r = \frac{E_c}{1-\nu_c^2} \left[I_{cy} + \left(\frac{E_s}{E_c} \right) I_{sy} \right]$ $G_{sy} = \frac{1-\nu_c}{2} D_x D_y \quad H = +D_x D_y \quad D_{xy} = \nu_c D_x D_y$ ν_c: Poisson's ratio for concrete. E_c, E_s: elastic modulus of concrete and steel, respectively. $I_{cx}, (I_{sx}), I_{cy}, (I_{sy})$: moment of inertia of the slab (steel bars) about neutral axis in the section $x = \text{constant}$ and $y = \text{constant}$, respectively.
Plate reinforced by equidistant stiffeners 	$D_s = H \frac{Et^3}{12(1-\nu^2)} \quad D_y = \frac{Et^3}{12(1-\nu^2)} + \frac{E'I}{s}$ E, E': elastic modulus of plating and stiffeners, respectively. ν: Poisson's ratio for plating. s: spacing between centre lines of stiffeners. I: moment of inertia of the stiffener cross-section with respect to midplane of plating.
Plate reinforced by a set of equidistant ribs 	$D_s = \frac{Est^3}{12[s-h+h(t/t_1)^2]} \quad D_y = \frac{EI}{s}$ $H = 2G'_{sy} + \frac{C}{s} \quad D_{sy} = 0$ C: torsional rigidity of one rib. I: moment of inertia about neutral axis of a T-section of width s (shown above). G'_{sy}: torsional rigidity of the plating. E: elastic modulus of the plating.

Figure 4.35 Various orthotropic plates

The governing equation for an orthotropic plate is similar to that of an isotropic plate and takes the form

$$D_x \frac{\delta^4 w}{\delta x^4} + 2H \frac{\delta^4 w}{\delta x^2 \delta y^2} + D_y \frac{\delta^4 w}{\delta y^4} = q \quad (4.98)$$

in which

$$D_x = \frac{h^3 E_x}{12}, \quad D_y = \frac{h^3 E_y}{12}, \quad H = D_{xy} + 2G_{xy}, \quad D_{xy} = \frac{h^3 E_{xy}}{12}, \quad G_{xy} = \frac{h^3 G}{12} \quad (4.99)$$

The expressions for D_x , D_y , D_{xy} and G_{xy} represent the flexural rigidities and the torsional rigidity of an orthotropic plate, respectively, and E_x , E_y and G are the orthotropic plate moduli. Practical considerations often lead to assumptions with regard to material properties, resulting in approximate expressions for elastic constants. The accuracy of these approximations is generally the most significant factor in the orthotropic plate problem. Approximate rigidities for some cases that are commonly encountered in practice are given in Figure 4.35.

General solution procedures applicable to the case of isotropic plates are equally applicable to the orthotropic plates. Deflections and stress resultants can thus be obtained for orthotropic plates of different shapes with different support and loading conditions. These problems have been researched extensively and solutions concerning plates of various shapes under different boundary and loading conditions can be found in Lee *et al.* (1971), Shanmugam *et al.* (1988, 1989a), Smith (1953), Timoshenko and Goodier (1970), Timoshenko and Krieger (1959) and Tsai and Cheron (1968).

Buckling of thin plates

Rectangular plates

Buckling of a plate involves out-of-plane movement of the plate and results in bending in two planes. A significant difference between axially compressed columns and plates is apparent if one compares their buckling characteristics. For a column, buckling terminates the ability of the member to resist axial loads, and the critical load is thus the failure load of the member. However, the same is not true for plates, owing to the membrane action of the plate. Subsequent to reaching the critical load, plates under compression will continue to resist increasing axial force, and will not fail until a load considerably in excess of the critical load is reached. The critical load of a plate is, therefore, not its failure load. Instead, one must determine the load-carrying capacity of a plate by considering its post-buckling behaviour.

To determine the critical in-plane loading of a plate by the concept of neutral equilibrium, a governing equation in terms of biaxial compressive forces N_x and N_y and constant shear force N_{xy} , as shown in Figure 4.36 can be derived, as

$$D \left(\frac{\delta^4 w}{\delta x^4} + 2 \frac{\delta^4 w}{\delta x^2 \delta y^2} + \frac{\delta^4 w}{\delta y^4} \right) + N_x \frac{\delta^2 w}{\delta x^2} + N_y \frac{\delta^2 w}{\delta y^2} + 2N_{xy} \frac{\delta^2 w}{\delta x \delta y} = 0 \quad (4.100)$$

The critical load for uniaxial compression can be determined from the differential equation:

$$D \left(\frac{\delta^4 w}{\delta x^4} + 2 \frac{\delta^4 w}{\delta x^2 \delta y^2} + \frac{\delta^4 w}{\delta y^4} \right) + N_x \frac{\delta^2 w}{\delta x^2} = 0 \quad (4.101)$$

which is obtained by setting $N_x = N_y = 0$ in Equation 4.100.

For example, in the case of a simply supported plate, Equation 4.100 can be solved to give

$$N_x = \frac{\pi^2 a^2 D}{m^2} \left(\frac{m^2}{a^2} + \frac{n^2}{b^2} \right)^2 \quad (4.102)$$

The critical value of N_x (i.e. the smallest value) can be obtained by taking $n = 1$. The physical meaning of this is that a plate buckles in such a way that there can be several half-waves in the direction of compression but only one half-wave in the perpendicular direction. Thus, the expression for the critical value of the compressive force becomes

$$(N_x)_{cr} = \frac{\pi^2 D}{a^2} \left(m + \frac{1}{m} \frac{a^2}{b^2} \right)^2 \quad (4.103)$$

The first factor in this expression represents the Euler load for a strip of unit width and length a . The second factor indicates in what proportion the stability of the continuous plate is greater than the stability of an isolated strip. The magnitude of this factor depends on the magnitude of the ratio a/b and also on the number m , which gives the number of half-waves into which the plate buckles. If a is smaller than b , the second term in the parenthesis of Equation 4.103 is always smaller than the first and the minimum value of the expression is obtained by taking $m = 1$, that is, by assuming that the plate buckles in one half-wave. The critical value of N_x can be expressed as

$$N_{cr} = \frac{k \pi^2 D}{b^2} \quad (4.104)$$

The factor k depends on the aspect ratio a/b of the plate and m , the number of half-waves into which the plate buckles in the x direction. The variation of k with a/b for different values of m can be plotted as shown in Figure 4.37. The critical value of N_x is the smallest value that is obtained for $m = 1$ and the corresponding value of k is equal to 4.0. This formula is analogous to Euler's formula for the buckling of a column.

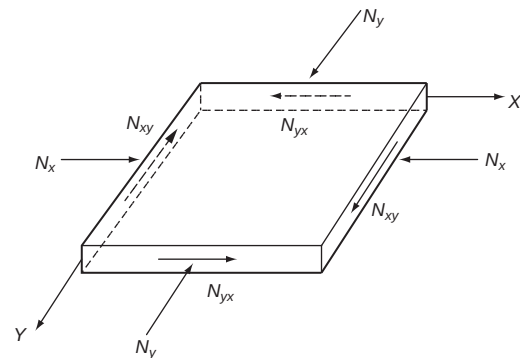


Figure 4.36 Plate subjected to in-plane forces

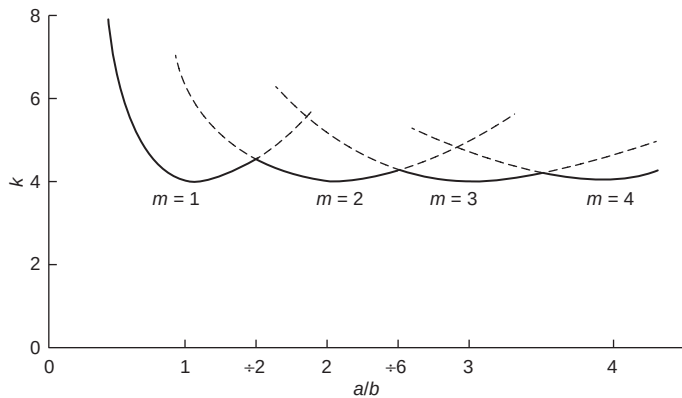


Figure 4.37 Buckling stress coefficients for uniaxially compressed plate

In the more general case in which normal forces N_x and N_y and the shearing forces N_{xy} act on the boundary of the plate, the same general method can be used. The critical stress for the case of a uniaxially compressed simply supported plate can be written as

$$\sigma_{cr} = 4 \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{h}{b}\right)^2 \quad (4.105)$$

The critical stress values for different loading and support conditions can be expressed in the form

$$f_{cr} = k \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{h}{b}\right)^2 \quad (4.106)$$

in which f_{cr} is the critical value of different loading cases. Values of k for plates with different boundary and loading conditions are given in Figure 4.38.

Detailed treatment of elastic buckling in plates may be found in Timoshenko and Gere (1961).

Buckling of plates in thin-plated structures

A major advantage of thin-plated steel sections is that their use leads to highly efficient and weight-effective members and structures. Figure 4.39 shows two cases of local buckling in typical plated structures. Case (b) is a plate girder in which the

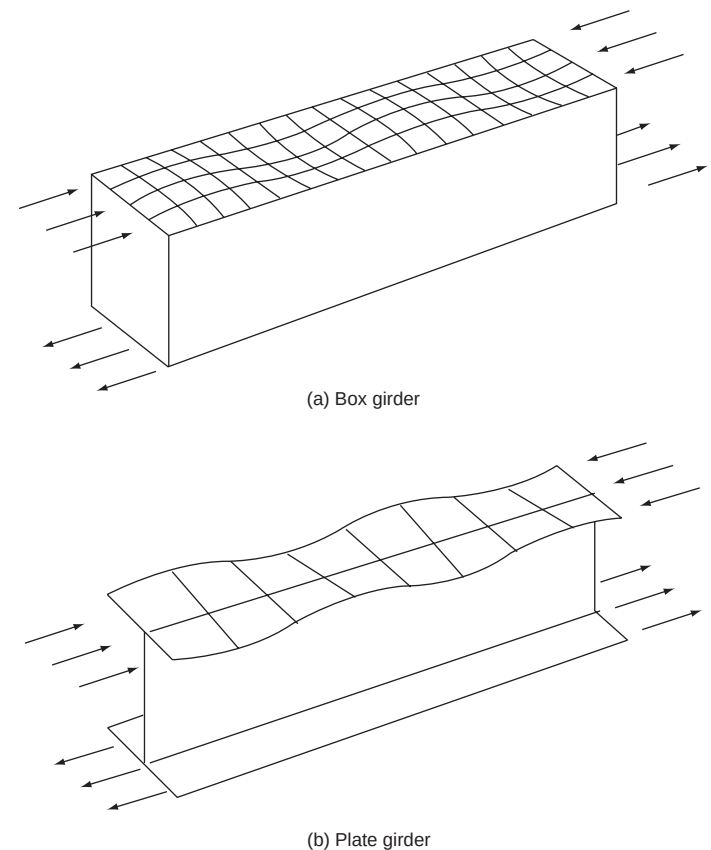


Figure 4.39 Buckling of flanges in box and plate girders

Case	Boundary and loading condition	Type of stress	Value of k for long plate
(a)		Compression	4.0
(b)		Compression	6.97
(c)		Compression	0.425
(d)		Compression	1.277
(e)		Compression	5.42
(f)		Shear	5.34
(g)		Shear	8.98
(h)		Bending	23.9
(i)		Bending	41.8

Figure 4.38 Values of k for plates with different boundary and loading conditions (s.s., simply supported)

top flange under compression is supported by the web along one edge and the flange is free to wave at its other longitudinal edge. In the case of a box girder top flange, the plate is supported by webs at both longitudinal edges, as shown in case (a).

In these two plates under uniform compression, the elastic critical stress can be expressed (Figure 4.40) as given in Equation 4.106, as

$$\sigma_{cr} = \frac{K\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2 \quad (4.107)$$

in which t is the thickness of the plate. The case when all four edges are simply supported approximates to the situation in the top flange of the box girder in Figure 4.39(a) and the value of K in Equation 4.107 is equal to 4; the mode of buckling will be as shown in Figure 4.41, depending on the aspect ratio (a/b) of the plate.

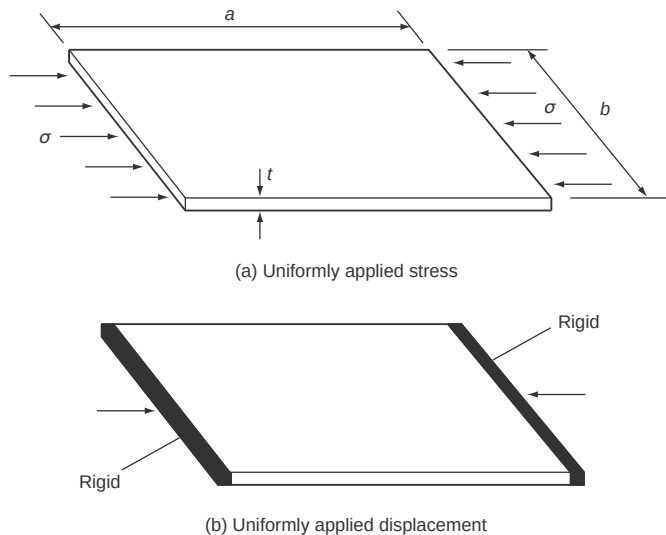


Figure 4.40 Axially loaded plate

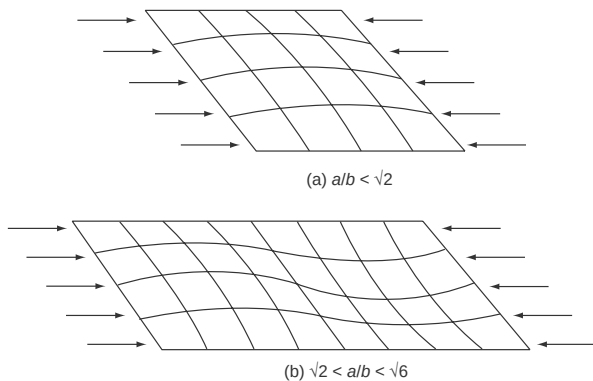


Figure 4.41 Buckled shapes in axially loaded plates

The value of K for a plate supported on three sides, as in the case of the flange plate in a plate girder, is 0.425 and the elastic critical stress corresponding to other types of in-plane loading can be computed by a formula similar to the one given in Equation 4.105 with different values for K . The post-buckling reserve strength of a plate can be envisaged in terms of its axial stiffness; for an ideally flat plate, the axial stiffness drops quickly to a much smaller value after buckling and remains relatively constant thereafter. Practical plates, however, exhibit a gradual loss of stiffness under in-plane loading, as shown in Figure 4.42.

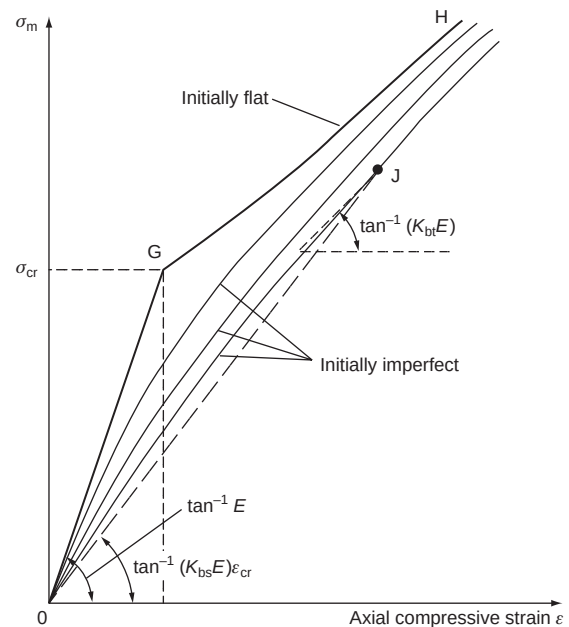


Figure 4.42 Axial stiffness of plates in compression

Numerical methods of analysis of plates, which include both geometric and material non-linearities, are now available; these analyses are capable of assessing the ultimate strength and post-collapse stiffness of plates with fabrication imperfections. However, most of the popular design methods are based on an 'effective width' approach, a concept originally proposed by Von Kármán *et al.* (1932) and subsequently improved by Winter (1947) to account for the effect of boundary conditions and plate imperfections. In accordance with this concept, the non-uniform stress distribution across the width of the buckled plate could be replaced by two uniform stress blocks of σ_{max} acting over two equal end strips of width $0.5b_{eff}$, where b_{eff} is the effective width, as shown in Figure 4.43.

The effective width may be obtained as

$$\frac{b_{eff}}{b} = \sqrt{\frac{\sigma_{cr}}{\sigma_{ys}}} \left[1 - 0.22 \sqrt{\frac{\sigma_{cr}}{\sigma_{ys}}} \right] \quad (4.108)$$

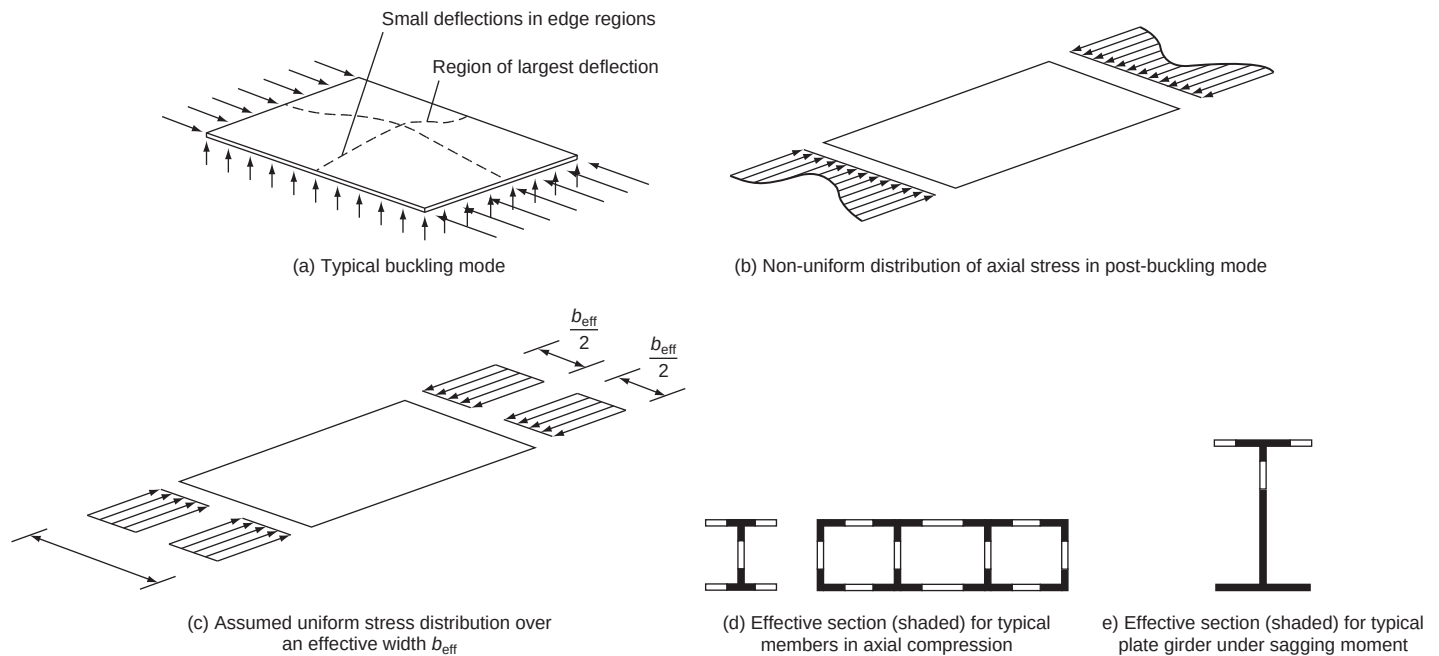


Figure 4.43 Effective width concept in axially compressed plates

A number of effective width formulae, to account for variations in boundary conditions, stress distribution and plate imperfection, have been proposed by various researchers (Faulkner, 1975; Horne and Narayanan, 1976a; Mulligan and Pekoz, 1984; Narayanan and Shanmugam, 1979a; Shanmugam *et al.*, 1989a).

In an analogous manner, the critical buckling stress of a rectangular panel loaded in shear (Figure 4.44) is given by:

$$\tau_{cr} = \frac{K\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{d}\right)^2 \quad (4.109)$$

As shown in Figure 4.44, a square element whose edges are oriented at 45° to the plate edges experiences tensile stresses on two opposing edges and compressive ones on the other two. These compressive stresses induce a form of local buckling; the mode involves elongated bulges oriented at about 45° to the plate edges. As with the compressive loading, a thin plate loaded in shear can support an applied stress well in excess of the elastic critical one. This is again due to the resistance to in-plane deformation. As the applied shear stress is increased beyond τ_{cr} , the plate buckles elastically and retains little stiffness in the direction in which the compressive component acts. However, the inclined tensile component is still resisted fully by the plate. The inclined buckles become progressively narrower and the plate acts like a series of bars in the tension direction, developing a so-called tension field. A further increase of applied stress causes plastic deformation in part of the tension field, which rotates to line up more closely with the plate diagonal. Tension field action is particularly important in plate and box girders, in which the function of the web plates is primarily to resist shear.

Stiffened compression flanges

Stiffened compression flanges of steel plate or box girders usually consist of a steel flange plate stiffened longitudinally by either open or closed stiffeners spanning between transverse stiffeners. Such compression flanges are subjected to the longitudinal stresses due to overall bending moment and axial force on the main girder; these stresses may vary across the width of the girder, owing to shear lag, and along the length, owing to the variation of bending moment along the span. When loaded in plane, the plate elements are extremely stiff. Even the relatively simple case of a stiffened panel consisting of a bed plate with equally spaced stiffeners must be analysed for a number

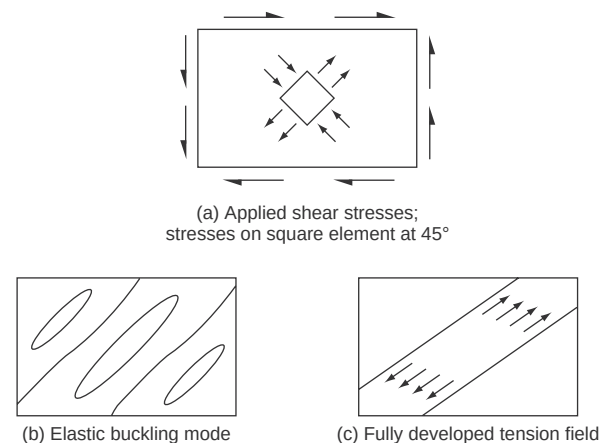


Figure 4.44 Web panels in shear; buckling and tension field action

of conditions (Chatterjee and Dowling, 1976; Faulkner, 1975; Horne and Narayanan, 1976a, 1976b). The ultimate capacity of such a stiffened plate is the smallest load causing either (a) overall buckling of the panel between cross-frames, (b) local buckling of the base plate between stiffeners or (c) local buckling of the stiffeners. A simple and approximate method proposed by Horne and Narayanan (1977) can be used to predict the ultimate capacity of a stiffened plate. The method is based on the effective width concept to account for local buckling in the base plate and the Perry–Robertson formula to account for the overall column buckling of the stiffened plate. The Perry–Robertson formula is given as

$$\frac{\sigma}{\sigma_{ys}} = \frac{1}{2} \left[1 + (1 + \eta) \frac{\sigma_e}{\sigma_{ys}} \right] - \sqrt{\frac{1}{4} \left[1 + (1 + \eta) \frac{\sigma_e}{\sigma_{ys}} \right]^2 - \frac{\sigma_e}{\sigma_{ys}}} \quad (4.110)$$

in which σ is the limiting applied axial stress on the effective strut section; σ_{ys} is the yield stress on the extreme compressive fibre; σ_e is the Euler stress of the effective strut; $\eta = \Delta y / r^2$; Δ is the maximum initial imperfection and eccentricity; y is the distance of the extreme compressive fibre and r is the radius of gyration of the effective section.

The ultimate capacity can, therefore, be estimated by aggregating the strength of the component stiffeners, along with the associated ‘effective’ plating, as shown in Figure 4.45. Loads causing local yield in base plates between stiffeners or in the stiffener outstands can be separately evaluated (Narayanan and Shanmugam, 1979b). A further consideration is the local torsional buckling of thin-walled open stiffeners; as the stiffener-induced collapse is catastrophic, it is usual to specify relatively stocky stiffeners, rather than to allow for such torsional buckling. The bridge design code BS 5400 (BSI, 2000) allows for initial imperfection and compressive welding residual stresses. Detailed methods to design compression flanges and the longitudinal stiffeners (flat, angle or T stiffeners) are included in the code.

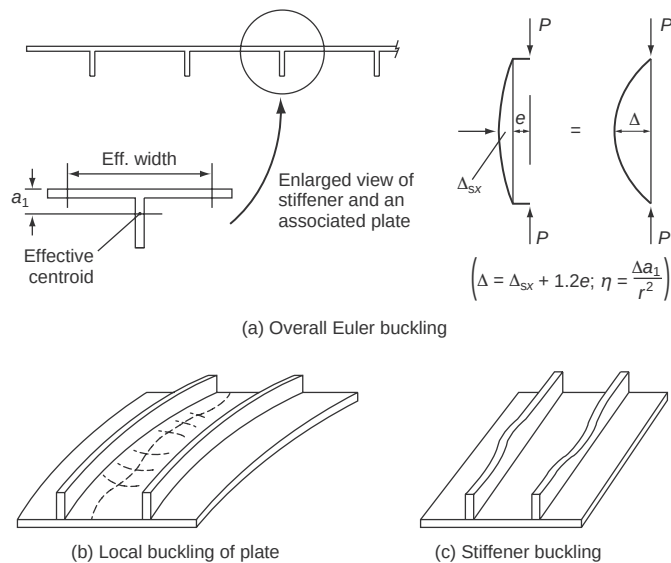


Figure 4.45 Ultimate load of a stiffened plate

The stiffened flange panel between the webs and cross-frames of the main girder can be assumed to behave as an orthotropic plate. This assumption is particularly beneficial in the case of stiffened flanges with stocky stiffeners where the buckling at the ultimate load is of a stable nature. In this case, the non-uniform distribution of flange stresses owing to shear lag or restrained warping may be neglected in the check for the ultimate state. Box girder compression flanges are normally subjected to longitudinal stress, owing to overall bending moment and axial force caused by vertical forces acting on the flange plate. The design procedures must provide for all the combinations of in-plane compressive stresses, in-plane shear stresses and in-plane transverse stresses in the flange plate.

Web plates

Webs in plate and box girders are normally reinforced by intermediate transverse stiffeners, and longitudinal stiffeners when the depth is large. Such reinforcement will significantly increase the ultimate load capacity of a girder, but the introduction of stiffeners will obviously increase fabrication cost and the self-weight of the girder. The ultimate strength of a stocky web is, of course, limited to its yield strength. An unstiffened girder with a slender web may be designed simply by neglecting any post-buckling reserve strength and assuming shear capacity of the web to be equal to the shear buckling resistance (V_{cr}), given by

$$V_{cr} = \tau_{cr} dt \quad (4.111)$$

where τ_{cr} is taken as the elastic critical shear buckling stress. Webs of intermediate slenderness may be designed with transverse stiffeners. The modern codes recommend the use of tension field action to determine the shear capacity in the case of slender webs (Chatterjee, 1980; Evans, 1983; Rockey *et al.*, 1977, 1978).

The behaviour of a plate or box girder subjected to an increasing shear load may be divided into three phases, as shown in Figure 4.46. Prior to buckling, equal tensile and compressive principal stresses are developed in the plate, as shown in Figure 4.46(a). In the post-buckling range, no more increases in compressive stress are possible and only an inclined tensile membrane stress field is developed, as shown in Figure 4.46(b). The magnitude of the tensile membrane stress is indicated by σ_t in Figure 4.46(b) and its inclination to the horizontal is shown as θ . Since the flanges of the girder are flexible, they will begin to bend inwards under the pull exerted by the tension field. A further increase in the load will result in yield occurring in the web under the combined effect of the membrane stress field and the shear stress at buckling. The value of σ_t at which yield occurs is identified as the basic tension field strength. Failure of the girder occurs when four plastic hinges form in the flanges of the girder, as shown in Figure 4.46(c). The resulting collapse mechanism then allows a shear displacement to occur as indicated. The ultimate shear capacity (V_b) can be obtained as

$$V_b = \tau_{cr} + y_b \sin^2 \theta \left(\cot \theta - \frac{b}{d} \right) dt + 4dt \sin \theta \sqrt{K_t p_{yw} y_b} \quad (4.112)$$

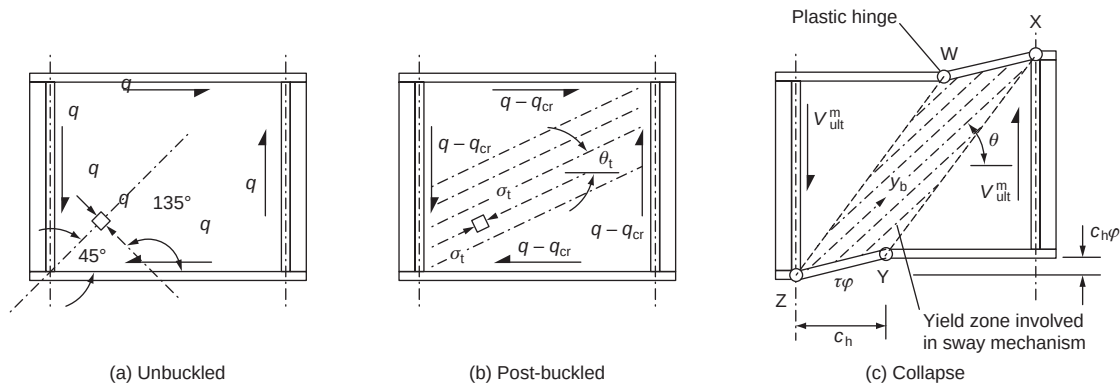


Figure 4.46 Buckling and post-buckling behaviour of web in shear

in which p_{yw} is the yield stress of web material, b is the width and d the depth of web panel and y_b is given by

$$y_b = [p_{yw}^2 - 3q_{cr}^2 + 2.25q_{cr}^2 \sin^2 2\theta]^{1/2} - 1.5q_{cr} \sin 2\theta \quad (4.113)$$

and $K_f = M_{pf}/(4M_{pw})$, M_{pf} and M_{pw} being the plastic moment capacity of a flange plate and a web plate, respectively.

The expression for the ultimate shear capacity is also written as

$$V_b = (q_b + q_f \sqrt{K_f}) dt \quad (4.114)$$

where

$$q_b = q_{cr} + \frac{y_b}{2\{a/d + \sqrt{1 + (a/d)^2}\}} \quad (4.115)$$

and

$$q_f = \left[4\sqrt{3} \sin\left(\frac{\theta}{2}\right) \sqrt{\frac{y_b}{p_{yw}}} \right] 0.6p_{yw} \quad (4.116)$$

In these equations, q_b represents the basic shear strength of the web panel; it combines the critical shear strength of the panel and the post-buckling strength derived from that part of

the web tension field supported by the transverse stiffeners. The term $q_f \sqrt{K_f}$ represents the contribution made by the flanges to the post-buckling strength. The flanges support part of the web tension field that pulls against them and finally develop plastic hinges at collapse. The relative importance of the various terms in Equation 4.114 is illustrated for a typical case in Figure 4.47.

The method must be used with caution in box girders in which the flange or web boundaries might have insufficient strength to develop adequate tension field capacity; moreover, owing to the bending of the box in the pre-buckling stage, the web panels will carry both longitudinal and shear stresses. The interaction between shear and in-plane compression (or tension) and bending stresses becomes important when examining the likely load combinations in the webs of box girder bridges, which are generally multiply stiffened. Based on an extensive parametric study, Harding (1983) has suggested interaction diagrams for the design of such plate panels:

$$\frac{\sigma_c}{S_c \sigma_{ys}} + \left[\frac{\sigma_b}{S_b \sigma_{ys}} \right]^2 + \left[\frac{\tau}{S_s \tau_{ys}} \right]^2 \leq 1 \quad (4.117)$$

where σ_c , σ_b and τ are the direct, bending and shear stresses and S_c , S_b and S_s are the knock-down factors that allow for the influence of b/t on the stability of plates.

Openings in webs

Openings in the webs of plate and box girders are necessary to facilitate inspection and for providing services. The introduction of openings in a web results in stress redistribution within the member and causes a reduction in its strength and stiffness. Approximate design methods are now available as a result of extensive research work on openings in plated structures. The first paper of relevance to thin perforated webs was that of Hoglund (1971), who reported on statically loaded plate girders containing circular and rectangular holes and subjected to transverse loading. The web plates of these girders were slender, having d/t values ranging from 200 to 300; they would, therefore, buckle before failure. In these experiments, the girders with holes that were in the high shear zone failed at

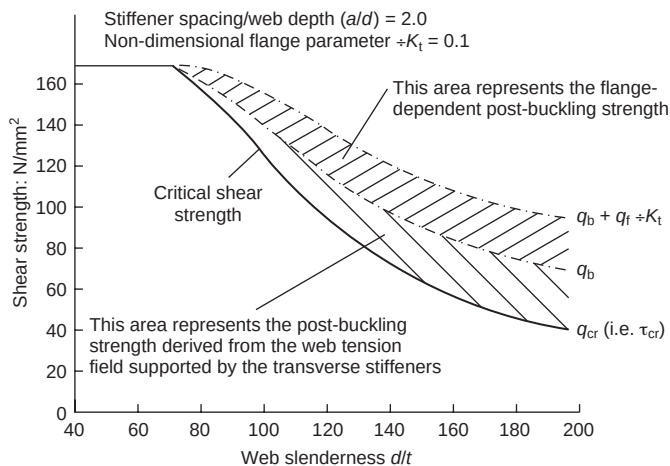


Figure 4.47 Variation of shear strength with web slenderness

significantly lower loads than those in the zone of high bending and low shear. These tests indicated the relative importance of shear failure criteria in plate girders with perforated webs. The tension field concept has been extended (Narayanan, 1983; Narayanan and Der Avenessian, 1983a, 1983b) to the cases of web plates containing centrally located circular and rectangular openings. Approximate equilibrium solutions have been proposed, based on further assumptions that the applied loading in the post-critical stage is carried by the membrane stresses developed along a diagonal band, the effective width of which is governed by the largest dimension of the hole, as shown in Figure 4.48. For central circular openings, the ultimate load can be obtained as

$$V_{ult} = (\tau_{cr})_{red} dt + \sigma_t^y t \sin^2 \theta [2c + d(\cot \theta - b/d)] - \sigma_t^y t D \sin \theta \quad (4.118)$$

where τ_{cr} is the reduced value of elastic critical shear stress of the hole-containing web. When the central opening is a rectangle of size $b_0 \times d_0$, the corresponding equilibrium equation is

$$V_{ult} = (\tau_{cr})_{red} dt + \sigma_t^y t \sin^2 \theta [2c + d(\cot \theta - b/d)] - \sigma_t^y t \sqrt{b_0^2 + d_0^2} \sin \theta \sin(\alpha + \theta) \quad (4.119)$$

where $\alpha = \arctan(d_0/b_0)$. The basis of the solution is the hypothetical tension band developed in the post-critical stage. If the loss of strength implicit in cutting a web hole is unacceptable, the web opening will need to be reinforced around its periphery, so that the opening can still be introduced. It would then be necessary to assess the strength of the reinforced web, with a view to examining its adequacy. Equilibrium solutions for the prediction of the ultimate capacity of girders containing a reinforced circular or rectangular hole have been proposed by Narayanan and Der Avenessian (1984). Methods to account for the presence of web openings in multi-cell box girders have also been proposed (Evans and Shanmugam, 1979a, 1981; Shanmugam and Evans, 1979).

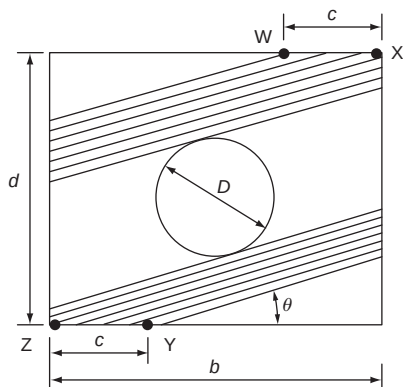


Figure 4.48 Opening in a plate girder web

Design of steel–concrete composite girders

In the last three decades, the importance of composite action between steel and concrete in composite plate girders has increased, owing to its advantage in terms of increased strength and stiffness. The full composite action between the steel and concrete can be achieved by providing shear connectors. If the concrete slab is simply laid over the steel girder, the girder may be designed as a regular steel plate girder. Composite design (steel and concrete) is simplified by assuming full interaction between steel and concrete; however, the mathematical form of the partial interaction theory is complex (Johnson, 2004). The imperfect interaction between the steel and concrete is possible, owing to the failure of the individual shear connectors (Oehlers *et al.*, 1997). When the steel section cannot accommodate the required numbers of shear connectors to achieve full interaction, partial interaction is the solution. Therefore, several research studies have been focused on the interaction between the connection between steel and concrete. However, this section discusses only the pioneer works on shear connections between steel and concrete.

The shear strength of composite plate girders was investigated by Baskar and Shanmugam (2003), Baskar *et al.* (2002) and Shanmugam and Baskar (2003). It was concluded that consideration of the effect of shear connectors will significantly improve the design shear strength of the plate girder. The shear strength of the composite slab is expressed by simply adding the shear strength of the concrete and steel girder. To account for the partial interaction, the following expression is introduced, assuming that the degree of interaction does not affect the shear strength of steel plate girder:

$$V_{cc} = V_a + \frac{N}{N_f} (V_f - V_a) \quad (4.120)$$

where V_{cc} is the shear resistance of the concrete slab in girders with partial interaction, V_a is the shear resistance of the concrete slab alone (which can be determined using Eurocode 2 (BSI, 2004)), V_f is the shear resistance of concrete slab for girders with full interaction, N is the actual number of shear connectors provided and N_f is the number of shear connectors required to achieve full shear connection (Johnson, 2004). It should be noted that V_f is taken to be equal to the pull-out capacity of shear connectors (Liang *et al.* 2004) of full shear connection, that is, $N/N_f = 1$.

A similar expression is available for determining the moment capacity of the steel–concrete composite plate girders:

$$M_{comp} = M_{plast} + \frac{N}{N_f} (M_{ult} - M_{plast}) \quad (4.121)$$

where M_{comp} is the moment capacity of the composite beam, M_{plast} is the moment capacity of the steel beam alone, M_{ult} is the moment capacity of the fully composite section, N is the number of shear connectors provided and N_f is the number of connectors required for full interaction.

Although the origin and format of the design of steel–concrete composite girders remain constant, new developments have been made in the existing equations for the design of composite plate girders with web perforations. A detailed discussion on the design and analysis of composite plate girders with web perforation is available in Yatim *et al.* (2013a, 2013b, 2015).

Design of steel–concrete composite girders with perforations

Service ducts are provided through the web of the steel beam by means of circular or rectangular openings. These openings significantly reduce the shear resistance of the beam and also induce local failure at the web, owing to shear. Therefore, shear transfer becomes an important design consideration in the design of beams with large service openings. The influence of large web openings, either in composite steel beam or bare steel, is complex. The following factors may influence the failure:

- opening shape (rectangular or square or circular)
- opening geometry (length, depth or diameter)
- location of large openings (moment zone or shear zone)
- load application (uniform or point load)
- web slenderness
- spacing between openings

However, the positioning of a large opening may minimise local failures and result in an economic design. There are many ways in which the effect of openings can be minimised.

- Locate the larger openings away from high shear zones.
- Provide web stiffeners may increase the bending resistance and it will prevent the local buckling of webs adjacent to the openings.
- Increase the thickness of the web.

The service perforations in the webs of the steel girders significantly reduce the strength and stiffness of the plate girders. However, the reduced strength can be compensated fully or partially by the composite girder effect (steel–concrete composite cross-section). The influence of perforation is elaborated earlier in this chapter. This section summarises the general behaviour and design guidelines for steel–concrete composite girders with perforations.

In general, for a composite beam with a larger opening, the failure mode and deformations of the perforated composite girder depend on the ratio of moment to shear, M/V , at the opening (Cho, 1982; Clawson and Darwin, 1980, 1982; Granade, 1968). The high M/V ratio at the openings will result in a flexural failure of the beam (yielding failure in steel and crushing failure in concrete) whereas the low M/V ratio leads to deformations in the openings. This deformation in the openings

can be differential or Vierendeel. The low M/V ratio will also result in transverse cracks in concrete. More detailed moment–shear interactions are discussed in Clawson and Darwin (1982). Although the effect of perforation on the overall deflection of the beam is small, a beam with a low M/V ratio will have a significant local deformation at the location of the perforation, leading to a transverse separation of the concrete deck.

At the initial stage, the following approximation may be used to determine the effect of an opening on the bending resistance of a composite beam:

$$M_{o,Rd} = M_{Rd}(1 - 0.35h_o/h) \quad \text{for unstiffened beams} \quad (4.122)$$

$$M_{o,Rd} = M_{Rd}(1 - 0.2h_o/h) \quad \text{for horizontally stiffened beams} \quad (4.123)$$

where M_{Rd} is the bending resistance of the unperforated composite beam, $M_{o,Rd}$ is the bending resistance of the composite beam at the opening, h_o is the depth of the opening and h is the depth of the steel beam.

These expressions are only simple approximations, but there are various cases involved in the design of composite beams with perforations. The following are various design cases: (i) steel beams with stiffened openings and unstiffened openings; (ii) different shapes of opening (rectangular, circular and elongated circular openings); (iii) varying positions of the neutral axis (in the top T of the web or in the concrete slab); (iv) shear connection at the opening; (v) the provision of a web post; (vi) all geometric configurations and parameters of the beams and openings. In addition, the provision of perforations can sometimes be asymmetrical or at the end of the beam (coped beam – close to the column connection) and this should be considered in the design. Detailed descriptions of the design of composite beams with perforations are available in Narayanan (2014) and SCI (2011).

The ultimate shear capacity of composite plate girders (V_{ult}) may be obtained as a sum of the shear capacity of a bare steel plate girder (V_s) and the contribution by the concrete slab to the shear capacity (V_c):

$$V_{ult} = V_s + V_c \quad (4.124)$$

The shear strength of the steel part of the girder may be obtained as described earlier and that of the concrete slab can be determined separately.

The composite action between the steel plate girder and the reinforced concrete slab depends on the connection between the two structural elements. Such connections are normally provided by shear connectors welded onto the plate girder flange. In the case of composite beams without shear connectors, with the concrete slab simply lying over the steel beam, each of the components acts independently, with relative slip occurring at the interface. If the components are connected, thus eliminating the slip, the action of horizontal shears on the interface of the slab and beam provide composite action. The tensile force transferred from the plate girder to the concrete slab by means of these shear connectors causes shear failure of the concrete in between two successive shear connectors.

Shear failure in the concrete slab is generally influenced by the arrangement, spacing and size of the shear connectors, the concrete strength and other properties of the composite member. In view of variations in these parameters, there is no unique way to define the behaviour of composite girders in this region and hence design for shear. Owing to composite action in the girder and the resulting load transfer between steel girder and concrete slab in the region of tension field action, there is a severe discontinuity in the overall behaviour of the girder. In view of the layout of the shear connectors in the slab, the slab shear strength must be determined differently. By considering two different regions along the span length of the girder, the shear strength of the slab part can be determined. Details of the computation may be found in Darehshouri *et al.* (2012, 2013).

Corrugations in webs

Previous sections in this chapter explored the structural behaviour of plain web plates in plate girders. The drawback of the plain web plate is the reduced shear strength when it is slender, owing to the decrease in elastic shear buckling capacity. However, the slender webs of the deep beams can be made more effective by introducing corrugations (internal stiffeners) in the web plates, thereby enhancing the shear capacity of the beam to reach post-buckling strength. In addition, the corrugations in the webs eliminate the need for transverse web stiffeners and increase the lateral stability of the plate girder, that is, the stability failure will not occur before the web reaches its ultimate capacity. Typically, the industry recommends a thickness less than 3 mm, owing to manufacturing limitations. This is because, when the thickness of the plate is less than or equal to 3 mm, the corrugation can be made by the cold-rolling process, which simplifies the fabrication process. With an increase in plate thickness, the possibility of cracks occurring at the corner increases in contrast to thin plates, enabling more flexibility in fabricating the desired shape. Although infinite corrugation shapes are possible with the cold-rolling process, the sinusoidal corrugation has higher local buckling capacity than the trapezoidal corrugation or other corrugations with flat plate portions, owing to the smaller half-wave buckling lengths.

The pioneer work on beams with the corrugated web was conducted by Peterson and Cord (1960) for the application of aircraft design. Later, the concepts of the corrugated web were incorporated in building design by Hamada *et al.* (1984). A review summary of the concept and development of structural members with corrugated webs is available in Elgaaly and Dagher (1990). In general, corrugated webs act as a series of flat plates that mutually support each other (Elgaaly *et al.*, 1992). Further updates on the behaviour and design of the beams with corrugated webs are presented in Elgaaly *et al.* (1996, 1997). Studies of the behaviour of corrugated webs subjected to in-plane loading were also made by Abbas *et al.* (2006) and Dubina *et al.* (2013, 2015). Prestressed beams with corrugated webs were analysed by Kim *et al.* (2011).

The design recommendations of the Eurocode for considering the effect of web corrugation are summarised next.

The ultimate shear capacity of the corrugated webs (all sides simply supported) shall be obtained as

$$V_{br} = x_c \frac{f_{yw}}{\gamma_m \sqrt{3}} h_w t_w \quad (4.125)$$

where x_c is the minimum of reduction factors for local buckling (x_{cl}) and global buckling (x_{cg}). The values of x_{cl} and x_{cg} can be obtained from Equations 4.126 and 4.130, respectively, as follows.

The reduction factor x_{cl} for local buckling shall be taken as

$$x_{cl} = \frac{1.15}{0.9 + \lambda_{cl}} \leq 1.0 \quad (4.126)$$

$$\lambda_{cl} = \sqrt{\frac{f_{yw}}{\tau_{cr1} \sqrt{3}}} \quad (4.127)$$

$$\tau_{cr1} = 4.83 E \left[\frac{t_w}{a_{max}} \right]^2 \quad (4.128)$$

where a_{max} shall be taken as the largest flat width of the plate (a_1 or a_2).

The value of τ_{cr1} for the sinusoidal corrugated plates shall be calculated as

$$\tau_{cr1} = \left(5.34 + \frac{a_1 s}{h_w t_w} \right) \frac{\pi^2 E}{12 (1 - \nu^2)} \left[\frac{t_w}{s} \right]^2 \quad (4.129)$$

where s is the unfolded length of the half-wave.

The reduction factor x_{cg} for global buckling shall be taken as

$$x_{cg} = \frac{1.5}{0.5 + \lambda_{cg}^2} \leq 1.0 \quad (4.130)$$

$$\lambda_{cg} = \sqrt{\frac{f_{yw}}{\tau_{crg} \sqrt{3}}} \quad (4.131)$$

$$\tau_{crg} = \frac{32.4}{t_w h_w} \sqrt{D_x D_z^3} \quad (4.132)$$

$$D_x = \frac{E t_w^3}{12 (1 - \nu^2)} \frac{w}{s} \quad (4.133)$$

$$D_z = \frac{E I_z}{w} \quad (4.134)$$

where I_z is the second moment of area of one corrugation of length w .

The ultimate moment capacity of the beam with corrugated webs (all sides simply supported) shall be obtained as

$$M_y = \min \left\{ \frac{b_1 t_1 f_{yf,r}}{\gamma_{mo}} \left[h_w + \frac{t_1 + t_2}{2} \right]; \frac{b_1 t_1 f_{yf,r}}{\gamma_{mo}} \left[h_w + \frac{t_1 + t_2}{2} \right]; \frac{b_1 t_1 x f_{yf}}{\gamma_{m1}} \left[h_w + \frac{t_1 + t_2}{2} \right]; \right\} \quad (4.135)$$

where $f_{yf,r}$ is the value of yield stress reduced due to transverse moments in the flanges:

$$f_{yf,r} = f_{yf} f_t \quad (4.136)$$

$$f_t = 1 - 0.4 \sqrt{\frac{\sigma_x(M_z)}{f_{yc}}} \quad (4.137)$$

and $\sigma_x(M_z)$ is the stress due to transverse moment in the flange and x is the reduction factor for out-of-plane buckling, according to Section 6.3 of EN 1993-1-1 (BSI, 2005).

Patch loading

The collapse or crippling of plate or box girders subjected to patch loading and distributed edge loading is a complex problem involving both material and geometric non-linearity.

Solutions are now available for the elastic critical loads of idealised web panels subjected to a variety of combined loading but such solutions show little or no correlation with experimental collapse loads. The localised stress distribution owing to patch loading will, in general, be combined with global bending and shear stresses. Whether it is the local or global stress distribution that is predominant will depend on the overall structural form and loading.

Although the problem of patch loading may seem to be of only minor significance in the overall design, it is, however, important to ensure that girders are not overstressed locally (e.g. during launching operations), since localised failure may precipitate overall structural failure. A large number of model tests in the past have shown that failure of slender plate girder subjected to patch loading occurs owing to the formation of plastic hinges in the flange accompanied by yield lines in the web. This type of failure, which is very localised, has become known as web crippling. Simple approximate solutions based on mechanism solutions have been proposed by Roberts (1983) for predicting the collapse or crippling load of plate girders subjected to patch loading, and these have been adopted in codes of practice.

Plate and box girder analysis

Introduction

The high bending moments and shearing forces associated with the carrying of large loads over long spans, as in the case of bridges, frequently necessitates the use of fabricated plate and box girders. The high torsional strength of box girders makes them ideal for girders curved in plan. In their simplest form, plate and box girders can be considered as an assemblage of webs and flanges, as shown in Figure 4.49. To reduce the self-weight of these girders and thus achieve economy, slender plate sections (having large lateral dimensions compared with their thicknesses) are employed. Hence, local buckling and post-buckling reserve strengths of plates are important design criteria (see the section on bracing requirements for bridge girders). Flanges in a box girder and webs in plate and box girders are often reinforced with stiffeners to allow for efficient use of thin plates. The designer must find a combination of plate thickness and stiffener spacing that will result in the most optimal section with reduced weight and fabrication cost.

Limit states design codes have placed greater emphasis on the development of new approaches based on the ultimate strength of plate and box girders and their components. Significant advances have been made in understanding the behaviour of steel-plated structures as a result of continuing research during the last 25 years. The initial stimulus for this interest in steel-plated structures was provided by the collapse of four steel box girder bridges during erection in the early 1970s. The CIRIA research guide *Structural Actions in Box Girders* (Horne, 1975) lists the following difficulties that are usually encountered by the designers of plated structures.

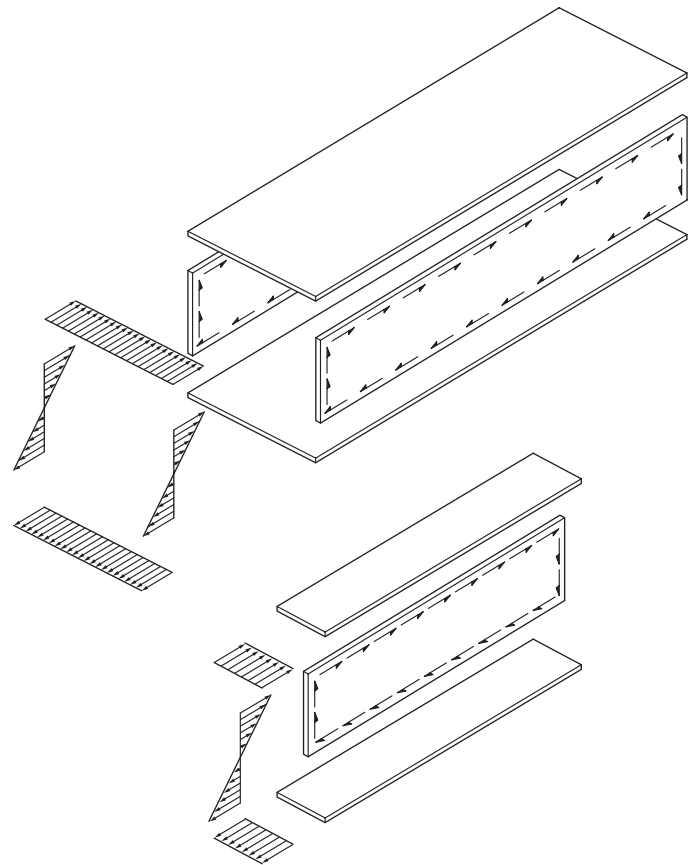


Figure 4.49 Box girder and plate girder as assemblages of plate elements

- The engineer's simple 'plane sections remain plane' theory of bending is no longer adequate, even for linear elastic analysis.
- Non-linear elastic behaviour caused by the buckling of plates can be of great importance and must be allowed for.
- Because of this complex non-linear elastic behaviour, and also because of stress concentration problems, some yielding may occur at loads that are quite low in relation to ultimate collapse loads. While such yielding may not be of great significance as regards either rigidity or strength, it means that simple maximum stress criteria are not sufficient.
- Because of the buckling problem in plates and stiffened panels, complete 'plastification' is far from being realised at collapse. Hence, simple plastic criteria are not sufficient either.
- Complex interactions occur between flanges, webs and diaphragms and the pattern of this interaction can change as the level of load increases.

Linear elastic analysis

In the case of plated beams, such as the one shown in Figure 4.50, the vertical (Figure 4.50(a)) and horizontal (Figure 4.50(d)) components of applied loads produce elastic bending stresses (Figures 4.50(b) and (e)) and shear stresses (Figures 4.50(c) and (f)) when they act through the shear centre. These

forces produce torque on the cross-section if they act eccentrically with respect to the shear centre. Box sections are very strong compared with I-sections in resisting torsion. The torque T in box sections could be resisted entirely by a 'shear flow' q acting around the box; the shear stress in any part of the wall of the box would be equal to q/t where t is the thickness of the plate forming the wall. The shear flow q is obtained as

$$q = \frac{T}{2A_B} \quad (4.138)$$

where A_B is the area enclosed by the box.

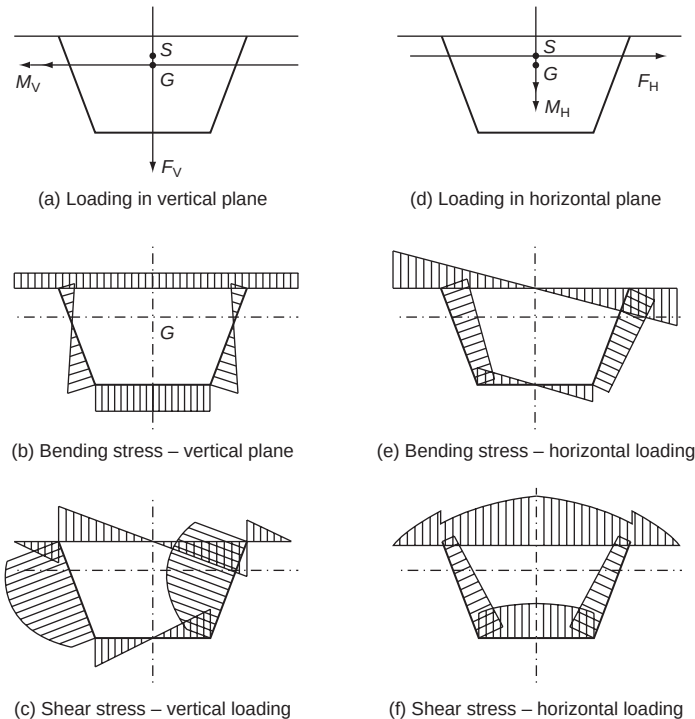


Figure 4.50 Bending of box girders

The angle of twist of the box per unit length, owing to the shear flow is given by

$$\theta = \frac{1}{2A_B G} \oint \tau ds \quad (4.139)$$

or

$$\theta = \frac{T}{4A_B^2 G} \oint \frac{ds}{t} \quad (4.140)$$

where G is the elastic shear modulus.

In the case of open sections, such as a plate girder, the angle of twist per unit length is expressed as

$$\theta = \frac{T}{GJ} \quad (4.141)$$

where

$$J = \frac{st^3}{3} \quad \text{or} \quad J = \frac{1}{3} \int_{\text{section}} t^3 ds \quad (4.142)$$

where J is the torsional constant of the section, s the length along the centre line of rectangular element, namely the web or a flange, and t is the thickness of the element.

The assumption of simple bending theory is reasonably accurate if the span-to-depth ratio of the plate or box girders exceeds about 4. However, because of differential bending in vertical planes, the cross-sections are subjected to warping. Another aspect of behaviour not allowed for in the simple treatment of torsion is the possible distortion of the cross-section. Distortion introduces additional stresses of various types, which have to be allowed for. However, distortion effects can be controlled by the rigidity and spacing of cross-frames or diaphragms. In the case of sections in which the width between webs is very large and nearly equal to the span, questions arise about the effectiveness of the complete width of the flange. There obviously has to be some limitation on the 'effective width' of flanges of higher width/span ratios. These limitations are due to the intrusion of shear lag.

Shear lag effects

Box girder structures in bridges are subjected to shear and bending so that the normal stress distribution in a cross-section must take account of the shear lag phenomenon, which influences both tension and compression. Because of the action of the large in-plane shear strain in the flanges, the longitudinal strains in the central areas are less than in those areas adjacent to the web-flange junction, and the vertical deflections of the central areas are less than the deflections at the web-flange junction. The result is that the distribution of compressive stresses across the flange is not uniform during the early stages of loading, and the effect of this is to reduce the elastic stiffness of the girder in bending. This first-order phenomenon induces a non-uniform normal stress distribution across the breadth of the flanges, the stresses being greatest along the web-flange junction because of compatibility requirements. The non-uniform stress distribution across the flange width is illustrated in Figure 4.51.

The procedure commonly adopted in design is to replace the actual width of the flange by an effective breadth, which, when used in conjunction with simple beam theory, leads to a correct estimate of the maximum flange stresses or beam deflections as required. The effective breadth factor for a stiffened flange depends on the geometry of the box, the ratio of stiffener area to plate area (A_s/bt_f), the support conditions and the loading distribution. Moffat and Dowling (1975) and Shanmugam and Evans (1981) have provided, based on extensive studies using the finite-element approach, a comprehensive picture of the shear lag effect on the behaviour of box girders. A detailed parametric study is very helpful to designers for computing the effective flange breadths at all positions on the span of a box girder of given plan dimensions and cross-sectional proportions, subjected to point or distributed loading. The variation of effective breadth factor ψ with aspect ratio B/L for the effective breadth at midspan, owing to a point load at midspan and to a uniformly distributed load is shown in Figure 4.52 for two values of A_s/bt_f (0 and 1.0).

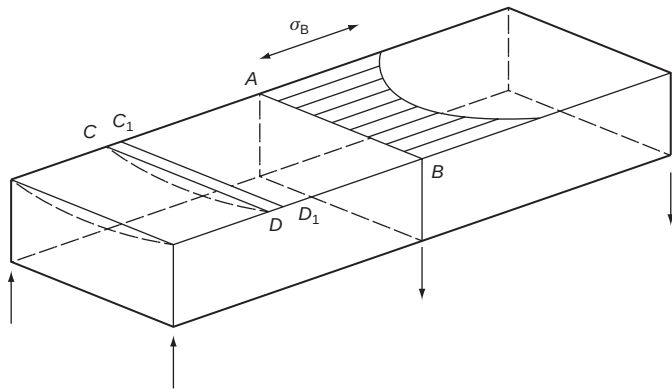


Figure 4.51 Shear lag in a box girder

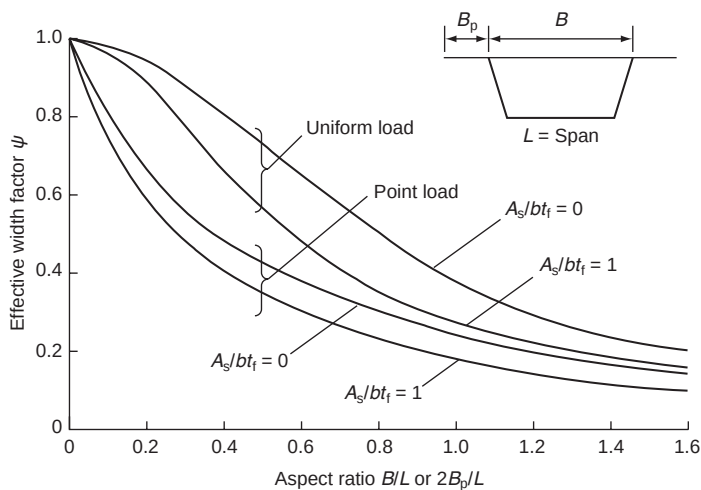


Figure 4.52 Effective breadth factors for shear lag at midspan of simply supported box

The most significant effect of shear lag on the behaviour of the point-loaded girders is to reduce the overall stiffness. If the girder continues to be loaded after yielding has occurred near the flange–web interface, the yielding will spread across the flange, giving a more uniform stress distribution. Test results have shown that a full redistribution of stress across the flange will have taken place as the girder approaches its ultimate strength and the neglect of shear lag effect has no significant weakening effect on the ultimate strength of the stiffened compression flanges.

Torsion

Vertical loads that act eccentrically with respect to the centre line in a box girder result in twisting of the box section. The twisting moment is resisted by pure shear stresses in the walls of a box, which is known to possess very high torsional resistance. Longitudinal normal stresses arising from the relative warping of the section under torsion are not considered in the theory of pure torsion; however, these stresses can attain

very large values when the closed cross-sections are flexible. For example, consider a general loading on a box section, as shown in Figure 4.53, in which a single vertical eccentric load is replaced by sets of forces representing vertical, torsional and distortional loading. The general loading in Figure 4.53 can be represented as two different components of loading, one causing bending and the other causing torsion, as shown in Figures 4.53(b) and (c). The torsional loading component can be subdivided further into a pure torsional component and a distortional component, as shown in Figures 4.53(d) and (e). Although the pure torsional component will normally result in negligible longitudinal stresses, the distortional component will always tend to deform the cross-section, leading to distortional stresses in the transverse direction and warping stresses in the longitudinal direction. The distortion of the cross-section will be resisted by cross-frames and diaphragms; hence, an accurate analysis involves evaluating the distortional warping and shear stresses and the associated distortional bending stresses in the transverse frames. The problem of torsion in box girders can be analysed more accurately by employing the well-known beam-on-elastic foundation analogy.

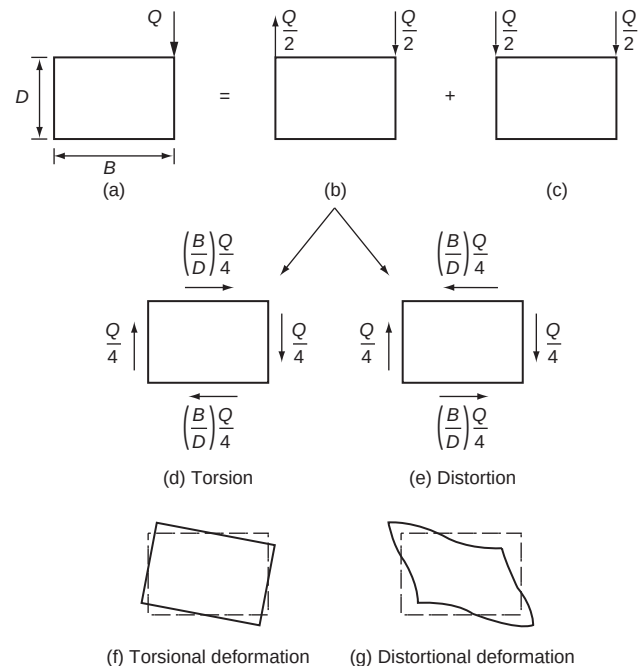


Figure 4.53 Idealisation of eccentric loading in box girder

Bracing requirements for bridge girders

The purpose of this section is to provide an idea of the behaviour and design of bracing systems for beams. The concept of beam bracing originated from the column bracing expressions developed by Winter (1960). In general, the concept of bracing design is simply to meet the stability requirement by the bracing member. The objective of installing a bracing system

in beams is to achieve the full capacity (yield moment capacity) of beams (Selvaraj and Madhavan, 2018, 2019a, 2019b). It was shown by Winter (1960) that both the stiffness and strength of the member is important. If the bracing design rules are based only on either strength or stiffness, the bracing system will be inadequate (Yura, 2001). Therefore, it is necessary to check both the stiffness and strength requirements of the beam bracing systems (Wang and Nethercot, 1990).

The beam bracing arrangements and design are complex and require detailing, compared with the column bracings (Selvaraj and Madhavan, 2019c, 2019d; Yura, 2001). The design of beam bracing should be based on the failure mode of the beam; the failure of the beam combines both torsion (cross section twist) and flexure (vertical deflection) (Selvaraj and Madhavan, 2019d). The bracing system of beams can be classified as lateral bracing and torsional bracing.

An adequate lateral bracing inhibits lateral deflection of the beam; however, the effectiveness of the lateral bracing depends on the location of the centre of twist of the beam and that of the lateral bracing. In addition, an effective beam bracing depends on the support condition of the beam. An additional advantage of effective beam bracing is the elimination of transverse stiffeners in the beam.

Torsional bracing, as the name implies, provides restraint against cross-sectional twist of the beam about its longitudinal axis. An example of torsional bracing might be a concrete slab on a composite steel beam. However, the concrete beam arrests both the lateral displacement and cross-sectional twist of the beam. The bracing that inhibits both displacements is much more effective and preferred in the design. Although the single bracing system can be sufficient for restraining the beam from lateral torsional buckling, the bracing requirement for arresting lateral translation and twist shall be determined individually, as it is more practical and conservative. Therefore, the bracing requirements are discussed separately.

Lateral bracing of beams

Effectiveness of lateral bracing

As mentioned previously, the effectiveness of beam bracing depends on various factors, such as the location of the beam bracing, beam support conditions and loading pattern. The behaviour of the beam cum bracing with these interdependent factors is described as follows. For a beam subjected to a uniform moment, the lateral bracing at the top flange will be much more effective than the lateral bracing at the centroid of the beam. This is because the centre of twist of the simply supported beam lies well below the tension flange; therefore, the bracing away from the centre of the twist will experience a smaller force. Now consider the beam with a concentric load at the midspan of the beam. In this case, the beam will not have a uniform moment along the length of the beam; however, owing to the load concentration, the ultimate moment at the midspan of the beam is significantly greater than in the uniform moment case. As the moment increases, the strength and stiffness of the

bracing requirement increases significantly compared with the uniform moment condition. It should be noted that all these cases are for the load applied through the centre of gravity of the specimen.

The next interdependent factor is the load applied through the top flange of the beam. In this case, the ultimate capacity of the beam significantly decreases, even with top flange bracing. The significant decrease in the effectiveness of the bracing is due to the change in the centre of twist of the beam owing to the top flange loading. The centre of twist is close to the centre of gravity of the beam. Therefore, the bracing at the centroid of the beam is ineffective. As described, the bracing requirement increases significantly with a decrease in the distance between the centre of twist and the location of the bracing. Therefore, it is clear that the application of load position must be considered in the design of a beam bracing system. In the design of a lateral beam bracing system, the strength requirement, as well as the stiffness requirement, should be met, as explained next.

Strength criteria for lateral beam bracings

The required strength for each bracing is determined by the bracing design method developed by Yura (2001). This bracing design concept was initially developed by Winter (1960) for sheathing braced design of cold-formed steel (CFS) members subjected to axial compression. However, Yura (2001) modified the design approach of Winter (1960) to account for the following design parameters: (a) different loading cases; (b) number of bracings; (c) single and double curvature and (d) discrete and relative bracing. It should be noted that the current bracing is a discrete one. Yura's modified formula for determining the bracing strength for the discretely braced structural member (F_{br}) is given as either

$$F_{br} = 0.01 C_L C_d M_f / h \quad (4.143)$$

or

$$F_{br} = 0.004 C_L C_d M_f / h \quad (4.144)$$

where C_L is the modification factor to account for the top flange loading effect, equal to unity for normal loading and $C_L = 1 + (1.2/n)$ for top flange loading; n is the number of bracings; C_d is the additional modification factor to account the number of curvatures (single or double), equal to unity ($C_d = 1$) for single curvature and $C_d = 1 + (M_S/M_L)^2$ for double curvature; M_S and M_L are the moments causing compression in the top and bottom flanges, respectively; M_f is the design moment capacity of the structural member and h is the depth (out-to-out) of the member or distance between the centroid of the force and the location of the bracing, whichever is conservative. Since the objective of the work is to avoid lateral torsional buckling of the steel girder, the design moment (M_f) has been taken as equal to yield moment capacity [M_y is the section modulus (S_x or S_y) multiplied by the yield stress (f_y)]. If the strength of the bracing is higher than the strength required (F_{br}) to achieve the design moment (M_f), it can be assumed that the beam will attain the yield moment capacity.

Stiffness criteria for lateral beam bracings

Yura's method of required bracing stiffness calculation (Yura, 2001) is from Winter's ideal lateral brace stiffness method (Winter, 1960), which was developed for sheathed CFS columns. Winter's method of bracing stiffness calculation is based on the elastic buckling stiffness of the column, where an increase in buckling load results in a corresponding increase in the stiffness required to brace the column. The modified formula for required bracing stiffness (β_i^*) is

$$\beta_i^* = 2 \frac{N_i C_b P_f}{L_b} C_d C_L \quad (4.145)$$

where the number 2 is a factor of safety, as suggested by Yura and N_i is a coefficient depending on the number of braces (n), as recommended by Winter, where, for discrete bracing $N_i = 4 - (2/n)$ and for relative bracing $N_i = 1$; n is the number of bracings; C_b is the bending coefficient, where $C_b = 1$ for uniform loading and $C_b = 1.75$ for three-point loading; $P_f = (\pi^2 E I_{yc})/L^2$; E is the Young modulus of the steel beam; I_{yc} is the moment of inertia of the compression flange; L is the unbraced length of the structural member that requires bracing; C_d is an additional modification factor to account for the number of inflection points (single or double curvature), where $C_d = 1$ for single curvature and $C_d = 1 + (M_S/M_L)^2$ for double curvature; M_S and M_L are the moments causing compression in the top and bottom flanges, respectively; C_L is a modification factor to account the top flange loading effect, where $C_L = 1$ for normal loading and $C_L = 1 + (1.2/n)$ for top flange loading and L_b is taken as the distance between the two bracing points. It should be noted that the stiffness required (β_i^*) should be matched by the axial stiffness of the bracing in order to restrain the steel beam from failure owing to lateral torsional buckling.

Torsional bracing of beams

Effectiveness of the torsional bracing

The twist of the beam can be restrained in one of three ways: (a) by providing beams with a similar cross-section in the transverse direction; (b) by providing cross-frames that connect both the top and bottom flanges of the beam or (c) by a steel-concrete composite beam along with floor or stringer beams at the bottom flange (Bradford and Pi, 2002; Mohebkah and Showkati, 2005; Park and Stallings, 2003; Valentino and Trahair, 1998). It should be noted that the provision of torsional buckling need not necessarily arrest the lateral movement of the beam. However, the beam will have a relative displacement or move as a whole.

It should also be noted that, for effective torsional restraint, it is essential for both the individual top flange and bottom flange to be braced. In addition, for torsional bracing, the difference in the effectiveness of bracing under the top flange loading and the centroid loading is insignificant (Yura, 2001). Similar to lateral bracing design, the torsional bracing should be designed for both strength and stiffness.

Strength and stiffness criteria for torsional beam bracings

The force and stiffness requirement for the torsional beam bracing are, respectively (Yura, 2001)

$$F_{br} = \frac{0.005 L L_b M_f^2}{(n h E I_{eff} C_{bb})} \quad (4.146)$$

$$B_T^* = \frac{2.4 L M_f^2}{(n h E I_{eff} C_{bb})} \quad (4.147)$$

where F_{br} and B_T^* are the force and stiffness required to brace the beam from torsional buckling; L is the beam span; L_b is the unbraced length (between the bracing points); M_f is the design moment of the beam; I_{eff} is the governing moment of inertia for the bracing design, typically the moment of inertia of the compression flange; C_{bb} is the bending coefficient (bending moment modification factor); n is the number of intermediate braces and h is the distance between the centroids of the two flanges.

Further discussion on bracings

Although Yura's work on bracing is significant and is currently being followed in the design of beam bracing systems, recent work on sheathing bracing designs for cold-formed steel beams by Selvaraj and Madhavan (2019e, 2019f, 2019g, 2019h, 2020) validates the same through experimental results. In particular, the experimental results indicate that the effect of bracing varies significantly for members that fail in minor axis buckling and lateral torsional buckling. Therefore, a new design expression for predicting the sheathing stiffness against the pull through failure is proposed. The equation proposed by Selvaraj and Madhavan (2019f) may be used in the design of bracing for a floor joist where a wood or cement sheet is used as a deck. As the concept of the bracing system design is simply a demand and supply approach, the bracing demand for the floor joist may be obtained from Yura (2001) and the strength and stiffness of the wood or cement sheet may be obtained from Selvaraj and Madhavan (2019f).

The generalised expression for predicting the stiffness of the sheathing board against pull-through failure (owing to the lateral torsional buckling of a CFS stud) is

$$k_p = \frac{\left(\frac{E}{58.4}\right) e^{(-0.1066d/2)}}{A \cdot \left(\frac{6274.4d}{E_s}\right) e^{(Bd/2)}} \quad (4.148)$$

$$A = 3112.4 E_s^{-0.909} \quad (4.149)$$

$$B = \frac{E_s}{142857.1} - 0.0437 \quad (4.150)$$

The proposed equations (Equations 4.148 to 4.150) for predicting the magnitude sheathing stiffness are valid only in SI units, where E and E_s are the Young modulus of steel (in N/mm²) and the tensile modulus of sheathing board (in N/mm²), respectively and d is the depth of the CFS stud (in mm).

Grillage analysis

The approximate representation of bridge decks as a grillage of interconnected beams is a convenient way of determining the general behaviour of the bridge under load. A grillage is a structure of rigidly connected longitudinal and transverse beams, each with a bending and torsional stiffness. At each junction of beams, deflection and slope compatibility equations can be set up. The general availability of high-speed computers has enabled the development of efficient grillage analysis. The method of analysis involves idealisation of the bridge deck through its representation as a plane grillage of discrete interconnected beams. Although the method is necessarily approximate, it has the great advantage of almost complete generality. At the joints of the grillage, any normal form of restraint to movement may be applied so that any support condition may be represented. Simply supported, built-in, continuous, discrete column support conditions and elastic foundations may all be represented without difficulty. The plan form of the deck presents no real problems. West (1971) has made recommendations, based on extensive investigation, on the use of grillage analysis for bridge decks.

There are basically three steps in the analysis procedure when analysing a bridge deck using the grillage approach: (a) idealisation of the physical deck into an equivalent grillage, (b) mathematical analysis of the grillage and (c) interpretation of the results and their use in the design of the deck. The following sections deal with the methods of idealisation and are based on the recommendations by West (1971).

Grillage idealisation of slab structure

The grillage method of analysis, pioneered by Lightfoot and Sawko (1959), represents the deck as an equivalent grillage of beams, as shown in Figure 4.54. The dispersed bending and torsion stiffnesses in every region of the slab are assumed for the purpose of analysis to be concentrated in the nearest equivalent beam. The slab's longitudinal stiffnesses are concentrated in the longitudinal beams, while the transverse stiffnesses are concentrated in the transverse beams. Ideally, the beam stiffnesses should be such that when prototype slab and equivalent grillage are subjected to identical loads, the two structures should deflect identically and the moments, shear forces and torsions in any grillage beam should equal the resultants of the stresses on the cross-section of the part of the slab that the beam represents. There are, however, some shortcomings; one has to compromise, since the grillage is an approximation of the real structure. First, equilibrium of any element of the slab requires that torques are identical in orthogonal direction and also that the twist is the same in orthogonal directions. In the equivalent grillage, this depends on the type of mesh, fine or coarse; it has been found that this condition is satisfied only when the grillage is sufficiently fine. Even so, it is often found that a coarse mesh is sufficient for design purposes. Second, the moment in any grillage beam is only proportional to the

curvature in it, while in the prototype slab the moment in any direction depends on the curvatures in that direction and the orthogonal direction. Bending stresses deduced from grillage results for distributed moments have, however, been shown to be sufficiently accurate for most design purposes.

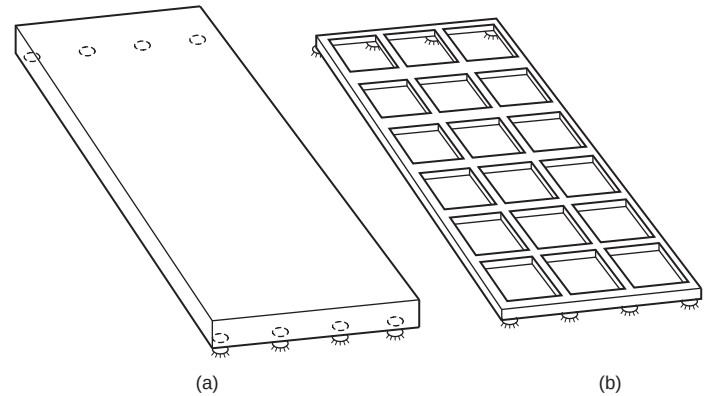


Figure 4.54 Grillage idealisation of a slab deck

It is common to have the deck in this type of structure wider than the span. A choice of nine longitudinal beams and five transverse beams is recommended for bridge decks consisting of slabs. The beams will, therefore, be of rectangular cross-section, having a width equal to the width or span of the deck or the number of beams chosen. The second moment of area and torsional inertia for the beam cross-section can be determined in the usual way. The torsional inertia of a rectangular section is calculated as $C = kbt^3$, in which b is the length of the long side of the cross-section, t the length of the short side and k a factor depending on the b/t ratio. The value of k can be obtained from any book on elasticity (Timoshenko and Goodier, 1970) and is taken as $1/3$ for $b/t > 10$.

Example 1

Consider a reinforced concrete slab 20 m long, 12 m wide and 1 m deep. The reinforcement is assumed to be of the same order of magnitude in the longitudinal and transverse directions; hence, the slab may be taken as isotropic. The proposed layout of the grillage consists of five longitudinal members and eight transverse members, as shown in Figure 4.55. The widths of the longitudinal and transverse members are, respectively, 3 m and 2.86 m. The second moment of area and torsion constant per unit width of the grillage member are calculated as

$$I_x = I_y = 1.0^3/12 = 0.0834 \text{ per metre width of the grillage member} \quad (4.151)$$

$$c_x = c_y = 1.0^3/6 = 0.167 \text{ per metre width of the grillage member} \quad (4.152)$$

For internal grillage members in the longitudinal direction, with 3 m width

$$I_x = 3.0 \times 0.0834 = 0.25 \text{ m}^4, \quad C_x = 3.0 \times 0.167 = 0.501 \text{ m}^4 \quad (4.153)$$

For edge members in the longitudinal direction

$$I_x = 1.5 \times 0.0834 = 0.125 \text{ m}^4, \quad C_x = 1.5 \times 0.167 = 0.251 \text{ m}^4 \quad (4.154)$$

Values of I and C for transverse members are calculated similarly and the grillage analysis can be completed.

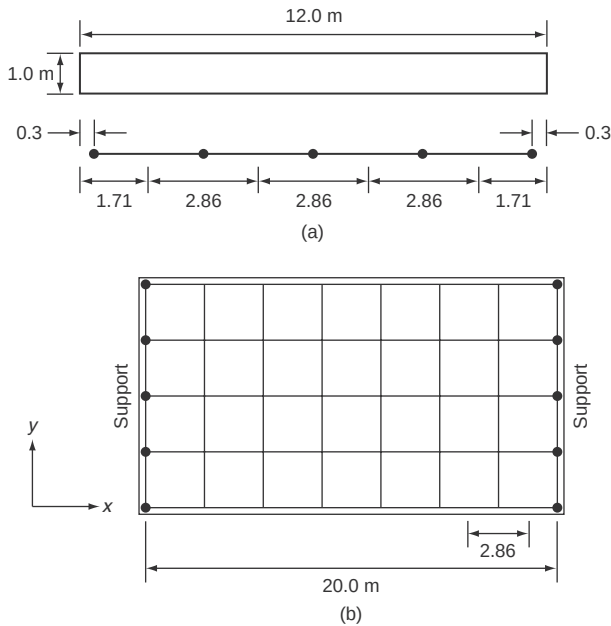


Figure 4.55 Example of an idealised slab deck

A shear-key deck can be represented by a grillage with longitudinal members coincident with the centre lines of the beams of the prototype. Each longitudinal grillage member has transverse outriggers, which are stiff. The shear keys are represented by the pinned joints between the outriggers of adjacent beams. More detailed treatment of this subject can be found in *Bridge Deck Behaviour* by Hambly (1991).

Beam-and-slab decks

Selection of a suitable grillage mesh for a beam-and-slab deck depends on the structural behaviour of a particular deck, rather than on a set of rules. Figure 4.56 shows four examples of suitable grillage arrangements for four different types of deck (Cusens and Pama, 1975). The deck shown in Figure 4.56(a) is virtually a grid of longitudinal and transverse beams. The grillage simulates the prototype closely by having its members coincident with the centre lines of the prototype's beams. For the deck given in Figure 4.56(b), it is convenient and physically reasonable to place longitudinal grillage members coincident with the centre lines of the prototype's beams. With no mid-span transverse diaphragms, the spacing of transverse grillage members is somewhat arbitrary. Where there is a diaphragm in the prototype, such as over a support, a grillage member should be coincident. In a deck with contiguous beams arranged very

closely, as shown in Figure 4.56(c), it becomes unmanageable to assume grillage beam members coincident with all beams. Therefore, it can be expedient to represent more than one beam by each longitudinal grillage member. It should also be noted that it is generally unwise to use a single grillage member to represent two beams of markedly different stiffnesses. The deck in Figure 4.56(d) has large beams with the width forming a significant fraction of the distance between the centre lines. The longitudinal beams are then best represented as slabs with two longitudinal members per beam.

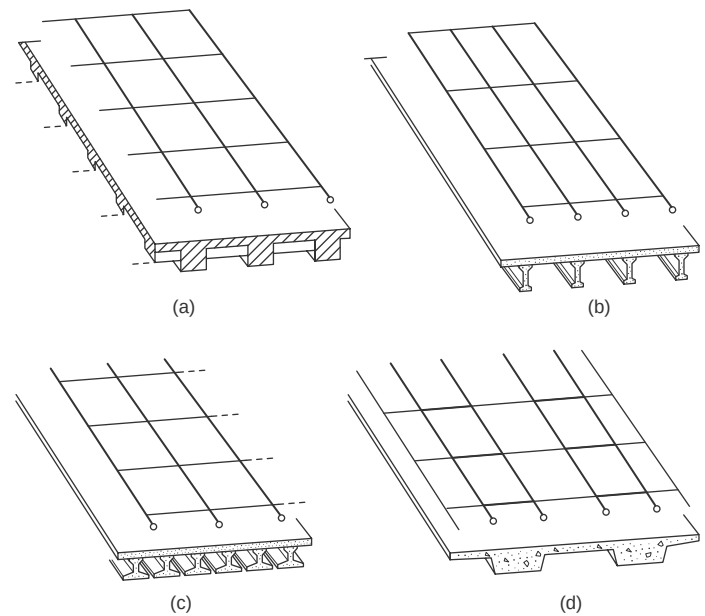


Figure 4.56 Different grillage idealisations of beam-and-slab decks

The general form of construction for the beam and slab shown in Figure 4.57 is to have relatively few (ten or fewer) longitudinal beams at 1.5–2.5 m spacing connected by a top slab. There will invariably be transverse diaphragms at the support, as well as within the span. The exception in this structural form is the use of inverted T-beams with a top slab only; these will be placed closer together; there will therefore be no more longitudinal beams. The natural choice of longitudinal grillage beams for the I-beam decks is to have them coincident with the physical beams. However, when considering inverted T-beam decks, if there are many more than nine physical beams, these should be replaced by about nine equally spaced grillage beams, positioned such that the centre lines of the edge grillage beams are coincident with the centre lines of the edge physical beams. It is recommended that the number of beams should be an odd number. Should the physical edge beams be different from the internal beams, they must be replaced by grillage beams of equivalent stiffness and the internal uniform section treated as before but with a reduced number of beams. If the deck is extremely wide and is formed from many physical beams, it may be necessary to increase the number

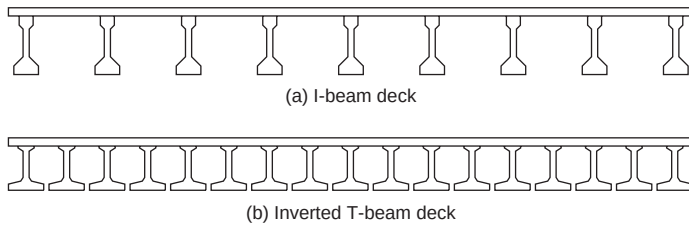


Figure 4.57 Grid-type structures

of longitudinal grillage beams, so that, as a general rule, one grillage beam does not replace more than two physical beams.

The next step is to determine the inertia for the beams. For the longitudinal beams, the moment of inertia, I can be calculated for a section consisting of a physical beam and the associated width of the top slab. It is recommended by West (1971) that it can be proportioned to all the grillage beams, as $(I \times \text{number of physical beams}) / \text{number of grillage beams}$. The torsional inertia of the beam cross-section is determined by idealising it as a section consisting of a number of rectangles. The total inertia of the section can be considered to be the sum of the inertia of the individual rectangles. The torsional inertia for an individual rectangle can be obtained in the same way as explained in the case of slab structure.

The transverse grillage beams within the span are considered to consist of a diaphragm plus slab. Other alternatives, such as considering a top slab of width corresponding to the width of a transverse grillage beam, are also recommended by West (1971). The torsional inertia of such sections may then be obtained as an I-section or a plain slab. It should, however, be ensured that the torsional stiffness used in the analysis is truly representative of the physical beam.

Example 2

West (1971) suggests a simple grillage arrangement for a bridge structure consisting of I-beams with an in situ concrete top slab, as shown in Figure 4.57. The actual structure of 25 m span and 17 m wide consists of nine precast I-beams at 2 m spacing, in situ reinforced concrete slab and in situ post-tensioned abutment diaphragms. It is proposed to choose the longitudinal beams to coincide with the physical beams, that is, nine beams of equal stiffness with the inertias calculated as for an internal beam with 2 m width of top slab. Using a ratio of 1.5:1, the spacing of transverse beams should be approximately 3 m. Nine beams at 3.125 m spacing are chosen in the transverse direction. The internal beams are of rectangles, each 3.125 m wide, while the abutment beams are assumed to be L-beams, each consisting of a full diaphragm concrete section with 1.56 m slab.

If this structure consists, in addition to post-tensioned abutments, of prestressed diaphragms positioned at quarter points, it is proposed that five transverse beams coincident with the physical beams can be used. The three internal beams are T-beams with 6.25 m of top slab, and the abutment diaphragms are L-beams with 3.125 m of top slab.

A grillage beam of MoT/C&CA standard beam M7 with top slab only, as shown in Figure 4.58(a) can be idealised as a section consisting of rectangles, as shown in Figure 4.58(b) for the purpose of torsional inertia calculations. The total inertia can be considered to be the sum of the inertias of the individual rectangles:

$$J_1 = 0.3 \times 160^3 \times 1000 \times 0.5 = 0.614 \times 10^9 \text{ mm}^4 \quad (4.155)$$

$$J_2 = 0.294 \times 75^3 \times x \times 400 = 0.050 \times 10^9 \text{ mm}^4 \quad (4.156)$$

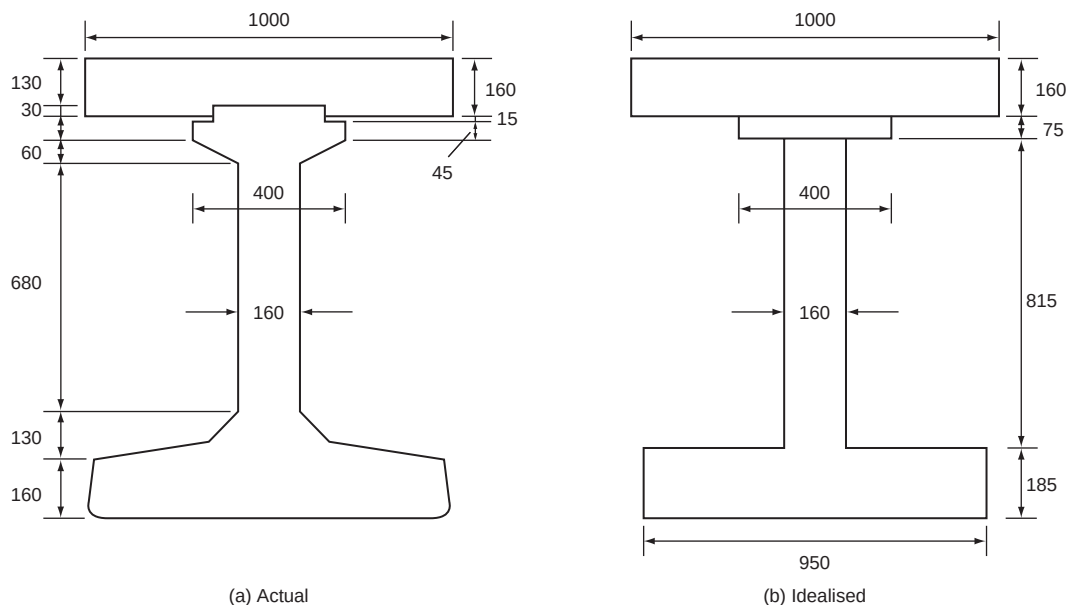


Figure 4.58 T-beam and top slab

$$J_3 = 0.292 \times 160^3 \times 815 = 0.975 \times 10^9 \text{ mm}^4 \quad (4.157)$$

$$J_4 = 0.292 \times 185^3 \times 950 = 1.756 \times 10^9 \text{ mm}^4 \quad (4.158)$$

The total inertia is

$$J = (0.614 + 0.05 + 0.975 + 1.756) \times 10^9 = 3.395 \times 10^9 \text{ mm}^4 \quad (4.159)$$

Example 3

Figure 4.59 shows part of a composite deck constructed of a reinforced concrete slab on steel beams. Hambly (1991) suggests that longitudinal grillage members are placed coincident with the centre lines of steel beams, and each represents part of the concrete slab, as shown in the figure. With modular ratio $m = 7$ for steel (short-term loading), the second moment of area and the torsional constant for the grillage beam are, respectively, obtained as

$$I_x = 0.21 \text{ m}^4 \text{ (steel beam and concrete slab)} \quad (4.160)$$

and

$$C_x = 0.000031 \times 7 + \frac{2.2 \times 0.2^3}{6} = 0.0032 \quad (4.161)$$

The second moment area I_x and torsional constant C_x are calculated for the composite section consisting of the steel beam and the associated width of the concrete slab, as shown in Figure 4.60(b).

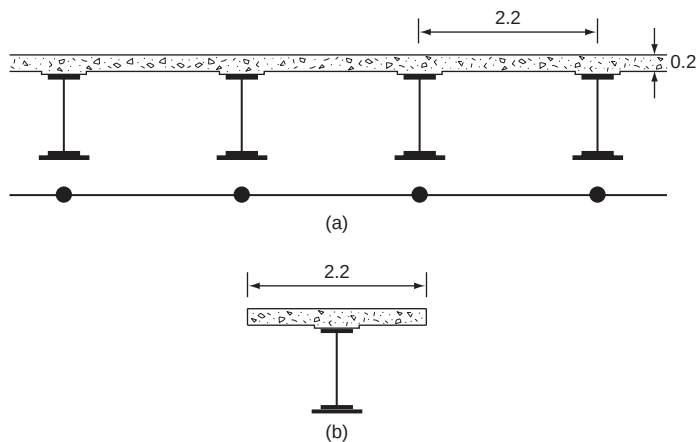


Figure 4.59 Steel beam and concrete slab deck

Example 4

For a beam-and-slab deck constructed of spaced prestressed precast concrete box beams and a reinforced concrete slab, as shown in Figure 4.60, Hambly (1991) suggests that longitudinal grillage members are placed coincident with the centre lines of beams, with additional 'nominal' members running along the centre lines of slab strips. The section properties of the nominal members are calculated for the width of the slab midway to neighbouring beams, hence

$$I_x = 1.4 \times \frac{0.25^3}{12} = 0.0018 \quad \text{and} \quad C_x = \frac{1.4 \times 0.25^3}{6} = 0.0036 \quad (4.162)$$

The properties of the beam members are calculated for the sections with flanges, including the area in nominal members (unless shear lag has reduced the effective width of flanges to less), but with previously calculated properties of 'nominal' members deducted. If the beams are much wider than those in Figure 4.61, in comparison with the beam spacing, account must be taken of the variation in transverse flexural stiffness between slab and beam. If the walls are thin, distortion of the cross-section must be considered.

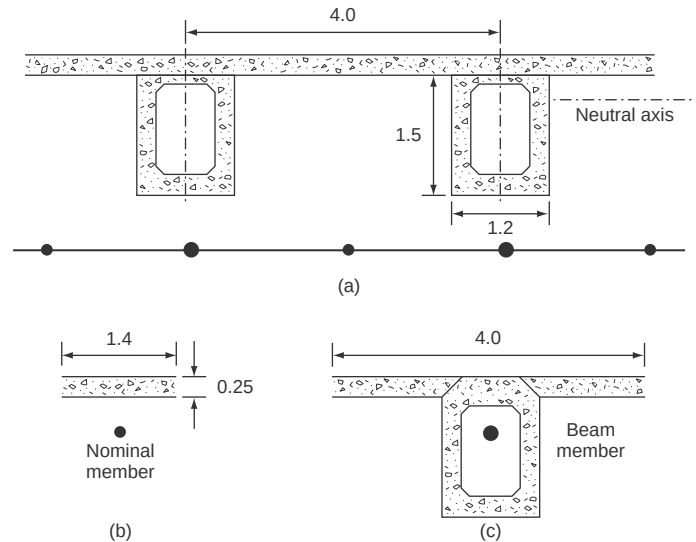


Figure 4.60 Cross-section of a deck with spaced box beams

Pseudo-slab structure

A pseudo-slab structure may be formed by either inverted T-beams acting in conjunction with a top slab (Figure 4.61(a)) or box beams with in situ voided slabs (Figure 4.61(b)). It is recommended that grillage beams for this type of bridge deck can be chosen such that the beams are coincident with the centre line of the physical beams, as in the case of beam-and-slab decks. The cross-section of the beams may be assumed as a box section and the second moment of area for each of the grillage beams is calculated as in the other cases. The torsional inertia for this type of structures should be carefully determined, to avoid underestimation of the torsional resistance of this type of cross-section. West (1971) has recommended that in the case of decks with in-span diaphragms the longitudinal and transverse inertia are calculated first as boxes and the values of C thus obtained be proportioned to all grillage beams as $0.5C$ multiplied by the number of physical beams divided by the number of grillage beams. For decks without diaphragms, however, West views the requirement of the grillage simulation against the actual behaviour of the deck. Two twisting inertias, one longitudinal and one transverse, are provided in order to reflect the overall twisting inertia of the actual deck. The twisting inertias for transverse and longitudinal beams are computed as $C_t = aC(1+a)$ and $C_l = C(1+a)$, respectively. In this, C

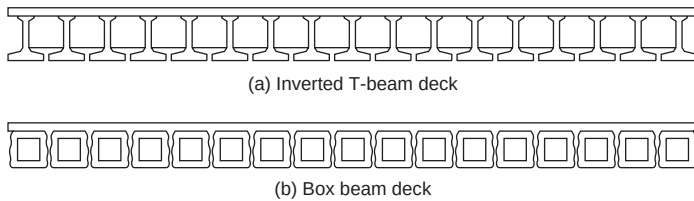


Figure 4.61 Pseudo-slab structures

represents the twisting inertia of the longitudinal cross-section and aC the twisting inertia of the transverse cross-section.

The torsional inertia C for a box section is obtained as

$$C = \frac{4A^2}{\oint \frac{ds}{t}} \quad (4.163)$$

in which A is the area inside the median line of the concrete walls and the denominator represents the sum of the lengths of the sides around the median line, each divided by the corresponding wall thickness. This expression is for thin-walled boxes but will give sufficiently accurate results for box sections where both the void dimensions are greater than the total thickness of concrete in the same direction.

Example 5

The expression for torsional stiffness referred to here is for thin-walled boxes but will give sufficiently accurate results for box sections where both the void dimensions are greater than the total thickness of concrete in the same direction. Consider the MoT/C&CA standard beam M7 used in pseudo-box construction shown in Figure 4.62 (West, 1971). The idealised section is shown in Figure 4.62(b). The thickness of the bottom in situ concrete is taken as the maximum thickness and the calculations are based on the median line:

$$A = 170 \times 800 + 775 \times 920 = 0.849 \times 10^6 \text{ mm}^2 \quad (4.164)$$

$$\oint \frac{ds}{t} = \frac{170}{200} \times 2 + \frac{775}{80} \times 2 + \frac{800}{160} + \frac{920}{130} = 33.15 \quad (4.165)$$

$$C = \frac{4 \times (0.849 \times 10^6)^2}{33.15} = 86.97 \times 10^9 \text{ mm}^4 \quad (4.166)$$

Skew decks

The recommendations made in the preceding sections on the choice of equivalent grillage beams also apply to skew decks.

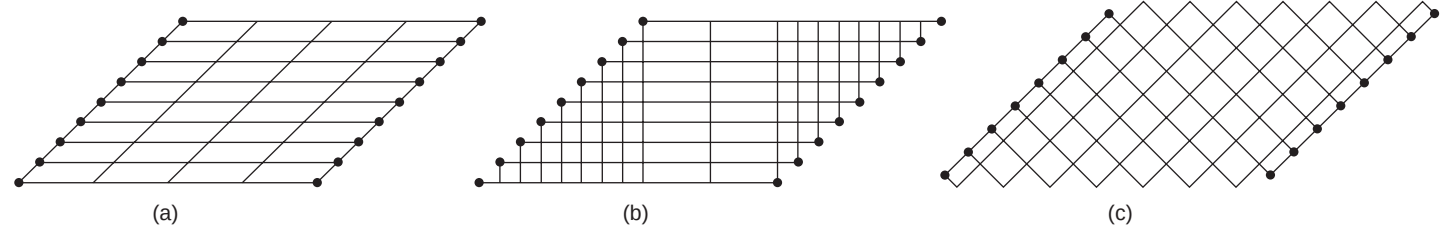


Figure 4.63 Different types of skew deck

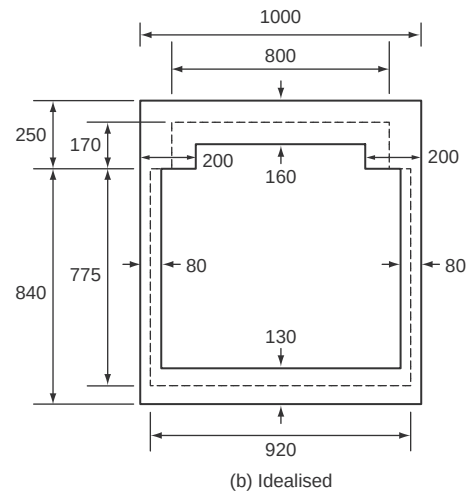
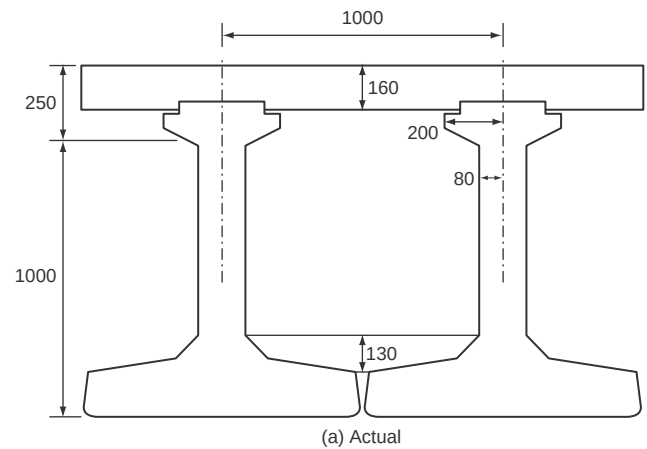


Figure 4.62 Inverted T-section as pseudo-box

There is clearly a choice between the selection of an orthogonal or skew system of grillage beams, although the directions chosen should correspond with the reinforcement layout. There are three possibilities (Cusens and Pama, 1975) for a wide skew deck, as shown in Figure 4.63. The skew system of beams is that in which transverse members are positioned parallel to the supports (Figure 4.63(a)) and the structural parameters are calculated using the orthogonal distance between the grillage beams. This system enables easier data preparation but the output will be in an inconvenient form, as moments will

relate to axes in the directions of the members. Alternatively, an orthogonal grid system (Figures 4.63(b) and (c)), in which transverse beams are positioned orthogonal to the longitudinal beams, may be employed. West (1971) has suggested that a diaphragm in the deck must be represented by an equivalent beam in the grillage simulation and that if they are within the span this will define the direction of the transverse members.

Grillage analysis for steel cellular structures

The application of a simplified grillage technique to the analysis of multi-cellular structures has been presented by Evans and Shanmugam (1979b) with reference to steel structures. It is proposed that a multi-cellular structure can be idealised as an equivalent grillage, as shown in Figure 4.64. The centre lines of the grillage members are assumed to coincide with those of the physical webs and the beam cross-section is assumed as an I-section consisting of a web plate and an assumed effective width of flange. The problems of shear lag and torsional stiffness in idealising the cellular structure into a grillage of discrete beam elements have been solved by incorporating suitable effective widths and torsional constants to the beam elements. It has been proposed that the flange effective width can be calculated in accordance with shear lag factors given in the Merrison design rules (Merrison, 1973) for bridges. These rules were based on extensive finite-element studies carried out by Moffat and Dowling (1975). Additional information on shear lag effects in box girders was provided by Evans and Taherian (1977). Torsional constants for grillage beams were assumed to be equal to that of a closed cell in the multi-cell structure. Several examples of multi-cell structures were analysed by the finite-element method and the proposed grillage method, to establish the accuracy of the method. The effect of web openings in grillage analysis was also considered by Evans and Shanmugam (1979b) and Shanmugam and Evans (1981). Further details on grillage analysis are given by Hambly (1991) and Cusens and Pama (1975). Grillage analysis can also be effectively adopted for sandwich panels with a combination of layers of materials. The application of grillage analysis has been extended to fibre-reinforced polymer composites in pressure vessels and marine structures (Blake *et al.*, 2009; Burgueño *et al.*, 2001; Sobey *et al.*, 2013).

Finite-element method

The advent of high-speed electronic digital computers has given tremendous impetus to numerical methods for solving engineering problems. Finite-element methods form one of the most versatile classes of such methods, which rely strongly on the matrix formulation of structural analysis. The application of finite-element analysis dates back to the mid 1950s with the pioneering work of Argyris (1960), Clough (1993) and others.

The finite-element method was first applied to the solution

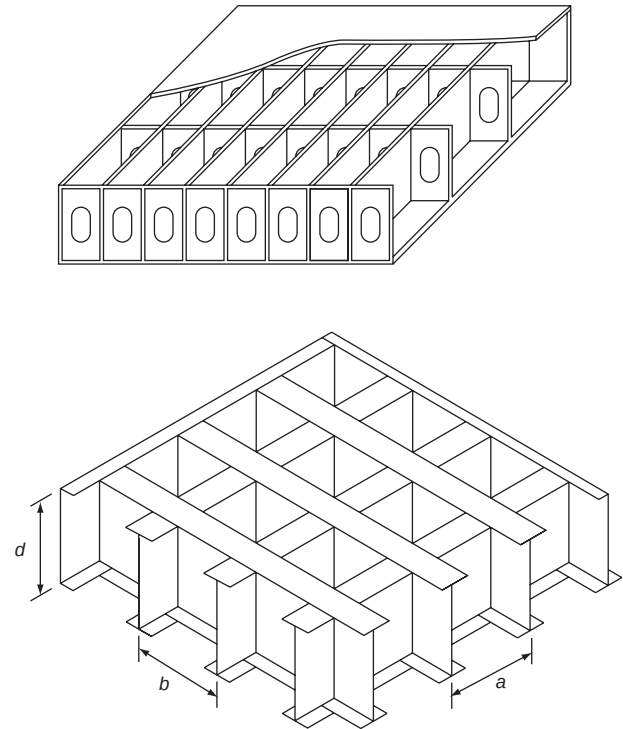


Figure 4.64 Grillage idealisation of multi-cell steel structures

of plane stress problems and subsequently extended to the analysis of axisymmetrical solids, plate-bending problems and shell problems. A useful listing of elements developed in the past is documented in textbooks on finite-element analysis.

Stiffness matrices of finite elements are generally obtained from an assumed displacement pattern. Alternative formulations are equilibrium elements and hybrid elements. A more recent development is the so-called strain-based element. The formulation is based on the selection of simple independent functions for the linear strains or change of curvature; the strain-displacement equations are integrated to obtain expressions for the displacements.

Concept of the finite-element method

The finite-element method is based on the representation of a body or a structure by an assemblage of subdivisions called finite elements, as shown in Figure 4.65. These elements are considered to be connected at nodes. Displacement functions are chosen to approximate the variation of displacements over each finite element. Polynomials are commonly employed to express these functions. Equilibrium equations for each element are obtained by means of the principle of minimum potential energy. These equations are formulated for the entire body by combining the equations for the individual elements so that the continuity of displacements is preserved at the nodes. The resulting equations are solved, satisfying the boundary conditions, to obtain the unknown displacements.

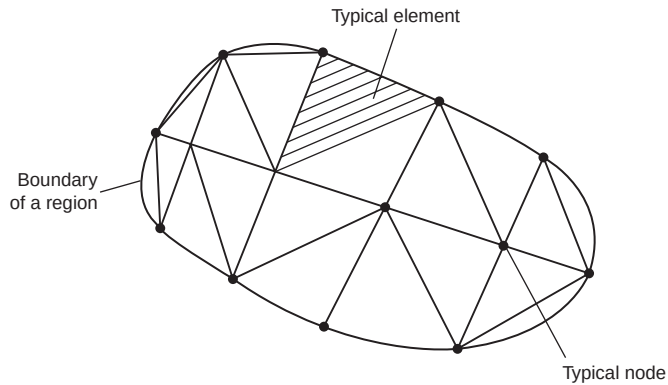


Figure 4.65 Assemblage of subdivisions

The entire procedure of the finite-element method involves the following steps. (a) The given body is subdivided into an equivalent system of finite elements. (b) A suitable displacement function is chosen. (c) The element stiffness matrix is derived using variational principles of mechanics, such as the principle of minimum potential energy. (d) The global stiffness matrix for the entire body is formulated. (e) The algebraic equations thus obtained are solved to determine unknown displacements. (f) Element strains and stresses are computed from the nodal displacements.

Basic equations from the theory of elasticity

Figure 4.66 shows the state of stress in an elemental volume of a body under load. It is defined in terms of three normal stress components, σ_x , σ_y and σ_z , and three shear stress components, τ_{xy} , τ_{yz} and τ_{zx} . The corresponding strain components are three normal strains, ϵ_x , ϵ_y and ϵ_z , and three shear strains, γ_{xy} , γ_{yz} and γ_{zx} . These strain components are related to the displacement components u , v and w at a point, as

$$\epsilon_x = \frac{\partial u}{\partial x}; \quad \gamma_{xy} = \frac{\partial v}{\partial x} + \frac{\partial u}{\partial y} \quad (4.167)$$

$$\epsilon_y = \frac{\partial v}{\partial y}; \quad \gamma_{yz} = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z} \quad (4.168)$$

$$\epsilon_z = \frac{\partial w}{\partial z}; \quad \gamma_{zx} = \frac{\partial w}{\partial z} + \frac{\partial u}{\partial x} \quad (4.169)$$

The relations given in Equations 4.167 to 4.169 are valid in the case of the body experiencing small deformations. If the body undergoes large or finite deformations, higher-order terms must be used.

The stress-strain equations for isotropic materials may be written in terms of the Young modulus and Poisson's ratio, as

$$\sigma_x = \frac{E}{1-\nu^2} [\epsilon_x + \nu(\epsilon_y + \epsilon_z)] \quad (4.170)$$

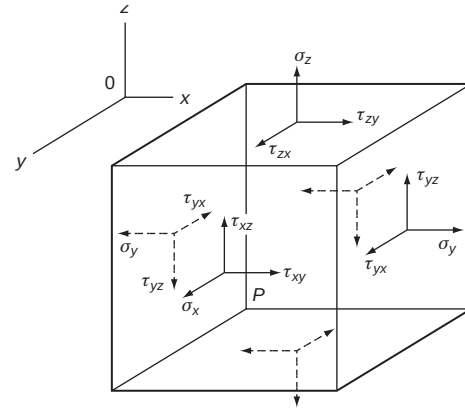


Figure 4.66 State of stress in an elemental volume

$$\sigma_y = \frac{E}{1-\nu^2} [\epsilon_y + \nu(\epsilon_z + \epsilon_x)] \quad (4.171)$$

$$\sigma_z = \frac{E}{1-\nu^2} [\epsilon_z + \nu(\epsilon_x + \epsilon_y)] \quad (4.172)$$

$$\tau_{xy} = G\gamma_{xy}, \quad \tau_{yz} = G\gamma_{yz}, \quad \tau_{zx} = G\gamma_{zx} \quad (4.173)$$

Plane stress

When the elastic body is very thin and no loads are applied in the direction parallel to the thickness, the state of stress in the body is said to be plane stress. A thin plate subjected to in-plane loading, as shown in Figure 4.67 is an example of a plane stress problem. In this case, $\sigma_z = \tau_{yz} = \tau_{zx} = 0$ and the constitutive relation for an isotropic continuum is expressed as

$$\begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix} = \frac{E}{1-\nu^2} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \begin{bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma_{xy} \end{bmatrix} \quad (4.174)$$

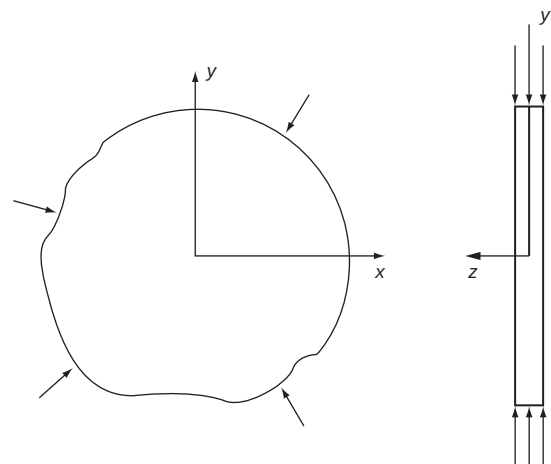


Figure 4.67 Plane stress problem

Plane strain

The state of plane strain occurs in members that are not free to expand in the direction perpendicular to the plane of the applied loads. Examples of some plane strain problems are retaining walls, dams, long cylinders and tunnels, as shown in Figure 4.68. In these problems, ε_z , γ_{yz} and γ_{zx} will vanish; hence

$$\sigma_z = \nu(\sigma_x + \sigma_y) \quad (4.175)$$

The constitutive relation for an isotropic material is written as

$$\begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix} = \frac{E}{(1-\nu)(1-2\nu)} \begin{bmatrix} (1-\nu) & \nu & 0 \\ \nu & (1-\nu) & 0 \\ 0 & 0 & \frac{1-2\nu}{2} \end{bmatrix} \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} \quad (4.176)$$

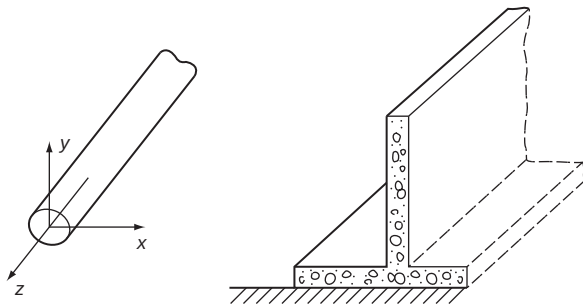


Figure 4.68 Practical examples of plane strain problems

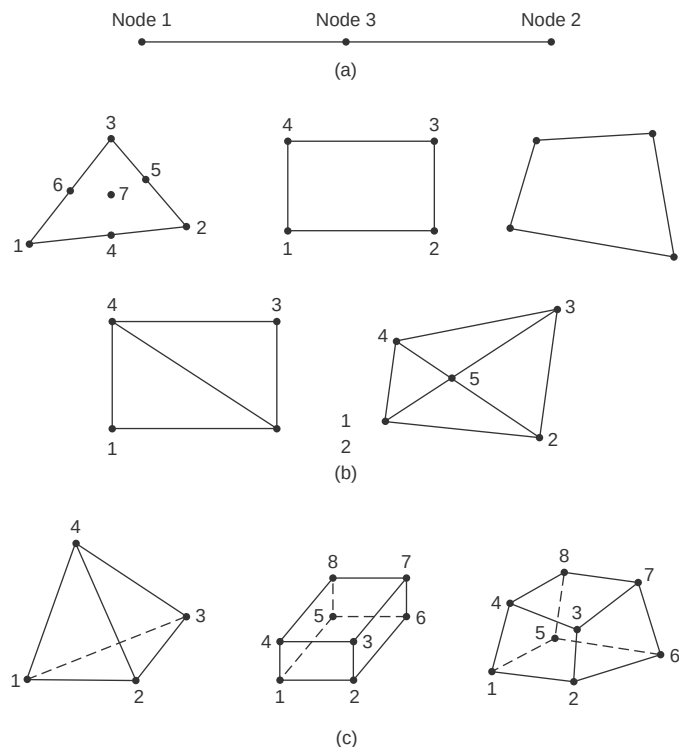


Figure 4.69 (a) One-dimensional element; (b) two-dimensional elements; (c) three-dimensional elements

Element shapes, discretisation and mesh density

The process of subdividing a continuum is an exercise of engineering judgement. The choice by an analyst depends on the geometry of the body. A finite element generally has a simple one-, two- or three-dimensional configuration. The boundaries of elements are often straight lines and the elements can be one-dimensional, two-dimensional or three-dimensional, as shown in Figure 4.69. While subdividing the continuum, one has to decide on the number, shape, size and configuration of the elements in such a way that the original body is simulated as closely as possible. Nodes must be located in locations where abrupt changes in geometry, loading and material properties occur. A node must be placed at the point of application of a concentrated load because all loads are converted into equivalent nodal-point loads.

It is easy to subdivide a continuum into a completely regular one having the same shape and size. But problems encountered in practice do not involve a regular shape; they may have regions of steep gradients of stresses. A finer subdivision

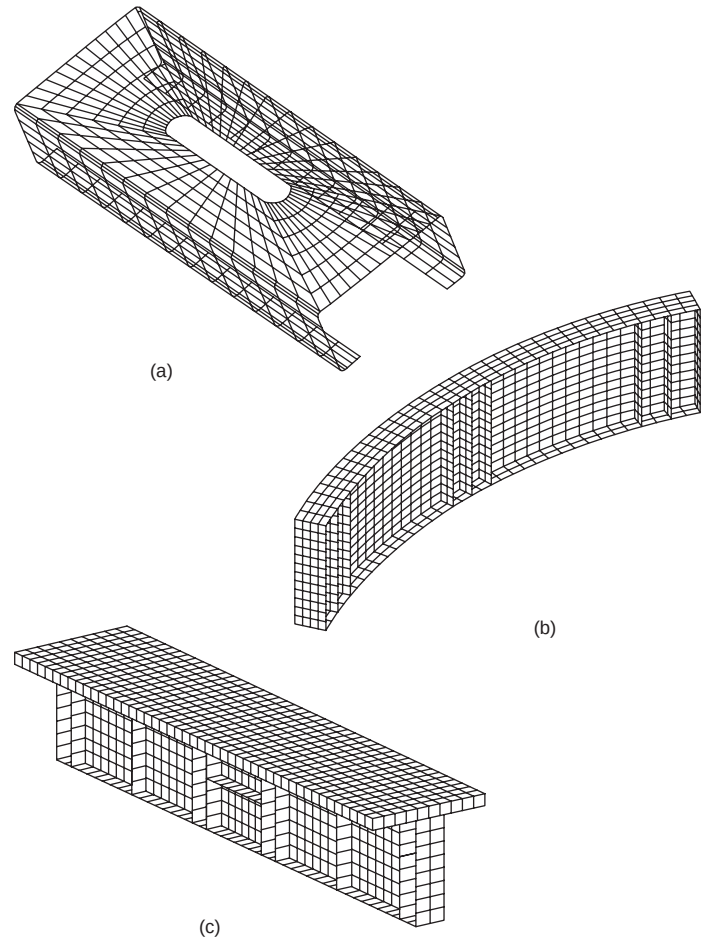


Figure 4.70 Typical examples of finite-element mesh: (a) slotted lipped channel; (b) plate girder curved in plan; (c) steel-concrete composite plate girder

may be necessary in regions where stress concentrations are expected, in order to obtain a useful approximate solution. Typical examples of mesh selection are shown in Figure 4.70.

Choice of displacement function

The selection of a displacement function is the important step in the finite-element analysis, since it determines the performance of the element in the analysis. Attention must be paid to select a displacement function that (a) has the same number of unknown constants as the total number of degrees of freedom of the element, (b) does not have any preferred directions, (c) allows the element to undergo rigid body movement without any internal strain, (d) is able to represent states of constant stress or strain and (e) satisfies the compatibility of displacements along boundaries with adjacent elements. Elements that meet both the third and fourth requirements are known as *complete elements*.

A polynomial is the most common form of displacement function. The mathematics of polynomials is easy to handle in formulating the desired equations for various elements and convenient in digital computation. The degree of approximation is governed by the stage at which the function is truncated. Solutions closer to exact solutions can be obtained by including a greater number of terms. The polynomials are of the general form

$$w(x) = a_1 + a_2x + a_3x^2 + \dots + a_{n+1}x^n \quad (4.177)$$

The coefficients a are known as *generalised displacement amplitudes*. The general polynomial form for a two-dimensional problem can be given as

$$u(x, y) = a_1 + a_2x + a_3y + a_4x^2 + a_5xy + a_6y^2 + \dots + a_my^n \quad (4.178)$$

$$v(x, y) = a_{m+1} + a_{m+2}x + a_{m+3}y + a_{m+4}x^2 + a_{m+5}xy + a_{m+6}y^2 + \dots + a_{2m}y^n \quad (4.179)$$

in which

$$m = \sum_{i=1}^{n+1} i \quad (4.180)$$

These polynomials can be truncated at any desired degree to give constant, linear, quadratic or higher-order functions. For example, a linear model in the case of two-dimensional problems can be given as

$$u = a_1 + a_2x + a_3y \quad (4.181)$$

$$v = a_4 + a_5x + a_6y \quad (4.182)$$

A quadratic function is given by

$$u = a_1 + a_2x + a_3y + a_4x^2 + a_5xy + a_6y^2 \quad (4.183)$$

$$v = a_7 + a_8x + a_9y + a_{10}x^2 + a_{11}xy + a_{12}y^2 \quad (4.184)$$

Nodal degrees of freedom

The deformation of the finite element is specified completely by the nodal displacement, rotations or strains, which are referred to as *degrees of freedom*. Convergence, geometric isotropy and potential energy function are the factors that determine the minimum number of degrees of freedom necessary for a given element. Additional degrees of freedom beyond the minimum number may be included for any element by adding secondary external nodes; such elements with additional degrees of freedom are called higher-order elements. The elements with more additional degrees of freedom become more flexible.

Isoparametric elements

The scope of finite-element analysis is also measured by the variety of element geometries that can be constructed. Formulation of element stiffness equations requires the selection of displacement expressions with as many parameters as there are node point displacements. In practice, for planar conditions, only the four-sided (quadrilateral) element finds as wide an application as the triangular element. The simplest form of quadrilateral, the rectangle, has four node points and involves two displacement components at each point for a total of eight degrees of freedom. In this case, one would choose four-term expressions for both u and v displacement fields. If the description of the element is expanded to include nodes at the mid-points of the sides, an eight-term expression would be chosen for each displacement component.

The triangle and rectangle can approximate the curved boundaries only as a series of straight-line segments. A closer approximation can be achieved by means of *isoparametric* coordinates. These are non-dimensionalised curvilinear coordinates whose description is given by the same coefficients as are employed in the displacement expressions. The displacement expressions are chosen to ensure continuity across element interfaces and along supported boundaries, so that geometric continuity is assured when the same forms of expressions are used as the basis of description of the element boundaries. The elements in which the geometry and displacements are described in terms of the same parameters and are of the same order are called *isoparametric elements*. The isoparametric concept enables one to formulate elements of any order that satisfy the completeness and compatibility requirements and that have isotropic displacement functions.

Element shape functions

The finite-element method is not restricted to the use of linear elements. Most finite-element codes, commercially available, allow the user to select between elements with linear or quadratic interpolation functions. In the case of quadratic elements, fewer elements are needed to obtain the same degree

of accuracy in the nodal values. Also, the two-dimensional quadratic elements can be shaped to model a curved boundary. Shape functions can be developed based on the following properties: (a) each shape function has a value of one at its own node and is zero at each of the other nodes; (b) the shape functions for two-dimensional elements are zero along each side that the node does not touch and (c) each shape function is a polynomial of the same degree as the interpolation equation.

Formulation of the stiffness matrix

It is possible to obtain all the strains and stresses within the element and to formulate the stiffness matrix and a consistent load matrix once the displacement function has been determined. This consistent load matrix represents the equivalent nodal forces, which replace the action of external distributed loads.

As an example, consider a linearly elastic element of any of the types shown in Figure 4.71. The displacement function may be written in the form

$$\{f\} = [P]\{A\} \quad (4.185)$$

in which $\{f\}$ may have two components $\{u, v\}$ or simply be equal to w , $[P]$ is a function of x and y only, and $\{A\}$ is the vector of undetermined constants. Applying Equation 4.185 repeatedly to the nodes of the element, one after the other, gives a set of equations of the form

$$\{D^*\} = [C]\{A\} \quad (4.186)$$

in which $\{D^*\}$ contains the nodal parameters and $[C]$ contains the relevant nodal coordinates. The undetermined constants $\{A\}$ can be expressed in terms of the nodal parameters $\{D^*\}$ as

$$\{A\} = [C]^{-1}\{D^*\} \quad (4.187)$$

Substituting Equation 4.187 into Equation 4.185:

$$\{f\} = [P][C]^{-1}\{D^*\} \quad (4.188)$$

Constructing the displacement function directly in terms of the nodal parameters gives

$$\{f\} = [L]\{D^*\} \quad (4.189)$$

where $[L]$ is a function of both (x, y) and $(x, y)_{i,j,m}$, given by

$$[L] = [P][C]^{-1} \quad (4.190)$$

The various components of strain can be obtained by appropriate differentiation of the displacement function. Thus

$$\{\varepsilon\} = [B]\{D^*\} \quad (4.191)$$

where $[B]$ is derived by differentiating appropriately the elements of $[L]$ with respect to x and y . The stresses $\{\sigma\}$ in a linearly elastic element are given by the product of the strain and a symmetrical elasticity matrix $[E]$. Thus

$$\{\sigma\} = [E]\{\varepsilon\} \quad (4.192)$$

or

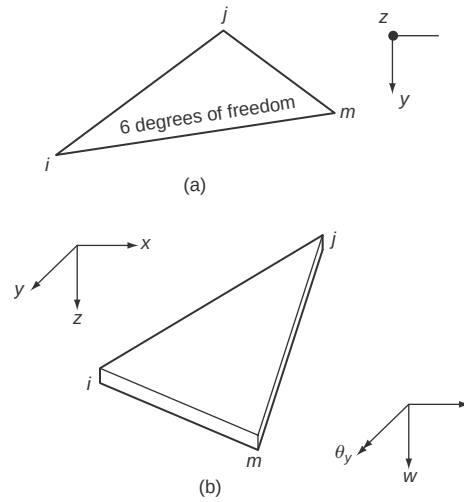


Figure 4.71 Degrees of freedom: (a) triangular plane stress element; (b) triangular bending element

$$\{\sigma\} = [E][B]\{D^*\} \quad (4.193)$$

The stiffness and consistent load matrices of an element can be obtained using the principle of minimum total potential energy. The potential energy of the external load in the deformed configuration of the element is written as

$$W = -\{D^*\}^T\{Q^*\} - \int_a \{f\}^T\{q\}da \quad (4.194)$$

In Equation 4.194, $\{Q^*\}$ represents concentrated loads at nodes and $\{q\}$ the distributed loads per unit area. Substituting for $\{f\}^T$, from Equation 4.189 gives

$$W = -\{D^*\}^T\{Q^*\} - \{D^*\}^T \int_a [L]^T\{q\}da \quad (4.195)$$

Note that the integral is taken over the area a of the element. The strain energy of the element integrated over the entire volume v is given as

$$U = \frac{1}{2} \int_v \{\varepsilon\}^T\{\sigma\}dv \quad (4.196)$$

Substituting for $\{\varepsilon\}$ and $\{\sigma\}$ from Equations 4.191 and 4.193, respectively:

$$U = \frac{1}{2} \{D^*\}^T \left(\int_v [B]^T [E] [B] dv \right) \{D^*\} \quad (4.197)$$

The total potential energy of the element is

$$V = U + W \quad (4.198)$$

or

$$V = \frac{1}{2} \{D^*\}^T \left(\int_v [B]^T [E] [B] dv \right) \{D^*\} - \{D^*\}^T \{Q^*\} - \{D^*\}^T \int_a [L]^T\{q\}da \quad (4.199)$$

Using the principle of minimum total potential energy gives

$$\left(\int_v [B]^T [E] [B] dv \right) \{D^*\} = \{Q^*\} + \int_a [L]^T\{q\}da \quad (4.200)$$

or

$$[K]\{D^*\} = \{F^*\} \quad (4.201)$$

where

$$[K] = \int_V [B]^T [E] [B] dv \quad (4.202)$$

and

$$\{F^*\} = \{Q^*\} + \int_a [L]^T \{q\} da \quad (4.203)$$

The stiffness matrix can now be developed for such problems as plates subjected to in-plane forces or beam elements or plates in bending. As mentioned before, the accuracy is dependent on the choice of an appropriate mesh. Many refined elements giving greater accuracy are described in standard books on finite-element methods (Desai and Abel, 1972; Nath, 1974; Zienkiewicz and Cheung, 1967). Finite-element methods are adopted nowadays for solving a wide range of problems. It is possible to buy finite-element software, which can be employed to solve many complex analytical problems.

Non-linear elements

Various behaviours are called 'non-linear'. Stress-strain relations may be non-linear in either a time-dependent or time-independent way. Displacements may cause loads to alter their distribution or magnitude. Connected parts may slip or stick. Gaps in a structure may open or close. The problem may be static or dynamic. Non-linear analysis is harder to understand and more involving because of the mathematical complexity. Nevertheless, non-linear analyses are becoming more common because of stringent design requirements and because finite elements and the computer have made non-linear analysis a practical possibility. Most analysis programs allow the user to choose from various kinds of non-linearity and solution algorithms. A user must understand the problem and the analytical tools well enough to make an intelligent choice. Even then, several analyses may be needed to get a satisfactory result. An incremental analysis gives an answer, but another analysis with a different step length is needed to estimate the quality of the answer. An iterative analysis may fail to converge because of a bug in the program or in the data, numerical error, greater non-linearity than the algorithm can accommodate or a prescribed load greater than the structure's collapse load (Bushnell, 1974).

Non-linear problems are usually solved by taking a series of linear steps. In structural terms, the process of non-linear analysis is explained by writing equilibrium equations in the incremental form:

$$[K] \{\Delta D\} = \{\Delta R\} \quad (4.204)$$

Here, the stiffness matrix $[K]$ is a function of displacements $\{D\}$ because the problem is non-linear. In turn, the current $\{D\}$ is the sum of preceding $\{\Delta D\}$ s. The current $[K]$, called the tangent stiffness, is used to compute the next step, $\{\Delta D\}$. The next step is to update $\{D\}$, update $[K]$, and all is ready

to take another step. In this way, a load-displacement curve is approximated as a series of straight-line segments and the accuracy of the results depends on the number of segments. Similar solutions can apply to both geometric and material non-linearity; available computer programs can allow both to be active at the same time.

The essential feature of geometric non-linearity is that equilibrium equations must be written with respect to the deformed geometry, which is not known in advance. Only if the nature of the problem is substantially unchanged by deformation is the problem called 'linear' and it is presumed that equilibrium equations can refer to the initial configuration. A large-displacement problem can be analysed in Lagrangian or Eulerian coordinates. If stress-strain relations are linear, or non-linear but elastic, there is a unique relation between stress and strain. But if there are plastic strains, the stress-strain relation is path-dependent, not unique: a given state of stress can be produced by many different straining procedures. In addition, different materials require different material theories. The essential computational problem of material non-linearity is that equilibrium equations must be written using material properties that depend on the strains, but the strains are not known in advance. For a detailed treatment of geometric non-linearity, material non-linearity, formulation, solution algorithm and choice of solution method, readers may refer to the reference books of Cook (1981) and Oden (1972).

Finite-strip method

The finite-strip method combines some of the benefits of the series solution of orthotropic plates with the finite-element concept. Bridge decks having the same cross-section from one end to the other (prismatic) can be analysed more economically and efficiently using this method. When it is not possible to find a displacement function applicable to all regions of a plate, it is suggested that the plate may be divided into discrete longitudinal strips, spanning between supports. The strips are connected by nodes that run along the sides of the strips, as shown in Figure 4.72. Simple displacement interpolation functions may be used to represent displacement fields within and between individual strips. For a longitudinal strip I (Figure 4.72) that is simply supported at its ends, the displacement function may be assumed as a third-degree polynomial in the form

$$w(x,y) = \sum_{n=1}^{\infty} (a_1 + a_2 y + a_3 y^2 + a_4 y^3) \sin \frac{n\pi x}{L} \quad (4.205)$$

satisfying the boundary conditions of the strip. The displacement function is assumed to be a simple polynomial in the transverse direction, resulting in discontinuity of stresses at the strip interfaces. This equation may be written in terms of the nodal line displacements, to establish continuity at the boundaries with adjacent strips. The formulation and solution is similar to the finite-element method and the detailed account of the method may be found in the references (Cheung and Cheung, 1971; Cheung *et al.*, 1970; Cusens and Pama, 1975).

Any form of loading may be conveniently handled by this method and the method is well-suited for computer use. The method has, however, some limitations, in the sense that it is effectively applicable only to prismatic structures with simply supported ends.

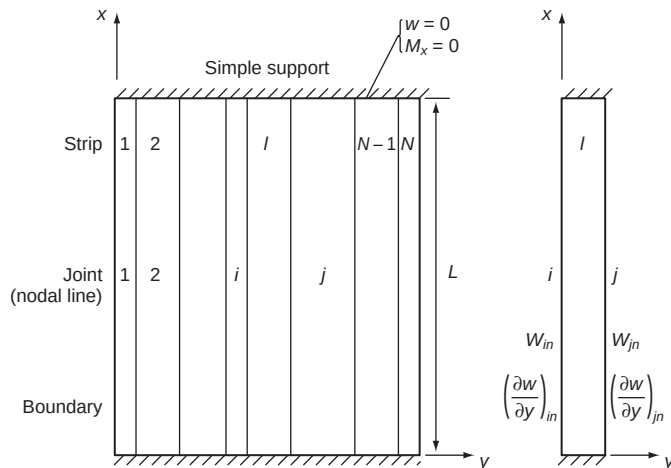


Figure 4.72 Finite-strip simulation for plate bending

New finite-strip analysis software has been developed for analysing the elastic buckling capacity of thin-walled prismatic sections (Papangelis and Hancock, 1995). It is a tedious task to determine the critical elastic buckling stress and the eigenmode that governs the local or distortional buckling mode of thin-walled structural members with complex shapes (Selvaraj and Madhavan, 2019i). Therefore, the purpose of this software is to simplify the buckling analysis for the various buckling shapes (local, distortional and global) of the thin-walled sections.

Simulation packages and latest improvements

Although it is difficult to predict complex structural, such as post-buckling, behaviour of components, the numerical techniques and simulation packages have reached a level of maturity that enables any kind of structural analysis. In addition, the speed of the simulation has also been increased significantly. Most of the latest simulation packages are user-friendly and open source. These computer programs are also being used for the structural topology optimisation of the structural components developed using additive manufacturing. Most of the generalised optimisation algorithms are freely available on the internet (Zuo and Xie, 2015).

Furthermore, a software interface has been generated to integrate two or more software packages, such as Matlab (mathematical) and Abaqus (finite-element), to simplify the post-processing task and reiterate the analysis in the case of deviation from the desired results (Papazafeiropoulos *et al.*, 2017). A large variety of element types are also being developed

for specific uses. For example, a new element type has been developed to simulate the behaviour of a shear wall ('shear-wall element' in OpenSees software) (Lu *et al.*, 2015) and a new model has been developed to simplify the partial interaction between steel and concrete in composite beams (Zona and Ranzi, 2011). The traditional process of finite-element analysis is also being transformed to an automatic process by scanning the physical object, where the node and element type is selected by an algorithm based on the complexity of the structure and desired results (Huang *et al.*, 2015).

Stiffness of supports: soil–structure interaction

General behaviour

Bridge decks are supported by piers and abutments, which are again supported by foundations resting on soil. The stiffness of piers, abutments, foundations and soil are all significant in analysing the bridge for safety.

Piers and abutments, together termed the substructure, are subjected to many types of loading, which include: live and dead loads from the superstructure; the dead load of the substructure; soil pressure; wind load on the superstructure, substructure and vehicles; pressure resulting from stream flow and ice; and earthquake load. In addition, the settlement of supports gives rise to secondary effects that result in significant changes to the distribution of forces within the bridge deck. It is, therefore, essential for a bridge designer to devote sufficient attention to assess the stiffness of substructures. The distribution of forces in the superstructure and substructure of a bridge depends on the relative stiffnesses of all its components. Continuous structures can be very effective in distributing forces, but they are also very sensitive to the effects of compressibility of supports and foundations. Any reasonable combination of these loads can be applied in the design of substructures.

The substructure is designed for stability, strength and limit of soil pressure. One load combination may be most critical for stability, while another loading may produce maximum stresses in the concrete or reinforcing steel, and a third combination may give the greatest soil pressure. If a portion of the pier is submerged, the buoyancy effect must be considered in determining the stability of the pier. Because of the many possible load combinations, substructure design is complicated. Careful soil investigation should be carried out to determine the stability, earth pressure and permissible soil pressures. If the bridge is to be located in an earthquake region, locations of fault zones, slide regions or any possible large unstable ground areas should be carefully studied before the bridge location and type are selected.

The distribution of forces throughout the foundation system of the structure is significantly influenced by the relative stiffnesses of all the components of the bridge, including the individual bearings. Continuous structures, though effective at distributing forces, are very sensitive to the effects of

compressibility of supports and foundations. For example, a box girder deck, which is very stiff against torsion and distortion, is very sensitive to differential settlement and the compression of bearings.

Depending on the geometry of the substructure, loads from the superstructure will be transmitted to the foundation, either by direct compression or by compression and bending. Piers, shown in Figures 4.73(a) to (c), transmit the deck loads to the foundations by direct compression. Conversely, structures that involve crosshead beams or cantilevers, as shown in Figures 4.73(d) and (e), may have significant flexibility (Hambly, 1991). If a bridge deck on such supports is analysed with a grillage approximation, it may be necessary to model the crosshead as well as the deck. It may be easier to model the structure as a space frame that reproduces the shape of the pier. It is then relatively simple to model compressible bearings using vertical members between the deck and crosshead, and to model the foundation stiffnesses. For example, the deck shown in Figure 4.74(a) supported on a pier (Figure 4.73(d)) can be idealised by a space frame model, as shown in Figure 4.74(b).

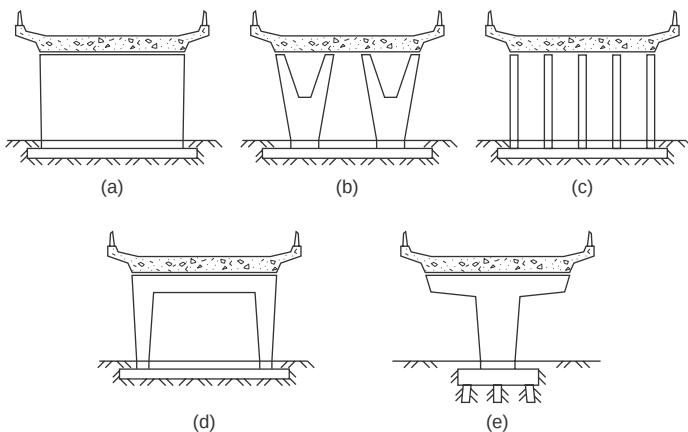


Figure 4.73 Bridge supports with different stiffnesses

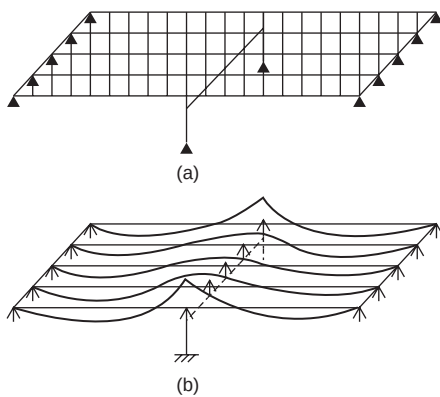


Figure 4.74 Space frame model of a bridge deck and the bending moment diagram

Foundation stiffness and interaction with soil

A foundation subjected to dynamic loading will experience a motion at the same frequency as the applied force. It is essential to impose a limit on the dynamic force that the foundations may experience. There is a prescribed limit for the dynamic motions to be permitted, and also a prescribed limit on the settlement that may develop. Methods based on the theory for an elastic half space have been proposed by Barkan (1962), Das (1990) and Gorbunov-Possadov and Serebrajanyi (1961) to determine the stiffnesses of shallow footing foundations (Figure 4.75). Equations from these methods have been simplified and approximate estimates of stiffnesses obtained by Hambly (1991). These simplified equations are reproduced for shallow footings that may slide or tilt across the shorter direction:

$$\text{shear modulus} \quad G = \frac{E}{2(1+\nu)} \quad (4.206)$$

$$\text{vertical} \quad K_z = \frac{2.5 GA^{0.5}}{(1-\nu)} \quad (4.207)$$

$$\text{horizontal stiffness} \quad K_x = 2G(1+\nu)A^{0.5} \quad (4.208)$$

$$\text{rocking stiffness} \quad K_m = \frac{2.5 GZ}{(1-\nu)} \quad (4.209)$$

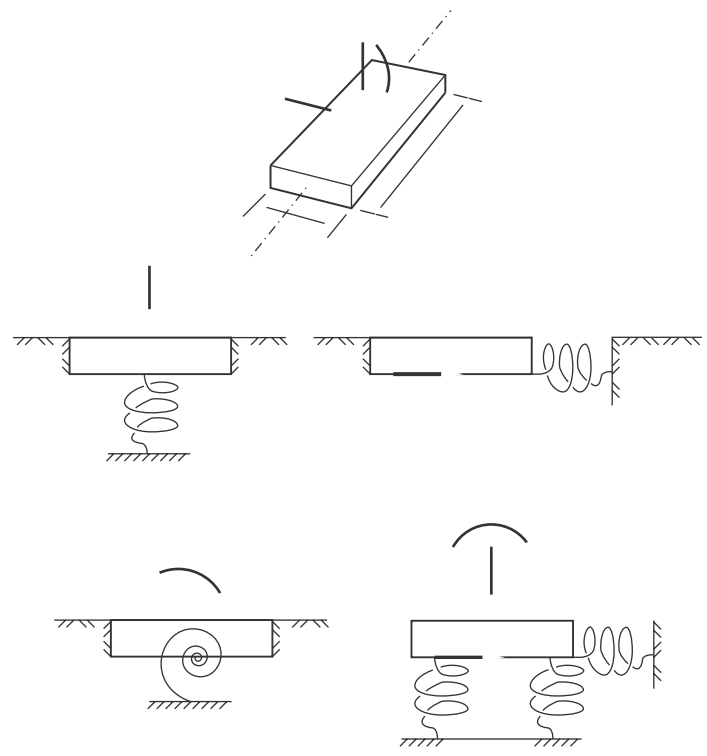


Figure 4.75 Spring models for the stiffness of footings

where G is the shear modulus of soil; E is the Young modulus of soil; ν is Poisson's ratio of the soil; A is the foundation area (bd) and Z is the foundation section modulus ($bd^2/6$). The vertical and rotational stiffnesses can be more conveniently represented by two parallel springs, as shown in Figure 4.75(e), for which the stiffness is taken as $K = 0.5K_z$. The parallel springs are spaced at a distance ℓ given by $\ell = 2(K_m/K_z)^{0.5} = 0.82b^{0.25}d^{0.75}$. These equations are approximate; however, they provide a quick means of determining the order of magnitude of foundation stiffnesses.

Detailed investigations of the stiffnesses of foundations for bridges over complicated ground conditions can be conducted using three-dimensional finite-element analysis. Finite-element programs that can represent the soils with very sophisticated non-linear stress-strain behaviour and coexisting pore water pressures are now available.

Soil data of the quality essential for sophisticated computer programs are not frequently available. Where the soil support is applied some depth below the usable portion of the structure, a pile foundation is most commonly used. Wooden, concrete or steel piles and steel tubes filled with concrete are common; Figure 4.76 shows the usual values of maximum length and maximum design load for various types of pile (Lambe and Whitman, 1979). The stiffness of pile foundations is more complicated, particularly if the foundations obtain their stiffnesses from the interaction of pile bending and lateral forces from the soil, rather than by axial compression of the piles. Several computer programs are available, which can be used to calculate the stiffnesses of pile groups, either using finite elements, or the theory for an elastic half space (Poulos and Davis, 1980).

Consider, for example, the foundations for a portal frame shown in Figure 4.77. The footings for the portal frame have $d = 3$ m (parallel to span) and $b = 12$ m. Hence

$$\text{Footing area} \quad A = 12 \times 3 = 36 \text{ m}^2 \quad (4.210)$$

$$\text{Section modulus of the footing} \quad Z = 12 \times 3^2 / 6 = 18 \text{ m}^3 \quad (4.211)$$

The stiffnesses, with Poisson's ratio of the order of 0.3 to 0.5, can be calculated approximately as

$$K_z = 1.5E(A)^{0.5} = 1.5E(36)^{0.5} = 9E \quad (4.212)$$

$$K_x = E(A)^{0.5} = E(36)^{0.5} = 6E \quad (4.213)$$

$$K_m = 1.5EZ = 1.5E(18) = 27E \quad (4.214)$$

For an assumed value of $E = 120$ MPa, $K_z = 1080$ MN/m, $K_x = 720$ MN/m and $K_m = 3240$ MN/m. The total width of the foundation is 12 m and stiffnesses per unit width are calculated as

$$K_z = 1080/12 = 90 \text{ MN/m}^2 \quad (4.215)$$

$$K_x = 720/12 = 60 \text{ MN/m}^2 \quad (4.216)$$

$$K_m = 3240/12 = 270 \text{ MN/m}^2 \quad (4.217)$$

The stiffnesses K_z and K_m are represented by two parallel springs of

$$K = 0.5K_z = 45 \text{ MN/m}^2 \quad (4.218)$$

at a spacing of

$$\ell = 2(K_m/K_z)^{0.5} = 2(270/90)^{0.5} = 3.464 \text{ m} \quad (4.219)$$

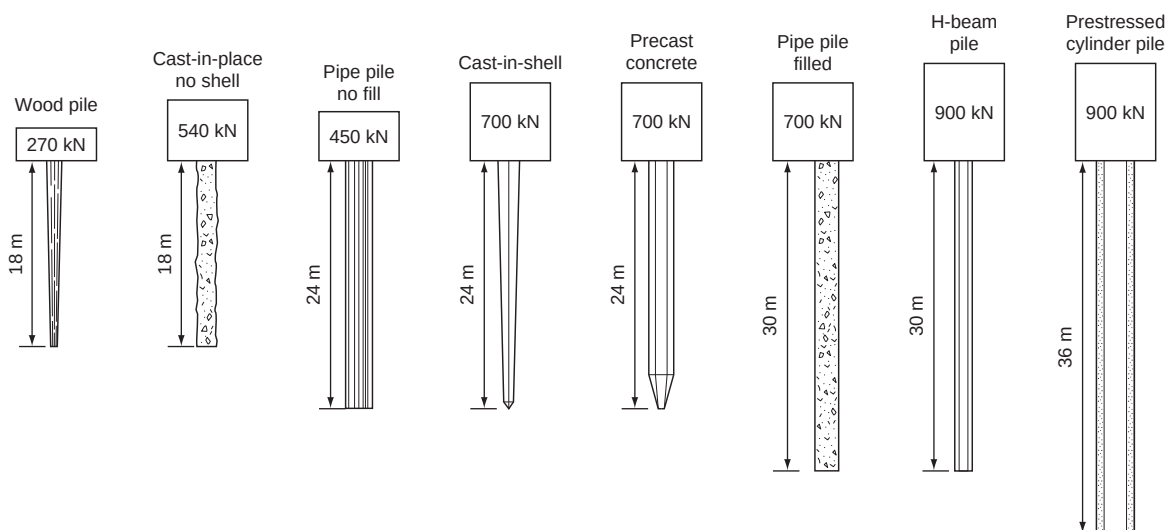


Figure 4.76 Usual maximum length and maximum design loads for various types of pile

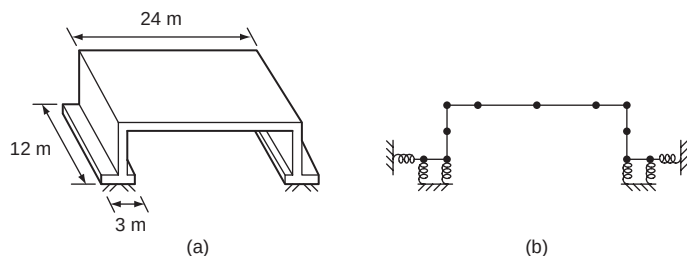


Figure 4.77 Arrangement and plane frame model of portal-type bridge

The influence of more complicated ground conditions can be investigated using finite-element analysis, where any degree of sophistication can be included.

Stiffness moduli of soils

Foundation siffnesses are functions of many variables that include the Young modulus of soil beneath the foundation. The elastic modulus of granular soils based on effective stresses is a function of grain size, gradation, mineral composition of the soil grains, grain shape, soil type, relative density, soil particle arrangement, stress level, prestress pore water pressure and soil cohesiveness. Typical values of the elastic modulus and Poisson's ratio for normally consolidated granular soils (Das, 1990) are given in Table 4.1. The undrained elastic modulus of cohesive soils is a primarily function of soil plasticity and overconsolidation. It can be determined from the slope of a stress–strain curve obtained from an undrained triaxial test (Holtz and Kovacs, 1981).

Type of soil	Elastic modulus: MPa	Poisson's ratio
Loose sand	10–24	0.20–0.40
Medium dense sand	17–28	0.25–0.40
Dense sand	35–55	0.30–0.45
Silty sand	10–17	0.20–0.40
Sand and gravel	69–170	0.15–0.35

Table 4.1 Typical values of elastic modulus and Poisson's ratio for granular soils (Das, 1990)

To obtain ground stiffnesses, one has to determine, by site investigation, the soil properties. This is not straightforward, because tests on small samples seldom provide reliable information of a large soil mass. Stiffnesses are best estimated from large-diameter plate bearing tests and from back analyses of observations of comparable structures on similar ground conditions. The elastic modulus and Poisson's ratio are not constants for a soil, but rather are quantities that approximately describe the behaviour of a soil for a particular set of stresses. Different values of elastic modulus and Poisson's ratio will apply for any other set of stresses. Good judgement is needed when choosing values of these parameters. It is difficult to estimate values of elastic modulus with much accuracy, and

test data for the particular soil will be necessary whenever an accurate estimate is needed. The terms *tangent modulus* and *secant modulus* are frequently used. Specific guidance and information, available in textbooks, is helpful for preliminary assessments of soil–structure interaction. However, at the final design stage, calculations should be supported by investigations of the specific site and foundation conditions. The stiffness analysis of soils on the structures can be determined by the approaches specified in Addenbrooke *et al.* (1997), Chen and Martin (2002), Jardine *et al.*, (1986), Kausel and Roësset (1981) and Karpurapu and Bathurst (1992).

Retaining walls

It is not uncommon to design bridge abutments as gravity-type retaining walls, typical dimensions of which are shown in Figure 4.78. Lateral active earth pressures for the design of low retaining walls are usually estimated using conservative design charts. Some designers also use equivalent fluid pressures to compute the active earth pressure as $p_a = \gamma_{eq} H$, in which γ_{eq} equals the product of the minimum active earth pressure coefficient, K_a , and the unit weight, γ , of the backfill material. More rigorous methods using Rankine or Coulomb theories can also be used to determine the earth pressure. All earth retention structures should be designed to sustain potential surcharge loadings. It is assumed that retaining walls are constructed with free-draining backfill materials and with effective drainage systems so that ponding of water, which might result in additional load on the retaining structure, can be minimised or eliminated. If it is not possible to preclude ponding, the wall should be designed to account for greater total pressures. The design of retaining walls involves several steps that include checks for overturning stability, bearing capacity, sliding, excessive settlement and overall stability of the earth mass that contains the retaining structure. Detailed discussions of the analysis and design of retaining walls subjected to earth pressure with static and seismic responses are available in Bathurst and Hatami (1998), Callisto and Soccodato (2009), Karpurapu and Bathurst (1992) and Pappin *et al.* (1986).

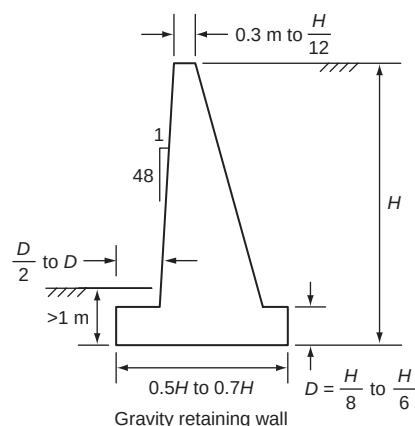


Figure 4.78 Typical dimensions of a gravity retaining wall

Stiffness from lateral earth pressure

Horizontal forces exerted on bridge supports are influenced by the earth pressure, thermal expansion and vehicle braking forces. The distribution of these forces on abutments and piers depends on their relative horizontal stiffnesses. Great care should be exercised while estimating these stiffnesses. Earth pressure–displacement diagrams can be employed to estimate the lateral stiffness of an abutment (Hambly and Burland, 1979). The following example (Hambly, 1991) demonstrates how the horizontal stiffness can be calculated from earth pressure resistance and rocking rotation of an abutment.

Consider the abutment of height 6 m shown in Figure 4.79. Initially, guess a displacement of 12 mm at deck level, that is, 0.002 of the height. The earth pressure coefficient would correspondingly change from about 0.4 to 1.5. The earth resistance force per unit width of abutment, with retained fill of density γ , is

$$P = \frac{1}{2} K \gamma H^2 \quad (4.220)$$

inclined to the horizontal at the angle of the wall friction. Along the wall, all components of stress increase linearly with depth, and so the resultant thrust acts at the third-point of the wall. With a uniform surcharge q_s (Figure 4.79), the total active thrust against the wall is given by the following:

$$P = \frac{1}{2} K \gamma H^2 + q_s H K \quad (4.221)$$

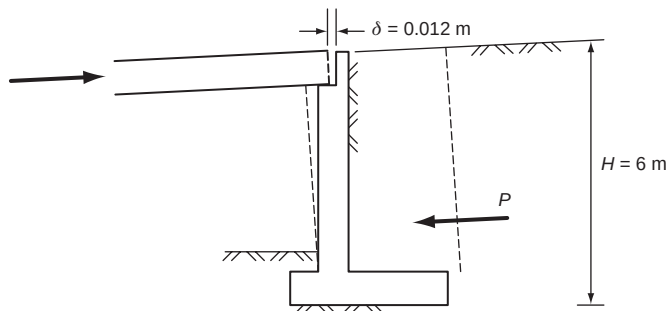


Figure 4.79 Bridge abutment resisting horizontal forces from deck

Note that the horizontal stress resulting from the surcharge is distributed uniformly with depth; hence, the resultant force corresponding to the surcharge is located at the midheight of the wall. Thus, the resultant of the total thrust, reflecting the effects of the surcharge and the weight of the soil, will lie between midheight and the third-point. The location of the resultant of the total thrust is found by vector addition of the thrusts for each of the two components.

The change in force, owing to K changing from 0.4 to 1.5 with $\gamma = 0.020 \text{ MN/m}^3$, is

$$P = (1.5 - 0.4) \times 0.020 \times 6^2 / 2 = 0.40 \text{ MN/m} \quad (4.222)$$

The resultant earth pressure force acts at one-third of the height (2 m) up the abutment wall. Thus, the resistance provided at deck level is (by taking moments about the foundation)

$$R = 0.40 \times 2/6 = 0.13 \text{ MN/m} \quad (4.223)$$

Since the assumed displacement at deck level is 0.012 m, the effective stiffness K_x per metre width is

$$K_x = 0.13 / 0.012 = 11 \text{ MN/m}^2.$$

An abutment 12 m wide would provide a stiffness of 132 MN/m, that is, a resistance force of 1.6 MN for the displacement of 12 mm.

Integral bridges

Bridges constructed without any movement joints between spans or between spans and abutments are called integral bridges. Integral bridges are becoming more widespread as designers endeavour to avoid the maintenance problems that develop at joints between simply supported decks (Hambly and Nicholson, 1990). The road surfaces are continuous from one approach embankment to the other. In this case, bridge deck, piers, abutments, embankments and ground must all be considered as a single compliant system. The load distribution in an integral bridge depends on the relative stiffnesses of all the components and all the material, and structural stiffnesses must, therefore, be estimated as realistically as possible. There will be thermal movement between the abutments and the approach road, but integral bridges are likely to be most useful in circumstances where the pavement does not have to be interrupted by mechanical expansion joints. Integral bridges are generally designed with the stiffnesses and flexibilities spread throughout the structure–soil system. Expansion and contraction of an integral bridge will cause the abutment to move. This movement will be resisted by earth pressure behind the abutments, and possibly by friction underneath. Abutments should normally be designed to minimise such resistance to movement, as these horizontal forces have to be carried as compression or tension by the deck. Even small abutments are likely to be sufficient to resist the horizontal traffic loads.

The analysis of the interaction of an integral bridge with its environment has been presented by Hambly (1991) with the aid of a global model. Figure 4.80(a) shows a longitudinal section of half of the length of a bridge with a run-on slab, while a plane frame model is shown in Figure 4.80(b). The deck is continuous over the piers (on a footing of 14 m × 3 m) and is built into the abutments (on a footing of 14 m × 2 m). The run-on slabs are modelled with pin joints at the abutments. Each abutment is supported by two spring supports and restrained horizontally by lateral springs on the footing and at the centroid of passive earth pressure. Each pier is supported on two spring supports and restrained horizontally by a lateral spring and has a moment release at the top to simulate a bearing with dowel. The spring stiffnesses of the supports are calculated using Equations 4.206 to 4.209. For example, under the action of short-term braking forces, the soil might have a Young modulus $E = 40 \text{ MPa}$. With Poisson's ratio γ , of the order of 0.3–0.5, stiffnesses can be calculated for springs under the footings of the piers as

2 vertical springs: $K = 0.5 \times 1.5EA^{0.5} = 200 \text{ MN/m}$ (4.224)

at spacing $l = 0.82 \times 14^{0.25} \times 3^{0.75} = 3.6 \text{ m}$ (4.225)

horizontal springs: $K_x = EA^{0.5} = 260 \text{ MN/m}$ (4.226)

In the same way, for springs under the footings of the abutments:

2 vertical springs: $K = 0.5 \times 1.5EA^{0.5} = 160 \text{ MN/m}$ (4.227)

at spacing $l = 0.82 \times 14^{0.25} \times 2^{0.75} = 2.7 \text{ m}$ (4.228)

horizontal springs: $K_x = EA^{0.5} = 210 \text{ MN/m}$ (4.229)

It is found from calculations under ultimate temperature loads that the displacement just exceeds 0.01 m and then the horizontal springs on the abutment footings are replaced by the limiting forces $F = 2.1 \text{ MN}$ resisting sliding. It is easy to check by using two vertical springs under each footing that no spring goes into tension under combined weight and temperature loading. It is shown that the total range of thermal movement is about 40 mm, or 20 mm at each abutment. No special treatment of shrinkage effects is required for the bridge (Nicholson, 1994).

More detailed treatment of integral bridges may be found in the publication by the UK Steel Construction Institute (Biddle *et al.*, 1997), which provides an introduction to the concepts

relating to integral bridges and illustrates ways in which the ordinary composite beam-and-slab deck bridge can be adapted to become an integral bridge. Worked examples in a companion publication (Way and Yandzio, 1997) illustrate many of the design aspects of integral bridges.

Structural dynamics

Equation of motion

The essential physical properties of a linearly elastic structural system subjected to external dynamic loading are its mass, stiffness properties and energy absorption capability or damping. The principle of dynamic analysis may be illustrated by considering a simple single-storey structure, as shown in Figure 4.81. The structure is subjected to a time-varying force $f(t)$. The term k is the spring constant, which relates the lateral storey deflection x to the storey shear force, and the dashpot relates the damping force to the velocity by a damping coefficient c . If the mass, m , is assumed to be concentrated at the beam, the structure becomes a single degree of freedom (SDOF) system. The equation of motion of the system may be written as

$$m\ddot{x} + c\dot{x} + kx = f(t) \quad (4.230)$$

Solutions to Equation 4.230 can give an insight into the behaviour of the structure under a dynamic situation.

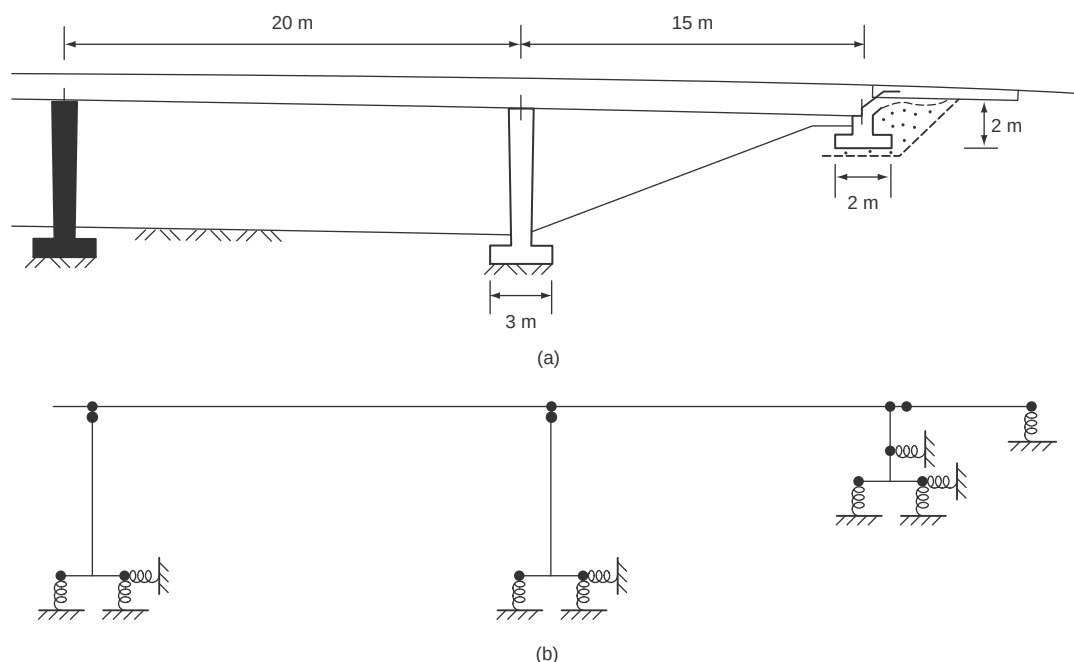


Figure 4.80 Longitudinal section and plane frame model of an integral bridge

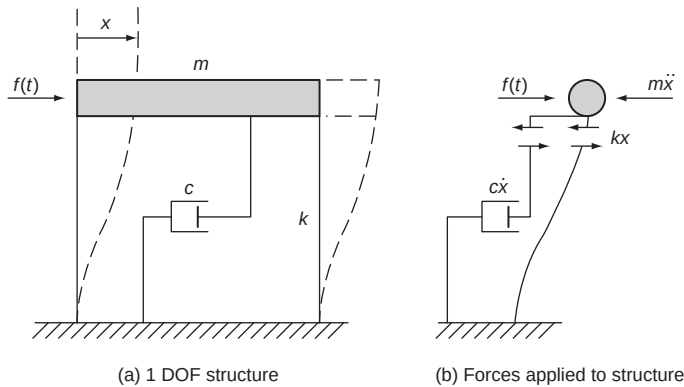


Figure 4.81 A single-storey structure – principle of dynamic analysis

Free vibration

In this case, the system is set to motion and allowed to vibrate in the absence of an applied force $f(t)$. Letting $f(t) = 0$, Equation 4.230 becomes

$$m\ddot{x} + c\dot{x} + kx = 0 \quad (4.231)$$

Dividing Equation 4.231 by the mass m :

$$\ddot{x} + 2\zeta\omega\dot{x} + \omega^2 x = 0 \quad (4.232)$$

where

$$2\zeta\omega = \frac{c}{m} \quad \text{and} \quad \omega^2 = \frac{k}{m} \quad (4.233)$$

The solution to Equation 4.233 depends on whether the vibration is damped or undamped.

Case 1: Undamped free vibration

In this case, $c=0$, and the solution to the equation of motion may be written as

$$x = A \sin \omega t + B \cos \omega t \quad (4.234)$$

where $\omega = \sqrt{k/m}$ is the circular frequency and A and B are constants that can be determined by the initial boundary conditions. In the absence of external forces and damping, the system will vibrate indefinitely in a repeated cycle of vibration with an amplitude of

$$\sqrt{A^2 + B^2} \quad (4.235)$$

and a natural frequency of

$$f = \frac{\omega}{2\pi} \quad (4.236)$$

The natural period is

$$T = \frac{2\pi}{\omega} = \frac{1}{f} \quad (4.237)$$

The undamped free vibration motion as described by Equation 4.234 is shown in Figure 4.82.

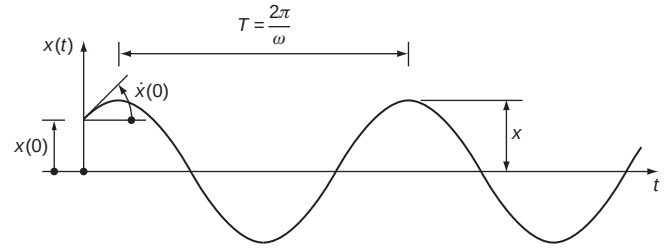


Figure 4.82 Undamped free vibration motion

Case 2: Damped free vibration

If the system is not subjected to applied force and damping is present, the corresponding solution becomes

$$x = A \exp(\lambda_1 t) + B \exp(\lambda_2 t) \quad (4.238)$$

where

$$\lambda_1 = \omega [-\zeta + \sqrt{\zeta^2 - 1}] \quad (4.239)$$

$$\lambda_2 = \omega [-\zeta - \sqrt{\zeta^2 - 1}] \quad (4.240)$$

The solution of Equation 4.238 changes its form with the value of the damping ratio, ζ , defined as

$$\zeta = \frac{c}{2\sqrt{mk}} \quad (4.241)$$

If $\zeta^2 < 1$, the equation of motion becomes

$$x = \exp(-\zeta\omega t)(A \cos \omega_d t + B \sin \omega_d t) \quad (4.242)$$

where ω_d is the damped angular frequency, defined as

$$\omega_d = \sqrt{(1 - \zeta^2)}\omega \quad (4.243)$$

For most building structures, ζ is very small (about 0.01) and therefore $\omega_d \approx \omega$. The system oscillates about the neutral position as the amplitude decays with time t . Figure 4.83 illustrates an example of such motion. The rate of decay is governed by the amount of damping present.

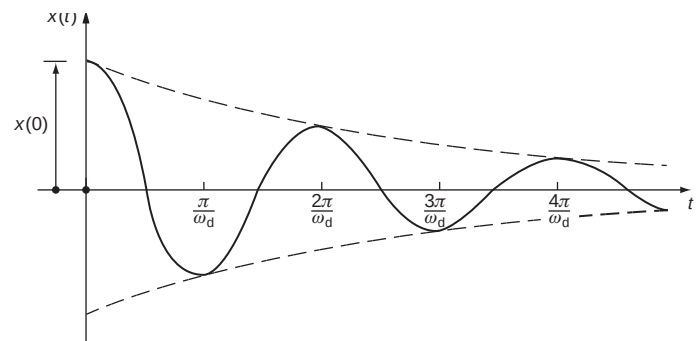


Figure 4.83 Damped free vibration motion

If the damping is large, oscillation will be prevented. This happens when $\zeta^2 > 1$ and the behaviour is generally referred to as overdamped. The motion of such behaviour is shown in Figure 4.84.

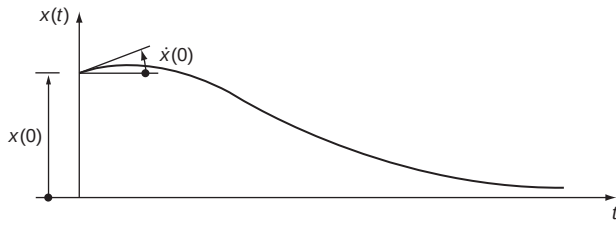


Figure 4.84 Damped free vibration motion – overdamped

Damping with $\zeta^2=1$ is called critical damping. This is the case where minimum damping is required to prevent oscillation; the critical damping coefficient is given as

$$c_{cr} = 2\sqrt{km} \quad (4.244)$$

The degree of damping in the structure is often expressed as a proportion of the critical damping value. Referring to Equations 4.241 and 4.244, we have

$$\zeta = \frac{c}{c_{cr}} \quad (4.245)$$

Forced vibration

If a structure is subjected to a sinusoidal motion such as a ground acceleration of $\ddot{x} = F \sin \omega_f t$, it will oscillate and after some time the motion of the structure will reach a steady state. For example, the equation of motion owing to the ground acceleration (from Equation 4.232) is

$$\ddot{x} + 2\zeta\omega\dot{x} + \omega^2 x = -F \sin \omega_f t \quad (4.246)$$

The solution to this equation consists of two parts; the complementary solution given by Equation 4.234 and the particular solution. If the system is damped, oscillation corresponding to the complementary solution will decay with time. After some time, the motion will reach a steady state, and the system will vibrate at a constant amplitude and frequency. This motion, which is called forced vibration, is described by the particular solution, expressed as

$$x = C_1 \sin \omega_f t + C_2 \cos \omega_f t \quad (4.247)$$

It can be observed that the steady forced vibration occurs at the frequency of the exciting force, ω_f , not the natural frequency of the structure, ω .

Substituting Equation 4.247 into 4.246, the displacement amplitude can be shown to be

$$X = -\frac{F}{\omega^2} \frac{1}{\sqrt{\left\{1 - \left(\frac{\omega_f}{\omega}\right)^2\right\}^2 + \left(\frac{2\zeta\omega_f}{\omega}\right)^2}} \quad (4.248)$$

The term $-F/\omega^2$ is the static displacement caused by the force owing to the inertia force. The ratio of the response amplitude relative to the static displacement $-F/\omega^2$ is called the dynamic displacement amplification factor, D , given as

$$D = \frac{1}{\sqrt{\left\{1 - \left(\frac{\omega_f}{\omega}\right)^2\right\}^2 + \left(\frac{2\zeta\omega_f}{\omega}\right)^2}} \quad (4.249)$$

The variation of the magnification factor with the frequency ratio ω_f/ω and damping ratio ζ is shown in Figure 4.85.

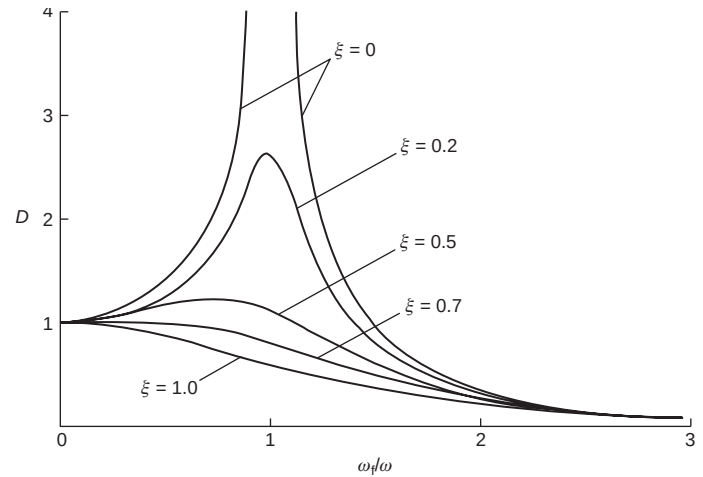


Figure 4.85 Variation of amplification factor

When the dynamic force is applied at a frequency much smaller than the natural frequency of the system ($\omega_f/\omega < 1$), the response is quasistatic. The response is proportional to the stiffness of the structure, and the displacement amplitude is close to the static deflection.

When the force is applied at a frequency much greater than the natural frequency ($\omega_f/\omega > 1$), the response is proportional to the mass of the structure. The displacement amplitude is less than the static deflection ($D < 1$).

When the force is applied at a frequency close to the natural frequency, the displacement amplitude increases significantly. The condition at which $\omega_f/\omega = 1$ is called resonance.

Similarly, the ratio of the acceleration response relative to the ground acceleration may be expressed as

$$D_a = \left| \frac{\ddot{x} + \ddot{x}_g}{\ddot{x}_g} \right| = \sqrt{\frac{1 + \left(\frac{2\zeta\omega_f}{\omega}\right)^2}{\left\{1 - \left(\frac{\omega_f}{\omega}\right)^2\right\}^2 + \left(\frac{2\zeta\omega_f}{\omega}\right)^2}} \quad (4.250)$$

where D_a is called the dynamic acceleration magnification factor.

Response to suddenly applied load

Consider the spring mass damper system of which a load P_o is applied suddenly. The differential equation is given by

$$M\ddot{x} + c\dot{x} + kx = P_o \quad (4.251)$$

If the system is started at rest, the equation of motion is

$$x = \frac{P_o}{k} \left[1 - \exp(-\zeta\omega t) \left\{ \cos \omega_d t + \frac{\zeta\omega}{\omega_d} \sin \omega_d t \right\} \right] \quad (4.252)$$

If the system is undamped, then $\zeta=0$ and $\omega_d=\omega$, and

$$x = \frac{P_o}{k} [1 - \cos \omega_d t] \quad (4.253)$$

The maximum displacement is $2(P_o/k)$ corresponding to $\cos \omega_d t = -1$. Since P_o/k is the maximum static displacement, the dynamic amplification factor is equal to 2. The presence of damping would naturally reduce the dynamic amplification factor and the force in the system.

Response to time-varying loads

Some forces and ground motions that are encountered in practice are rather complex in nature. In general, numerical analysis is required to predict the response of such effects, and the finite-element method is one of the most common techniques to be employed in solving such problems.

The evaluation of responses owing to time-varying loads can be conducted using the piecewise exact method. In this method, the loading history is divided into small time intervals. Between these points, it is assumed that the slope of the load curve remains constant. The entire load history is represented by a piecewise linear curve, and the error in this approach can be minimised by reducing the length of the time steps. A description of this procedure is given in Nicholson (1994).

Other techniques employed include Fourier analysis of the force function followed by solution for Fourier components in the frequency domain. For random forces, random vibration theory and spectrum analysis may be used (Dowrick, 1988; Warburton, 1976).

Multiple degree systems

In multiple degree systems, an independent differential equation of motion can be written for each degree of freedom. The nodal equations of a multiple degree system of n degrees of freedom may be written as

$$[M]\{\ddot{x}\} + [c]\{\dot{x}\} + [k]\{x\} = \{F(t)\} \quad (4.254)$$

where $[M]$ is a symmetrical $n \times n$ matrix of mass, $[c]$ is a symmetrical $n \times n$ matrix of damping coefficient and $\{F(t)\}$ is the force vector, which is zero in the case of free vibration.

Consider a system under free vibration without damping; the general solution of Equation 4.254 is assumed in the form

$$\begin{Bmatrix} x_1 \\ x_2 \\ \vdots \\ x_n \end{Bmatrix} = \begin{bmatrix} \cos(\omega t - \phi) & 0 & 0 & 0 \\ 0 & \cos(\omega t - \phi) & 0 & 0 \\ \vdots & \vdots & \vdots & \vdots \\ 0 & 0 & 0 & \cos(\omega t - \phi) \end{bmatrix} \begin{Bmatrix} C_1 \\ C_2 \\ \vdots \\ C_n \end{Bmatrix} \quad (4.255)$$

where angular frequency ω and phase angle ϕ are common to all x_i . In this assumed solution, ϕ and C_1, C_2, C_n are the constants to be determined from the initial boundary conditions of the motion and ω is a characteristic value (eigenvalue) of the system.

Substituting Equation 4.255 into Equation 4.254 yields

$$\begin{bmatrix} k_{11} - m_{11}\omega^2 & k_{12} - m_{12}\omega^2 & \cdots & k_{1n} - m_{1n}\omega^2 \\ k_{21} - m_{21}\omega^2 & k_{22} - m_{22}\omega^2 & \cdots & k_{2n} - m_{2n}\omega^2 \\ \vdots & \vdots & \ddots & \vdots \\ k_{n1} - m_{n1}\omega^2 & k_{n2} - m_{n2}\omega^2 & \cdots & k_{nn} - m_{nn}\omega^2 \end{bmatrix} \begin{Bmatrix} C_1 \\ C_2 \\ \vdots \\ C_n \end{Bmatrix} \cos(\omega t - \phi) = \begin{Bmatrix} 0 \\ 0 \\ \vdots \\ 0 \end{Bmatrix} \quad (4.256)$$

or

$$[[k] - \omega^2[m]] \{C\} = \{0\} \quad (4.257)$$

where $[k]$ and $[m]$ are $n \times n$ matrices, ω^2 and $\cos(\omega t - \phi)$ are scalars and $\{C\}$ is the amplitude vector. For a non-trivial solution, $\cos(\omega t - \phi) = 0$; thus, solution of Equation 4.257 requires the determinant of $[[k] - \omega^2[m]] = 0$. The expansion of the determinant yields a polynomial of n th degree as a function of ω^2 , the n roots of which are the *eigenvalues* $\omega_1, \omega_2, \dots, \omega_n$.

If the eigenvalue ω for a normal mode is substituted in Equation 4.257, the amplitude vector $\{C\}$ for that mode can be obtained; $\{C_1\}, \{C_2\}, \{C_3\}, \dots, \{C_n\}$ are therefore called the eigenvectors, the absolute values of which must be determined through initial boundary conditions. The resulting motion is a sum of n harmonic motions, each governed by the respective natural frequency ω , written as

$$\{x\} = \sum_{i=1}^n \{C_i\} \cos(\omega_i t - \phi_i) \quad (4.258)$$

Distributed mass systems

Although many structures may be approximated by lumped mass systems; in practice, all structures are distributed mass systems consisting of infinite numbers of particles. Consequently, if the motion is repetitive, the structure has an infinite number of natural frequency and mode shapes. The analysis of a distributed-parameter system is entirely equivalent to that of a discrete system, once the mode shapes and frequencies have been determined, because in both cases the amplitudes of the modal response components are used as generalised coordinates in defining the response of the structure.

In principle, an infinite number of these coordinates are available for a distributed-parameter system, but in practice only a few modes, usually those of smaller frequencies, will provide a significant contribution to the overall response. Thus, the problem of a distributed-parameter system can be converted to a discrete system form in which only a limited number of modal coordinates is used to describe the response.

Flexural vibration of beams

The motion of the distributed mass system is best illustrated by a classical example of a uniform beam of span length L and flexural rigidity EI and a self-weight of m per unit length, as shown in Figure 4.86(a). The beam is free to vibrate under its self-weight. From Figure 4.86(b), dynamic equilibrium of a small beam segment of length dx requires

$$\frac{\partial V}{\partial x} dx = m dx \frac{\partial^2 y}{\partial t^2} \quad (4.259)$$

in which

$$\frac{\partial^2 V}{\partial x^2} = \frac{M}{EI} \quad (4.260)$$

and

$$V = -\frac{\partial M}{\partial x}, \quad \frac{\partial V}{\partial x} = -\frac{\partial^2 M}{\partial x^2} \quad (4.261)$$

Substituting these equations into Equation 4.259 leads to the equation of motion of the flexural beam:

$$\frac{\partial^4 y}{\partial x^4} + \frac{m}{EI} \frac{\partial^2 y}{\partial t^2} = 0 \quad (4.262)$$

This equation can be solved for beams with given sets of boundary conditions. The solution consists of a family of vibration modes with corresponding natural frequencies. Standard results are available in Figure 4.87 to compute the natural frequencies of uniform flexural beams of different supporting conditions. Methods are also available for dynamic analysis of continuous beams (Clough and Penzien, 1993).

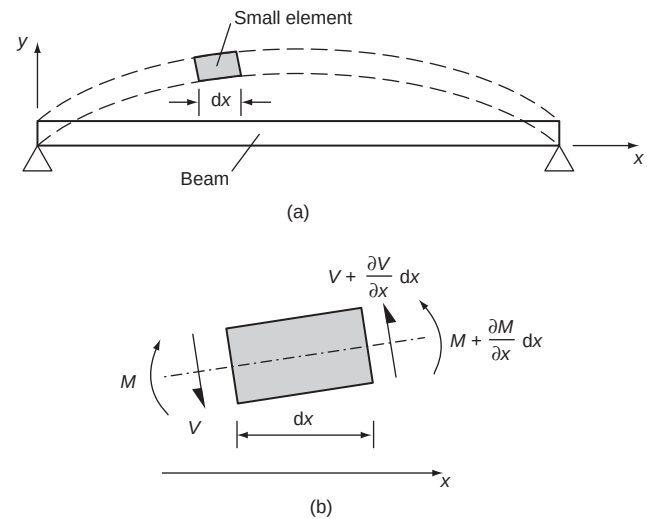


Figure 4.86 Motion of distributed mass system

Frequencies and mode shapes of beams in flexural vibration

$t_n = \frac{k_n}{2\pi} \sqrt{\frac{EI}{mL^4}} \text{ Hz}$ $n = 1, 2, 3$		$L = \text{length (m)}$ $EI = \text{flexural rigidity (Nm}^2\text{)}$ $M = \text{mass per unit length (kg/m)}$	
Boundary conditions	K_n : $n = 1, 2, 3$	Mode shape $y_n \left(\frac{x}{L} \right)$	A_n : $n = 1, 2, 3$
Pinned–pinned 	$(n\pi)^2$	$\sin \frac{n\pi x}{L}$	
Fixed–fixed 	22.37 61.67 120.90 199.86 298.55 $(2n+1)^2 \frac{\pi^2}{4}$: $n > 5$	$\cos h \frac{+K_n x}{L} - \cos \frac{+K_n x}{L}$ $-A_n \left(\sin h \frac{+K_n x}{L} - \sin \frac{+K_n x}{L} \right)$	0.98250 1.00078 0.99997 1.00000 0.99999 1.0; $n > 5$
Fixed–pinned 	15.42 49.96 104.25 178.27 272.03 $(4n+1)^2 \frac{\pi^2}{4}$: $n > 5$	$\cos h \frac{+K_n x}{L} - \cos \frac{+K_n x}{L}$ $-A_n \left(\sin h \frac{+K_n x}{L} - \sin \frac{+K_n x}{L} \right)$	1.00078 1.00000 1.0; $n > 3$
Cantilever 	3.52 22.03 61.69 120.90 199.86 $(2n-1)^2 \frac{\pi^2}{4}$: $n > 5$	$\cos h \frac{+K_n x}{L} - \cos \frac{+K_n x}{L}$ $-A_n \left(\sin h \frac{+K_n x}{L} - \sin \frac{+K_n x}{L} \right)$	0.73410 1.01847 0.99922 1.00003 1.0; $n > 4$

Figure 4.87 Frequencies and mode shapes of beams in flexural vibration

Shear vibration of beams

Beams can deform by flexure or shear. Flexural deformation normally dominates the deformation of slender beams. Shear deformation is important for short beams or in higher modes of slender beams. Figure 4.88 gives the natural frequencies of uniform beams in shear, neglecting flexural deformation. The natural frequencies of these beams are inversely proportional to the beam length L rather than L^2 , and the frequencies increase linearly with the mode number.

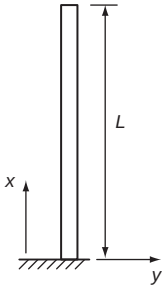
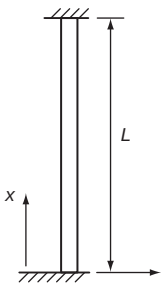
$f_n = \frac{K_n}{2\pi} \sqrt{\frac{KG}{\rho L^2}} \text{ HZ}$ $L = \text{length (m)}$ $K = \text{shear coefficient (Cowper, 1966)}$ $G = \text{shear modulus} = E/[2(1 + \nu)]$ $\rho = \text{mass density}$		
Boundary conditions	$K_n: n = 1, 2, 3 \dots$	Mode shape $y_n \left(\frac{x}{L}\right)$
Fixed-free 	$n\pi: n = 1, 2, 3 \dots$	$\cos \frac{n\pi x}{L}; n = 1, 2, 3 \dots$
Fixed-fixed 	$n\pi: n = 1, 2, 3 \dots$	$\sin \frac{n\pi x}{L}; n = 1, 2, 3 \dots$

Figure 4.88 Frequencies and mode shapes of beams in shear vibration

Combined shear and flexure

The transverse deformation of real beams is the sum of flexure and shear deformations. In general, numerical solutions are required to incorporate both the shear and flexural deformation in predicting the natural frequency of beams. The following simplified formula may be used to estimate the beam's frequency in respect of beams with comparable shear and flexural deformations:

$$\frac{1}{f^2} = \frac{1}{f_f^2} + \frac{1}{f_s^2} \quad (4.263)$$

in which f is the fundamental frequency of the beam, and f_f and f_s are the fundamental frequencies predicted by the flexure and shear beam theory, respectively (Rutenberg, 1975).

Damping

Damping is found to increase with the increasing amplitude of vibration. It arises from the dissipation of energy during vibration. The mechanisms that contribute to energy dissipation are material damping, friction at interfaces between components and energy dissipation owing to the foundations interacting with soil, among others. Material damping arises from the friction at bolted connections and frictional interaction between structural and non-structural elements, such as partitions and cladding.

The amount of damping in a building can never be predicted precisely, and design values are generally derived based on dynamic measurements of structures of a corresponding type. Damping can be measured based on the rate of decay of free vibration following an impact, by spectral methods based on analysis of response to wind loading or by force excitation using a mechanical vibrator at varying frequencies to establish the shape of the steady-state resonance curve. However, these methods may not be easily carried out if several modes of vibration close in frequency are presented.

Numerical analysis

Many less complex dynamic problems can be solved without much difficulty by hand methods. For more complex problems, such as the determination of natural frequencies of complex structures, the calculation of response to time-varying loads, and response spectrum analysis to determine seismic forces, may require numerical analysis. The finite-element method has been shown to be a versatile technique for this purpose.

The global equations of an undamped motion, in matrix form, may be written as

$$[M] \{\ddot{x}\} + [k] \{\dot{x}\} = \{F(t)\} \quad (4.264)$$

where

$$[K] = \sum_{i=1}^n [k_i]; \quad [M] = \sum_{i=1}^n [M_i]; \quad [F] = \sum_{i=1}^n [f_i] \quad (4.265)$$

are the global stiffness, mass and force matrices, respectively, and $[k_i]$, $[M_i]$ and $\{f_i\}$ are the stiffness, mass and force of the i th element, respectively. The elements are assembled using the direct stiffness method to obtain the global equations, such that intermediate continuity of displacements is satisfied at common nodes and, in addition, inter-element continuity of acceleration is also satisfied.

Equation 4.264 is the matrix equations discretised in space. To obtain a solution of the equation, discretisation in time is also necessary. The general method used is called direct integration. There are two methods for direct integration: *implicit* and *explicit*. The second, and simplest method, is an explicit method known as the central difference method (Biggs, 1964). The first method, more sophisticated but more versatile, is an implicit method known as the Newmark method (Newmark, 1959). Other integration methods are also available in the works

by Bathe (Bathe, 1982).

The natural frequencies are determined by solving Equation 4.264 in the absence of force $F(t)$, as

$$[M] \{\ddot{x}\} = [K] \{x\} = 0 \quad (4.266)$$

The standard solution for $x(t)$ is given by the harmonic equation in time:

$$\{x(t)\} = \{X\} e^{i\omega t} \quad (4.267)$$

where $\{X\}$ is the part of the nodal displacement matrix called the natural modes, which are assumed to be independent of time, i is the imaginary number and ω is the natural frequency.

Differentiating Equation 4.267 twice with respect to time gives

$$\ddot{x}(t) = \{X\} (-\omega^2) e^{i\omega t} \quad (4.268)$$

Substitution of Equations 4.267 and 4.268 into Equation 4.266 yields

$$e^{i\omega t} ([K] - \omega^2 [M]) \{x\} = 0 \quad (4.269)$$

Since $e^{i\omega t}$ is not zero, this gives

$$([K] - \omega^2 [M]) \{X\} = 0 \quad (4.270)$$

Equation 4.270 is a set of linear homogeneous equations in terms of displacement mode $\{X\}$. It has a nontrivial solution if the determinant of the coefficient matrix $\{X\}$ is non-zero, that is

$$[K] - \omega^2 [M] = 0 \quad (4.271)$$

In general, Equation 4.271 is a set of n algebraic equations, where n is the number of degrees of freedom associated with the problem.

Seismic bridge behaviour

It is imperative for a bridge designer to understand the important principles of structural dynamics in order to apply the appropriate modelling and analysis tool to a bridge seismic response problem. A detailed treatment of this subject is given in *Pile Foundation Analysis and Design* by Priestley *et al.* (1996); a brief account of the subject extracted from this book is given in the following sections.

The dynamic excitation and response of a bridge subjected to earthquake ground motion in the form of ground acceleration $\ddot{u}_g(t)$ can best be explained by means of a single degree of freedom model of a bridge structure. This simplified model is applicable only if the bridge is straight, consisting of a large number of equal spans and piers of equal height or stiffness. It is assumed that all piers are exposed to the same coherent earthquake ground acceleration $\ddot{u}_g(t)$ perpendicular to the bridge axis and that the superstructure moves as a rigid body. In this model, the seismic mass m is assumed to be lumped at the top of a single-column bent at a height H above ground level. The cantilever stiffness of the assumed massless bridge pier can be expressed by k , which is the force required to produce a

unit displacement at the centre of mass relative to the column base. Furthermore, if damping of the bridge system can be expressed in the form of viscous damping, the characteristic damping force required to resist a unit velocity at the point of mass can be expressed as c . The dynamic response of the bridge model can be expressed in the form

$$u_t = u_s + u_g \quad (4.272)$$

in which u_t is the total displacement, u_s is the structural displacement of the cantilever pier and u_g is the ground displacement relative to an absolute frame of reference.

Equilibrium of all forces acting on the system at this single displacement degree of freedom requires that the inertia force $f_i(t) = m \ddot{u}_t(t)$, the viscous damping force $f_d(t) = c \dot{u}_s(t)$ and the restoring force $f_s(t) = k u_s(t)$ can be combined in a single equilibrium equation at the unknown displacement degree of freedom, in the form

$$m(\ddot{u}_g + \ddot{u}_s) + c \dot{u}_s + k u_s = 0 \quad (4.273)$$

or

$$m \ddot{u}_s + c \dot{u}_s + k u_s = m \ddot{u}_g \quad (4.274)$$

which represents the equation of motion of the single degree of freedom bridge model under transverse earthquake ground accelerations.

Bridge dynamic response characteristics

Independent of the specific dynamic input, each bridge system is represented within the elastic range by dynamic response modes typically referred to as the natural modes of vibration, characterised by independent mode shapes Φ_i with corresponding periods of vibration T_i . The number of characteristic mode shapes and vibration periods of a bridge model depend on the selected number of dynamic degrees of freedom, defined during the analytical model discretisation. In the case of a prototype bridge, an infinite number of vibration modes are featured. Conversely, a selected finite number of degrees of freedom and associated modes of vibration are featured in the analytical model of a bridge. However, it is possible to capture the governing dynamic response of a bridge by the contribution of a limited number of vibration modes. A good indication of the dynamic response of a bridge can often be obtained from the fundamental or lowest mode of vibration. Single degree of freedom models, therefore, serve as an invaluable tool for design office use.

Modelling of bridge structures

The mathematical formulation of the bridge behaviour satisfying a particular assessment or design requirement for a quantitative response determination should be simple. The model has to capture the physical and mechanical interactions of earthquake input and structure response. It should be capable of describing adequately the geometric domain, the seismic mass, the connection and boundary conditions and the loading

of the prototype. For each element in the model, member end forces and deformation relationships are defined by kinematic, constitutive and static relationships. Also, the correct seismic weight or mass associated with each degree of freedom also needs to be determined. It is essential to characterise the soil–structure interface and loading history with the earthquake loading to the bridge generated by ground motion. In the case of longer bridges, consisting of a number of spans and frames, attention should be paid to the necessity and validity of modelling the entire bridge in a global model. The model should be capable of capturing the effects of curves in plan and elevation, highly skewed supports, ramps and interaction between frames, in order to meet the seismic demand assessment. Some specific modelling issues in respect of bridge components are given next.

Superstructure

Most bridge structures can be considered as linear structures, where the span length L between bents is larger than the width B or depth D of the superstructure. In many cases, the bridge superstructure can be assumed to move as a rigid body under seismic loads. The entire modelling problem is reduced to the stiffness modelling of the bents, with geometric constraints simulating the rigid superstructure. In long and narrow bridges and interchange connectors, for example, the superstructure cannot be considered rigid. In such cases, the superstructure can be modelled as a grillage of beam elements or as a spine with beam elements following the centre of gravity of the cross-section along the length of the bridge. Equivalent member properties for the spine or beam elements representing the overall effective superstructure stiffness can be obtained by using the methods described in the section on grillage analysis. For superstructures carrying wide roadways, the spine model may be erroneous, particularly when combinations of earthquake forces with gravity loads and other loads need to be investigated.

Single-column bents

In the seismic response analysis of bridges, the bents or supports are the critical structural elements that provide gravity load and earthquake force transfer to the ground and ground motion input to the bridge superstructure. For a generic single-column bent, different discretisations must be considered, depending on the bent geometry and the expected seismic response. For an essentially elastic response and a prismatic column, a single-column element connected at nodes 2 and 3 at the superstructure soffit and the top of the footing is sufficient to model the seismic response. For a tapered or flared single-column bents or bents, where column mass effects may be important, a stepped discretisation may be used. In the case of bridges with very tall or slender columns, the combined influence of gravity load and displacement can be significant, even though the displacement may be small compared with the overall bridge dimensions. In these cases, only a second-order or P -delta effect needs to be considered by evaluating

equilibrium in the deformed configuration. The P -delta effect in the analytical modelling can be accounted for by means of reduced capacity for lateral loads and the yield and ultimate deformation limit states, together with bending moment contributions from the lateral earthquake force F and the vertical load P owing to gravity.

Multicolumn bents

In multicolumn bents, the framing action or coupling between columns contributes to the seismic response in terms of stiffness, capacity and axial load levels in the various frame members during cyclic earthquake loading. In the analytical model, all of these effects can be incorporated in a planar frame model along the bent axis, consisting of beam and column elements with appropriate effective member properties. Special elements for joints and boundary conditions may be provided if required.

Foundations

Foundations of bridge piers are the structural elements at or below ground level that support the pier and provide vertical, lateral and rotational resistance to gravity loads and seismic forces. This resistance depends on the type and geometry of the foundation, the characteristics of the surrounding soil and the interaction between soil and structure. Spread or pile footings are considered to be rigid bodies. It is, therefore, possible to model at a single point with boundary springs at the bottom of the column or pier model at the end of the effective length extension link into the footing. Only a vertical, a translational and a rotational boundary spring need to be defined for a two-dimensional column, bent or frame model. In a three-dimensional model, however, six springs, one for each possible degree of freedom at the column base, are required. For a pile shaft, the flexibility of the pile and the surrounding soil should be modelled in a seismic demand analysis.

Abutments

In the modelling of abutments, assumptions made for abutment stiffnesses and capacities, as well as damping of the surrounding soil mass, can have a significant effect on the analytical dynamic response characteristics. This is particularly true in shorter bridge structures. Abutments are, generally, massive structures; they interact with large soil mass, exhibit significantly higher stiffness values and thus attract higher seismic forces. Abutment capacity and stiffness characteristics are often based on empirical relationships.

Methods of analysis

Tools for the seismic response quantification of bridges range from simple linear elastic static analyses in the form of hand calculations to three-dimensional dynamic non-linear response history analyses for global bridge systems with large numbers of degrees of freedom.

Static or quasistatic analysis

It is most convenient, in many cases of seismic analysis, to apply the seismic actions in the form of an equivalent static force to the bridge model, particularly when seismic force distributions or likely deformation modes can be estimated. The magnitude of the equivalent static force is either specified as the absolute acceleration coefficient a_s or obtained from the design- or assessment-level earthquake in the form of an expected peak ground acceleration, which must then be amplified by an appropriate spectral acceleration response factor for specific site or soil characteristics. The acceleration coefficient a_s is then multiplied by the total seismic weight W_s to determine the earthquake force $E = W_s a_s$ lumped at the centroid of seismic mass, or distributed proportional to the expected fundamental mode shape. This force and distribution pattern is then applied to the structure, either as a static monotonic force or in the form of a quasistatic stepwise monotonic or cyclic force.

Response spectrum analyses

Modal spectral analyses are demand analysis tools for determining maximum response quantities from the spectrum of a given ground motion or from smoothed design spectra. Analysis models used for modal spectral analysis are linear elastic models based on effective stiffness properties and on assumed equivalent viscous damping ratios. Response spectrum analyses can be performed (a) for bridge systems expected to perform essentially in the linear elastic range, based on cracked or effective stiffness properties; (b) for the inelastic response of a bridge system, where the equivalent response is linearised to the initial effective stiffness and subsequently modified by means of equal energy or equal displacement principles and (c) for substitute structure analyses.

Time-history analyses

This analysis utilises a particular earthquake ground motion input and quantifies the bridge response for the earthquake input in the form of time histories of the various response quantities. Like the static or quasistatic analyses, models with linear elastic material behaviour, non-linear cyclic material characteristics and geometric non-linearities, such as the P -delta effect or full non-linear geometry, can be analysed. However, in addition to two or three spatial dimensions, an additional dimension, t , now has to be accounted for. Step-by-step integration in the time domain, superposition of normalised modal time histories in the time domain or evaluation of frequency-dependent response contributions with transformation to and superposition in the time domain can be applied for the earthquake time-history analysis of bridge models. Of the three methods, stepwise linearised modal time-history analyses have been successfully employed in the non-linear dynamic response analysis of global bridge models.

Concluding remarks

A comprehensive overview of the basic concepts governing the analysis of bridge structures is provided in this chapter. The background theory for the analysis of beams, trusses, plated structures, bracing systems, and so on, as well as the commonly used analytical tools, such as finite-element methods, finite-strip methods and grillage analysis, are all covered in sufficient detail. The assumptions implicit in the simplified theoretical models, and their limitations, are discussed. Charts and graphs are included, where appropriate, and are aimed at providing rapid solutions for typical problems. The chapter ends with a short treatment of soil–structure interaction and an introduction to structural dynamics.

The goal is to cater to the needs of young engineers in the early years of their careers; it is hoped that the text will also assist more experienced engineers to refresh their memories, as well as providing the background information to the provisions in the codes of practice. The code specifications discussed in this chapter are very limited, owing to space limitations. However, some code specifications are included in later chapters.

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Chapter 5

Bridge dynamics

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Most bridge engineers rarely undertake any form of dynamic analysis, but instead rely on codified pseudostatic load factors and semi-empirical rules to ensure satisfactory dynamic structural response. As bridge engineers and architects employ ever more innovative structural forms and use of materials there is an increasing need to understand and control dynamic response. Irrespective of this trend, it is useful for every practising bridge engineer to understand the fundamental concepts behind dynamics and to be aware of the analysis methodologies that are available. This chapter is a practical guide containing sufficient theory to provide grounding in the subject, supplemented with plenty of practical insights and recommendations for the practising engineer. There is a wealth of good literature on the subject; it is not the intent of the chapter to repeat this material but to put it into a readily digestible context that will appeal to the reader. Extensive references to more detailed texts are provided.

Introduction

The effects of dynamic loading on bridges are often addressed through the use of pseudostatic impact factors. As a result, many bridge engineers are unfamiliar with the principles and application of dynamic analysis. As society places increased demands on engineers to design bridges with improved economy, aesthetic appeal and in-service performance, the need for more rigorous dynamic assessment will undoubtedly increase. In this respect, it is notable that the Eurocodes provide more detailed guidance on evaluating bridge dynamic performance than was previously available in the British Standards. This chapter builds on the information contained in the previous chapter on structural analysis, considering both the principles of structural dynamics and their specific application to the most common forms of vibration encountered in the design of bridges.

Principles of structural dynamics

It is necessary to understand some of the fundamental concepts of structural dynamics before exploring the specific applications that are most relevant to the bridge engineer. Chapter 4 provides a comprehensive review of the principles and techniques of structural analysis; that discussion is complementary to a treatment of the more general case of dynamic equilibrium that it is at the heart of this chapter.

Dynamic equilibrium

Force equilibrium within any structure of whatever complexity can be described by the following relationship based on the physical laws first proposed by Newton:

$$F(t) = F(t)_I + F(t)_D + F(t)_S \quad (5.1)$$

where the force vectors at time t are:

- $F(t)$ vector of externally applied forces
- $F(t)_I$ vector of inertia forces
- $F(t)_D$ vector of energy dissipation or damping forces
- $F(t)_S$ vector of internal forces within the structure.

All physical structures behave dynamically when subjected to externally applied loads or displacements. As such, they obey Newton's second law in that the inertia forces are equal to the mass multiplied by the acceleration:

$$F(t)_I = M\ddot{x}(t) \quad (5.2)$$

Where loads are applied very slowly, as in the case of self-weight or superimposed loads, the inertia forces can be neglected and the equation of dynamic equilibrium reduces to its static form. Thus, static analysis is simply a specialised treatment of dynamic equilibrium in which inertial and damping forces are ignored.

Equation 5.1 is equally valid for both linear and non-linear systems, as it describes a fundamental state of energy conservation. Where it is appropriate to make an approximation of linear structural behaviour, the equation of dynamic equilibrium can be rewritten, as

$$F(t) = M\ddot{x}(t) + C\dot{x}(t) + Kx(t) \quad (5.3)$$

in which M is the mass matrix, C is the viscous damping matrix and K is the static stiffness matrix of the structural system under consideration. The time dependant vectors $\ddot{x}(t)$, $\dot{x}(t)$ and $x(t)$ are the structural acceleration, velocity and displacement, respectively. The solution of Equation 5.3 provides the engineer with insights into the dynamic behaviour of the structure.

Free vibration of structures

When no external loads are applied to a structure that is in motion, it is said to be in a state of free vibration and Equation 5.3 can be rewritten as

$$M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = 0 \quad (5.4)$$

The motion of any point within a structure when vibrating in one of its natural modes is very close to simple harmonic motion (SHM) and can be described by

$$x(t) = x_0 \sin(\omega_n t) \quad (5.5)$$

$$\dot{x}(t) = x_0 \omega_n \cos(\omega_n t) \quad (5.6)$$

$$\ddot{x}(t) = -x_0 \omega_n^2 \sin(\omega_n t) \quad (5.7)$$

where x_0 is the maximum amplitude of vibration and ω_n is the angular frequency of vibration in radians per second.

This SHM can be most easily visualised as an equivalent lumped mass–spring system where a mass m is suspended from a linear elastic spring with stiffness k . The solution to Equation 5.4 depends on whether there is damping within the system. Where no damping is present, the equation of motion is rewritten as

$$M\ddot{x}(t) = -Kx(t) \quad (5.8)$$

and substitution of the expressions for $\ddot{x}(t)$ and $x(t)$ given in Equations 5.5 and 5.7 yields

$$M\omega_n^2 = K \quad (5.9)$$

Hence, the angular frequency is

$$\omega_n = \sqrt{KM} \quad (5.10)$$

and

$$f = \frac{1}{2\pi} \sqrt{KM} \quad (5.11)$$

where f is the frequency in cycles per second, or hertz, as is most commonly stated.

Since most bridge systems mainly respond in their fundamental or first mode of vibration, this relationship is especially useful as an initial means to ascertain the dominant natural frequency of a structure. From Equation 5.11, it is clear that to undertake this calculation it is only necessary to obtain the equivalent spring stiffness of the structure and the magnitude of the corresponding vibrating mass. The equivalent spring stiffness, K is readily calculated using linear elastic theory or from a computer program, where the deflection of the structure for a specified force can be determined, or from static testing. Alternatively, it is possible to calculate the participating mass for a particular mode of vibration if the stiffnesses and frequencies of the structure are known either by dynamic testing or by eigenvalue extraction using a computer analysis program. The eigenvalue is simply the participating mass to stiffness ratio K/M that is fundamental to Equation 5.11.

Equivalent single degree of freedom systems

In reality, even simple structures are complex multi-degree of freedom systems, however given that many bridges have a

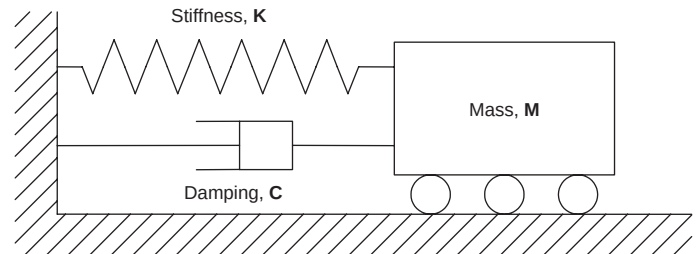


Figure 5.1 Single degree of freedom (SDOF) oscillator

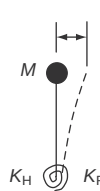
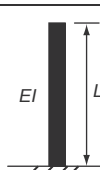
Bridge system	Equivalent SDOF system	Natural frequency
Longitudinal bending frequency of columns on spread footings or piled foundations supporting simply supported spans		$f_1 = \frac{1}{2\pi} \sqrt{\frac{\left(\frac{3EI}{L^3} + K_R + K_H\right)}{M}}$ where: EI is the column stiffness L is the column height from top of foundation to deck centroid K_R is the foundation rotational stiffness K_H is the foundation lateral stiffness M is the deck permanent and superimposed mass
Longitudinal bending frequency of a freestanding bridge tower or pylon of near constant cross-section		$f_1 = \frac{3.52}{2\pi L^2} \sqrt{\frac{EI}{m}}$ where: EI is the tower stiffness L is the tower height m is the mass per unit height of the tower on the assumption that it is distributed evenly about its height

Figure 5.2 Natural bending frequency equations for bridge columns and free-standing pylons

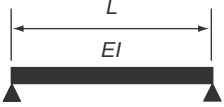
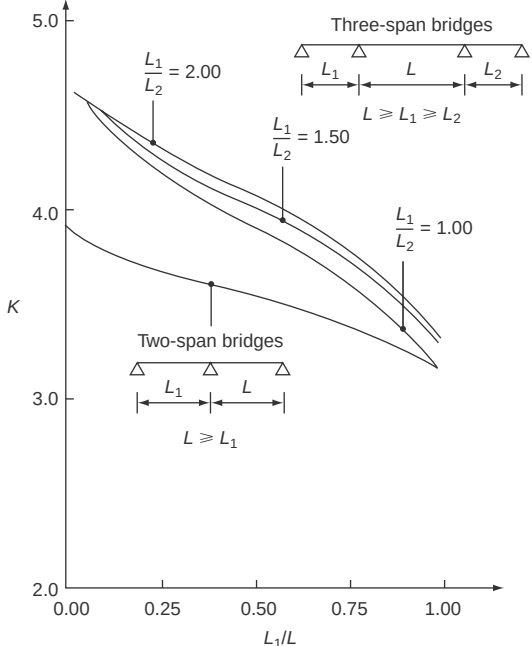
Bridge system	Equivalent SDOF system	Natural frequency
Vertical bending frequency of a plate or box girder bridge deck		$f_{1B} = \frac{K^2}{2\pi L^2} \sqrt{\frac{EI}{m}}$ where: EI is the deck flexural stiffness in vertical bending L is the span length m is the mass per unit length of the deck based on the midspan cross-section including both dead and superimposed dead loads K is a dimensionless constant depending on the span arrangement as follows: For single-span bridges or multi-span simply supported bridges: $K = \pi$ if simply supported $K = 3.924$ if propped cantilever $K = 4.730$ if built-in supports For two-span continuous bridges: K is derived from the 'two-span bridge' curve in the figure below. For three-span continuous bridges: K is derived from the figure below using the appropriate curve for three-span bridges where L_1 is the longest side span and L_2 is the length of the other side span and $L > L_1 > L_2$. If $L_1 > L$ then K is obtained from the curve for two-span bridges ignoring the shorter side span. For symmetrical four-span continuous bridges: K may be obtained from the 'two-span bridge' curve, treating each half as an equivalent two-span bridge For non-symmetrical four-span continuous bridges and continuous bridges with more than four spans: K may be obtained from the figure below using the appropriate curve for three-span bridges choosing the main span, L, as the longest internal span. Note: If the value $\sqrt{\frac{EI}{m}}$ at the support exceeds twice the value at midspan or is less than 80% of the midspan value then the expressions above should not be used.
Factor K used for derivation of fundamental vertical bending frequency (reproduced from Eurocode 1-4)		

Figure 5.3 Vertical bending frequency equations for typical bridge deck configurations (reproduced from Eurocode 1, Part 1-4 (BSI, 2005))

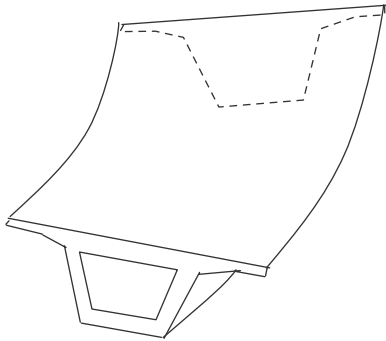


Figure 5.4 Torsional frequency equations for box girder bridge deck (reproduced from Eurocode 1, Part 1-4 (BSI, 2005))

predominant fundamental mode of vibration along a particular axis, for practical purposes it is often adequate to treat them as equivalent single degree of freedom (SDOF) oscillators, as shown in Figure 5.1. Thus, for simple structures it is possible to use hand calculation methods to determine natural frequencies rather than eigenvalue extraction using a computer analysis program.

Derivation of the natural frequencies of a number of equivalent SDOF systems, including cantilever, simply supported, built-in and continuous beams, has been undertaken by Buchholdt and Moossavi Nejad (1997). A number of the most useful natural frequency equations are given in Figures 5.2 to 5.4.

It is recommended that, even when using computerised eigenvalue extraction methods, hand calculations are undertaken of the fundamental frequency to provide a simple check of the results.

Multi-degree of freedom systems

All real structures possess an infinite number of degrees of freedom; however, for the purposes of practical analysis, they are usually approximated to a defined number. This is possible by assuming that the distributed mass within the structure is replaced with lumped masses and that the elastic members are weightless. In this manner, any structure can be idealised as a mass–spring system with the number of mass–spring components equal to the number of degrees of freedom, as shown in Figure 5.5.

The complexity of the algebra required for multi-degree of freedom (MDOF) systems means that, in most cases, computerised analysis using standard eigenvalue solvers is the most efficient solution method. The eigenvalue has been described already and is simply the square of the angular frequency.

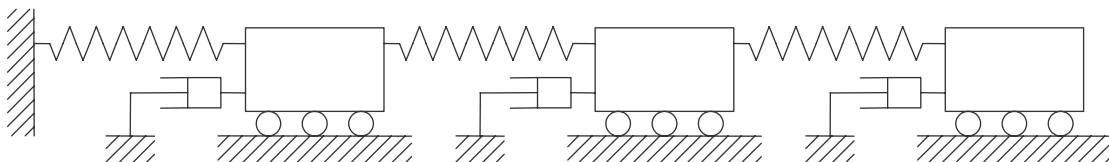


Figure 5.5 Equivalent multi-degree of freedom (MDOF) spring system

The eigenvector is the other important result from the analysis and describes the mode shape for each natural frequency. The eigenvalue and the eigenvector are of fundamental importance in frequency-domain analysis, where externally applied forces due to wind, earthquake or waves are described in terms of their frequency content in power spectra. This method is a very powerful and efficient means of dynamic analysis for elastic structures, providing a means of determining the response to random vibrations without the need to resort to time-domain analysis and the inherent difficulties in producing representative time histories.

For an N degree of freedom (DOF) structure, N eigenvalues and eigenvectors will result. For practical purposes, it is only necessary to determine a small subset of eigenvalues, such that it can be assured that the analysis has captured the predominant modal responses.

Damping is ignored within the calculation of eigenvalues, as normal levels of structural damping have an insignificant effect on the frequencies calculated while considerably increasing the numerical effort required.

Damping

Damping is simply the dissipation of energy, typically within a structure as heat or by radiation into the ground from foundations. Without replacement of this 'lost' energy, the amplitude of vibration will inevitably reduce to the point at which the structure returns to a state of rest. Heat energy is generated in a number of different ways within a structure, with the following as the most common sources:

- **Dry friction** – heat is generated by friction between structural components. For example, at bolted connections in a steel deck or between a foundation and the surrounding soil.
- **Fluid friction** – where a bridge pier or foundation is vibrating within water, heat is generated due to friction between the structure and the fluid.
- **Hysteretic** – hysteresis infers a degree of non-linearity in the real behaviour of the structure. For example, the opening and closing of cracks within concrete results in some internal friction that generates heat. Hysteretic damping can also be deliberately introduced into a structure by including non-linear elements, such as elastomeric bearings or dampers.
- **Hydrodynamic** – in bridges, this form of damping is most commonly associated with the introduction of tuned liquid dampers (TLDs) or tuned sloshing water dampers (TSWDs).
- **Aerodynamic** – as bridges vibrate within wind, flow heat is generated as friction due to the relative velocities between the air and the structure.

It is difficult or impossible to calculate damping mathematically, as the forces that contribute to it may vary with internal stresses, amplitude, velocity, acceleration and a number of other factors. However, over the years, a number of models

have been developed that produce approximations of damping that are satisfactory for all practical purposes. Of these models, the simplest and thus most commonly adopted is that of viscous damping.

The viscous damping model assumes that the damping force, F_d , is proportional to the structure velocity. The damping force can therefore be expressed as

$$F_d = C\dot{x}(t) \quad (5.12)$$

where C is the damping constant or coefficient of viscous damping and $\dot{x}$ is the velocity of vibration at time t . Therefore, the unit of C is newton second per metre.

If a structure is displaced from rest and viscous damping assumed, Equation 5.4 can be rewritten in its characteristic form:

$$Ms^2 + Cs + K = 0 \quad (5.13)$$

The following parameters are substituted:

$$\omega^2 = \frac{K}{M} \quad (5.14)$$

$$\xi = \frac{C}{2M\omega} \quad (5.15)$$

yielding

$$s^2 + 2\xi\omega s + \omega^2 = 0 \quad (5.16)$$

The roots to this characteristic equation are

$$s = \omega(-\xi \pm \sqrt{\xi^2 - 1}) \quad (5.17)$$

and fall into one of the following three cases:

- (i) $C = 2\sqrt{KM}$, known as *critical damping* and denoted C_c ,
- (ii) $C > 2\sqrt{KM}$, known as *overdamping*,
- (ii) $C < 2\sqrt{KM}$, known as *underdamping*.

The ratio C/C_c is called the damping ratio and is written as ξ . Thus, the damped angular natural frequency, ω_d is

$$\omega_d = \omega_n \sqrt{1 - \xi^2} \quad (5.18)$$

When $\xi < 1$, the free vibration response is given by the following equation, which is shown graphically in Figure 5.6:

$$x(t) = x(0) e^{-\xi\omega_n t} \cos(\omega_d t) \quad (5.19)$$

The product $\xi\omega_n t$ is referred to as the *logarithmic decrement of damping* and is usually denoted δ . The level of viscous damping within structures is quoted using one of these two parameters; it is important that the engineer distinguishes between them. They are related as

$$\xi = \frac{\delta}{2\pi} \quad (5.20)$$

When $\xi \geq 1$, the structure is said to be overdamped and oscillation will be prevented. The motion of such behaviour is shown in Figure 5.7.

Where $\xi = 1$, the damping is called critical damping and

represents the minimum damping required to prevent oscillation of the structure. In bridge structures, the level of damping is usually very small; thus, bridges respond as underdamped.

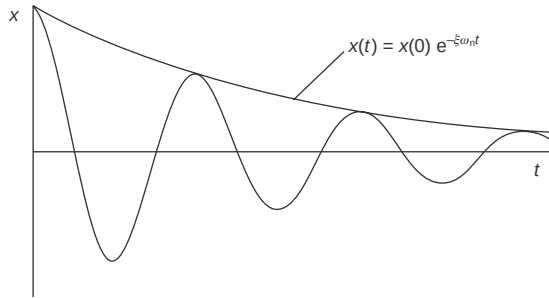


Figure 5.6 Damped free vibration response

The decay function shown in Figure 5.6 can be observed and measured in a real structure through the sudden release of a load or by forcing vibration at a resonant frequency and then stopping the vibrator. If the motion after release is recorded on a computer or a pen-recorder, the logarithmic decrement of damping can readily be calculated by comparing the amplitude of motion over a number of complete cycles, as

$$\delta = \frac{1}{n} \ln \frac{x_0}{x_n} \quad (5.21)$$

where n is the number of complete cycles of vibration; x_0 and x_n are the amplitudes at the start and end of n cycles, respectively.

Another common model is to assume that the damping matrix is proportional to the stiffness and mass matrices. This is known as Rayleigh damping and is commonly used in time history analysis software. In this model, the damping matrix $\mathbf{C}$ is defined as

$$\mathbf{C} = \eta \mathbf{M} + \delta \mathbf{K} \quad (5.22)$$

where η and δ are proportionality constants, chosen to give the correct order of magnitude for the damping at two frequencies.

If the required damping at a circular frequency of ω_s is ζ_s and the required damping at a circular frequency of ω_r is ζ_r , then the values of η and δ are

$$\eta = \frac{2 \omega_r \omega_s (\zeta_s \omega_r - \zeta_r \omega_s)}{\omega_r^2 - \omega_s^2} \quad (5.23)$$

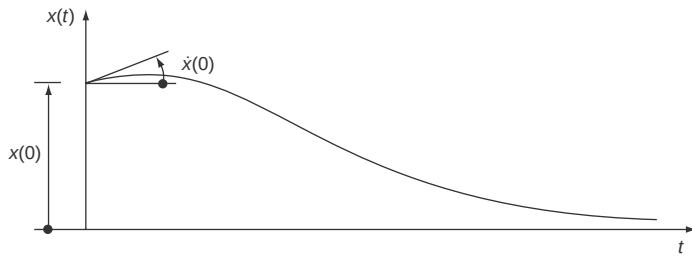


Figure 5.7 Overdamped free vibration response

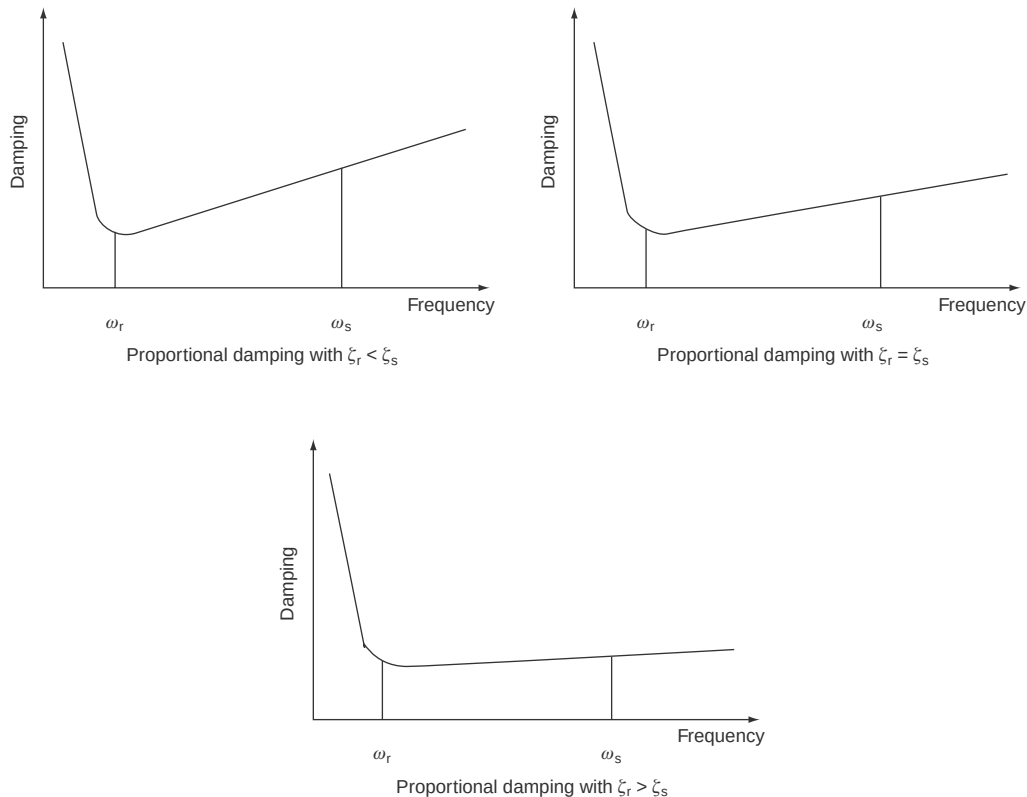


Figure 5.8 Rayleigh damping

$$\delta = \frac{2(\zeta_s \omega_s - \zeta_r \omega_r)}{\omega_r^2 - \omega_s^2} \quad (5.24)$$

Figure 5.8 shows how Rayleigh damping varies with frequency for various values of specified damping. Where $\zeta_r < \zeta_s$, the damping increases very rapidly with frequency, thus highlighting one of the limitations of this model. It is therefore essential that the constants η and δ are calculated for each analysis on the basis of the range of frequencies of interest. Rayleigh damping has some notable characteristics, which affect the way that damping is applied to the structure. In particular, damping of frequencies below ω_r is predominantly controlled by the mass coefficient η , whereas above ω_s the stiffness coefficient, δ , dominates.

The mathematical models of damping described both assume that damping is distributed throughout the structure and is relatively small in magnitude. They will cease to be adequate where damping is greater than 10% of the critical value; however, this is not generally the case for most bridges. Where damping is localised within the structure, as in the case of dampers and isolators, explicit modelling of these devices should be adopted. In a number of structural analysis programs available on the market, it is possible to model such devices on the basis of their theoretical or tested properties.

Typical values of logarithmic decrement of structural damping for common forms of bridge construction are given in Table 5.1. Further guidance is to be found in the Eurocodes.

Structural type		Structural damping, δ
Reinforced concrete towers and pylons		0.03
Steel bridges	welded	0.02
	preloaded bolts	0.03
	non preloaded bolts	0.05
Composite bridges		0.04
Concrete bridges	prestressed without cracks	0.04
	with cracks	0.10
Timber bridges ¹		0.06–0.12
Bridges, aluminium alloys		0.02
Bridges, glass or fibre-reinforced plastic ¹		0.04–0.08
Bridge cables and stays	parallel cables	0.0045
	spiral cables	0.015

Note 1: The values for timber and plastic composites are indicative only

Table 5.1 Typical values of structural damping

Forced vibration of structures

Forced vibration results when an external transient load acts on the structure mass. This external force $F(t)$ is known as the ‘forcing term’ within the equation of dynamic equilibrium (Equation 5.1) and can be characterised as either harmonic or arbitrary in nature (see Figure 5.9). Even seemingly random force vibrations, such as those generated by wind, waves and earthquakes, can be decomposed into a series of harmonic

components through the use of Fourier analysis. This approach can also be applied to impulse type forces caused by blast explosions or impact and is therefore a very powerful means of undertaking frequency-domain analysis for linear dynamic problems.

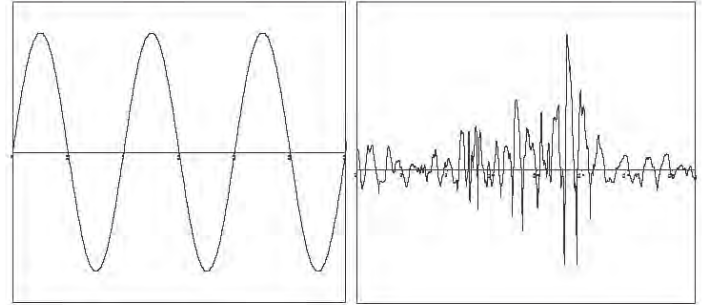


Figure 5.9 Harmonic and random forcing functions

True harmonic loading is typically caused by rotating machinery and is thus not particularly common on bridges. However, the treatment of harmonic loading is fundamental to an understanding of the frequency-domain analysis of random loading.

The harmonic forcing function takes the form

$$F(t) = F_0 \sin(\omega t) \quad (5.25)$$

where F_0 is the amplitude and ω is the angular frequency of the loading. Substituting this into the generalised equation of dynamic equilibrium (Equation 5.3) results in

$$M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = F_0 \sin(\omega t) \quad (5.26)$$

Dividing this equation by the mass, M , gives

$$\ddot{x} + 2\zeta \omega_n \dot{x} + \omega_n^2 x = q_0 \sin(\omega t) \quad (5.27)$$

The solution to this equation consists of two parts; the complementary solution and the particular solution. If the system is damped, oscillation corresponding to the complementary solution will decay with time. After some time, the motion will reach a steady state, and the system will vibrate at a constant amplitude and frequency. This motion, which is called forced vibration, is described by the particular solution, expressed as

$$x = C \sin(\omega t) + D \cos(\omega t) \quad (5.28)$$

It can be observed that the steady forced vibration occurs at the frequency of the excited force, ω , not the natural frequency of the structure, ω_n .

Substituting Equation 5.28 into Equation 5.27, the displacement amplitude can be shown to be

$$X = -\frac{F}{\omega^2} \frac{1}{\sqrt{\left\{1 - \left(\frac{\omega}{\omega_n}\right)^2\right\}^2 + \left(\frac{2\zeta\omega}{\omega_n}\right)^2}} \quad (5.29)$$

The term $-F/\omega^2$ is the static displacement caused by the force due to the inertia force. The ratio of the response amplitude relative to

the static displacement $-F/\omega^2$ is called the dynamic displacement amplification factor, D , given as

$$D = \frac{1}{\sqrt{\left\{1 - \left(\frac{\omega}{\omega_n}\right)^2\right\}^2 + \left(\frac{2\xi\omega}{\omega_n}\right)^2}} \quad (5.30)$$

The variation of the magnification factor with the frequency ratio ω/ω_n and damping ratio ξ is shown in Figure 5.10. This relationship is known as a *frequency response function*.

When the dynamic force is applied at a frequency much smaller than the natural frequency of the system ($\omega/\omega_n < 1$), the response is quasistatic. The response is proportional to the stiffness of the structure, and the displacement amplitude is close to the static deflection.

When the force is applied at a frequency much greater than

the natural frequency ($\omega/\omega_n > 1$), the response is proportional to the mass of the structure and the displacement amplitude is less than the static deflection ($D < 1$).

When the force is applied at a frequency close to the natural frequency, the displacement amplitude increases significantly. The condition at which $\omega = \omega_n$ is called *resonance*.

Response to general loading

If the loading follows a general periodic, but not harmonic, or a transient time function, there is no particular solution available for Equation 5.2. One means of calculating the SDOF response to such loading is to work in the frequency domain

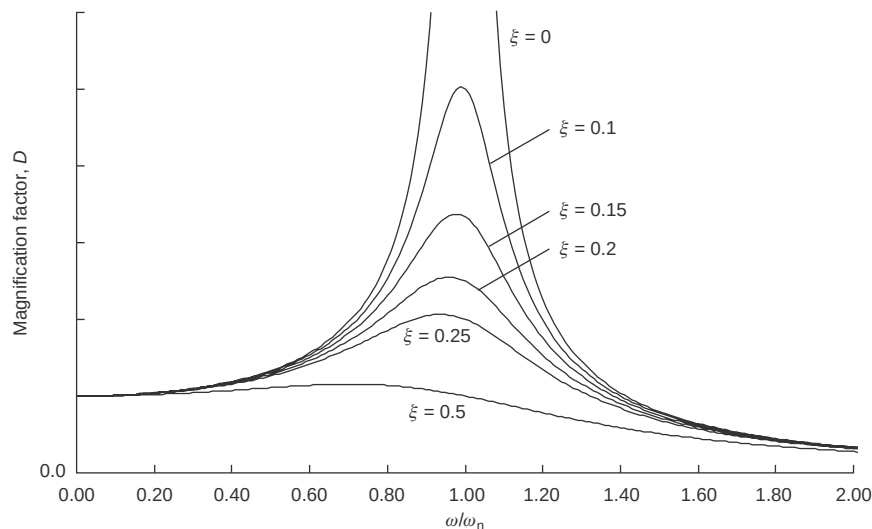


Figure 5.10 Frequency response function

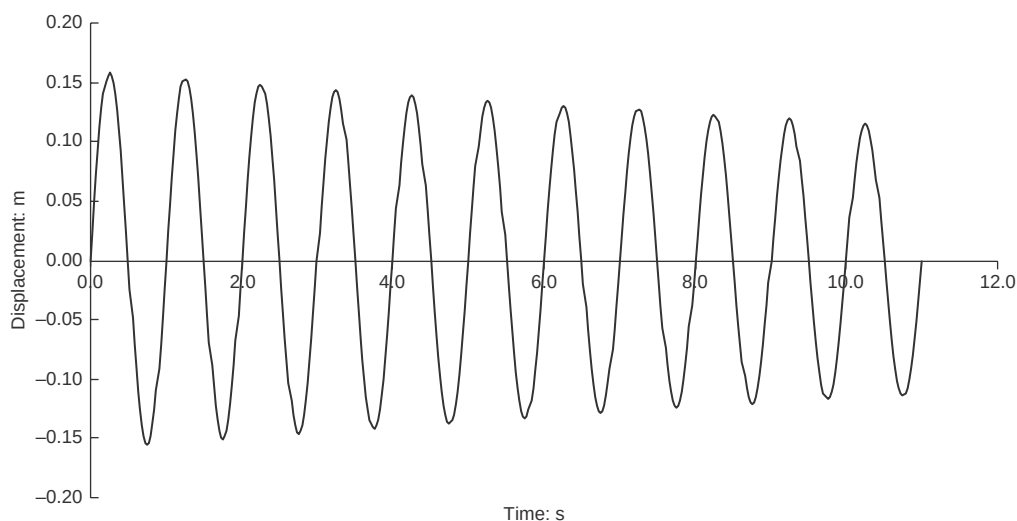


Figure 5.11 Impulse response function for a 1 Hz SDOF oscillator ($\xi < 0.005$, $m = 1$)

and decompose the periodic forcing function into its harmonic components. This can be undertaken using Fourier analysis.

Impulse loading

The impulse response function defines the response of the system to a unit impulse – that is, an infinite force applied for zero time, such that the integral of (force×time)=unity. Duhamel's integral (Equation 5.31) computes the response to an arbitrary load variation by reducing the variation to a series of impulses, and summing the responses from each individual impulse:

$$x(t) = \int_0^t p(\tau) \cdot h(t - \tau) d\tau \quad (5.31)$$

where the impulse response function

$$h(t) = \frac{1}{m \omega_d} \cdot e^{-\zeta \omega_n t} \sin(\omega_d \cdot t) \quad (5.32)$$

in which $x(t)$ is the displacement response of the point in the structure under consideration, $p(t)$ is the load time history of the mode, ω_d is the damped natural frequency of the mode calculated in accordance with Equation 5.18, m is the modal mass and ζ is the structural damping ratio. The numerical computation of the integral can be implemented with great efficiency using Simpson's rule, which provides an exact evaluation for piecewise-linear load variation.

The impulse response function for a typical SDOF system is shown in Figure 5.11.

Time-domain methods

Dynamic analysis undertaken in the time-domain requires an incremental method in which the equilibrium equation (Equation 5.1) at each point in the structure is solved at times Δt , $2\Delta t$, $3\Delta t$, and so on. There are a large number of incremental solution methods, many of which are implemented in modern structural analysis software. Linear elastic problems are most efficiently solved using Duhamel's method, described previously, using a modal analysis technique. This approach is capable of providing very rapid solutions to arbitrary load time histories, but is absent from many commonly used computer codes. A more common method involves a complete solution of the complete set of equilibrium equations at each time step using iterative procedures. Where the analysis is non-linear, the full structure stiffness matrix will be reformed at each time step. Consequently, time-domain analysis is computationally demanding, leading to long analysis times for structures with large numbers of degrees of freedom. In addition, artificial or numerical damping must be added to most incremental solution methods in order to obtain stable solutions. For this reason, engineers must choose the input parameters very carefully so as to obtain reliable results. All incremental solution schemes can fundamentally be classified as either *explicit* or *implicit* integration methods.

Explicit schemes do not involve the solution of a set of linear equations at each step. Basically, these methods use the differential equation at time t to predict a solution at time $t + \Delta t$. Explicit methods are conditionally stable with respect to the size of the time step. The time step adopted should be such that $\Delta t \leq \Delta t_{cr}$, where

$$\Delta t_{cr} = \frac{2}{\omega_{max}} \quad (5.33)$$

in which ω_{max} is the structure maximum angular frequency. This relationship requires the time step to be less than the time taken for a sound wave to traverse the smallest element within the finite-element model. Therefore, for most real structures, which contain stiff elements, a very small time step is required in order to obtain a stable solution.

Explicit algorithms are generally used for problems that require small time steps irrespective of the stability requirements. These problems are classed as wave propagation problems because the behaviour of the wave front, dominated by high-frequency components, is of engineering importance. This category includes the shock response from explosive or impact loading. Explicit integration schemes include the central difference method and the Newmark method (with $\beta < 1/4$).

Implicit methods attempt to satisfy the differential equation at time, t , after the solution at time ' $t - \Delta t$ ' is found. These methods require the solution of a set of linear or non-linear equations at each time step. Larger time steps may be used in implicit dynamics, as the integration schemes employed are referred to as being 'unconditionally stable'. However, the use of too large a time step may lead to a loss of accuracy in the solution obtained. Where the analysis is non-linear, the use of too large a time step may also result in iterative convergence failure. Owing to the experience required in choosing suitable time steps, many software programs incorporate automatic time stepping or 'adaptive' algorithms, which vary the time steps during the analysis to ensure the most efficient balance between computational accuracy and speed. For implicit analysis, the time steps are generally much greater than the critical time step and consequently the higher modes are not integrated accurately. Therefore, it is usually advantageous if the integration algorithm dampens the response of the higher modes by progressive amplitude decay. This type of algorithm is termed a dissipative algorithm. Ideally, the algorithm should exhibit no numerical dissipation in the lower modes that govern the response of the structure, while providing high dissipation in the higher modes that have not been integrated accurately. The aim of using dissipative algorithms is to dampen the background noise that is associated with higher modes that are insignificant in the response of the structure, thus making the analysis more efficient. Implicit algorithms are generally used for inertial problems, where the response is governed by the low-frequency components. This category includes seismic and wind buffeting response. Implicit schemes include the Newmark family of methods (with $\beta \geq 1/4$), the Hilber–Hughes–Taylor method, the average acceleration method and the Runge–Kutta method. Most software programs will only incorporate a limited selection of these schemes.

The subsequent sections of this chapter examine the practical application of the principles previously described to the most common types of dynamic excitation experienced by bridges.

Wind-induced vibration

When considering the dynamic response of structures to wind, two broad categories of mechanism might occur. Short-term variations in wind speed produce turbulent flow, or buffeting, that might result in resonant response of the structure. Excitation might also occur in smooth flow. These responses are categorised as aerodynamic instabilities.

All bridges are subject to the wind; however, some types are more susceptible to dynamic response than others. The natural frequencies of many simple bridge types are sufficiently high as to avoid a dynamic response. In this case, it is usually sufficient to consider the wind to be a quasistatic load on the structure. Where more slender structures are proposed that respond at lower frequencies, the aerodynamic response may be significant. In this case, the bridge design must be carefully examined to ensure that no divergent instabilities occur and that excitation due to wind gusting is within acceptable limits.

In the UK, guidance on determining the susceptibility of bridge designs to aerodynamic excitation is provided by the UK Design Manual for Roads and Bridges Standard CD 363 (Highways England, 2020a).

Definition of wind

Wind results from the movement of air particles in the earth's atmosphere. The energy causing this movement is derived from solar radiation coupled with the radiation of heat away from the earth's surface. This produces temperature differences and thus pressure gradients, which in turn cause acceleration of air particles between areas of high and low pressure.

Away from the earth's surface, the pressure system is relatively stationary, as the pressure gradients are balanced by the earth's centripetal and Coriolis accelerations. Nearer ground level, significant drag is experienced as a result of the surface roughness, resulting in disturbances to this pressure system. The effect of surface roughness is to reduce the wind speed near the ground, to cause the wind to flow in directions that are not parallel to the isobars and to produce turbulent flow characteristics.

Where man-made obstructions, such as bridges, are placed in the path of the wind, significantly increased disturbance to the wind flow may be caused. This increases the potential for aerodynamic instability.

The characteristics of the wind in any particular direction may be described by the following components:

- mean wind speed
- turbulent or gust component.

These components are further described in the following sections.

Mean wind speed

Recording periods of between 10 min and 2 h provide reasonably stable measurements of the mean component of the wind. In the UK, a recording period of 1 h at a height of 10 m is adopted. As previously highlighted, the mean wind speed varies with height from the ground, owing to drag at the earth's surface. A number of laws have been derived to describe this variation, the most popular of which is the logarithmic law adopted by Eurocode 1, Part 1-4 (BSI, 2005a). This gives the mean wind speed $V_m(z)$ at height z above the ground as a function of the basic wind speed V_b as

$$V_m(z) = C_r(z) \cdot C_0(z) \cdot V_b \quad (5.34)$$

where $C_r(z)$ is the roughness factor, calculated as

$$C_r(z) = k_r \ln\left(\frac{z}{z_0}\right) \quad \text{for } z_{\min} \leq z \leq 200 \text{ m} \quad (5.35)$$

$$C_r(z) = C_r(z_{\min}) \quad \text{for } z < z_{\min} \quad (5.36)$$

In other words, a logarithmic profile is taken above the reference height $z_{\min}$, and a constant value is taken below $z_{\min}$. Within this equation, z_0 is the roughness length and k_r is the terrain factor, calculated as

$$k_r = 0.19 \left(\frac{z_0}{0.05}\right)^{0.07} \quad (5.37)$$

Within these expressions, z_0 and $z_{\min}$ are determined from Table 5.2 for various categories of terrain.

Terrain category		z_0 :m	$z_{\min}$:m
0	Sea or coastal area exposed to the open sea	0.003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0.01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0.05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0.3	5
IV	Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m	1.0	10

Table 5.2 Roughness length and reference height for different terrain categories (from BS EN 1991-4)

In Equation 5.34, $C_0(z)$ is the orographic factor, which takes account of the increase in mean wind speed upwind of isolated cliffs or escarpments. Annex A of Eurocode 1 Part 1-4 (BSI, 2005a) provides a method for calculating this factor.

Turbulence properties

- Variance and standard deviation. The variance of the gust velocity is the mean square value of its fluctuations in the three orthogonal directions. It has been observed that the greatest part of this variance is associated with the along-wind component. The variance

is both a function of ground roughness and height. The standard deviation $\sigma_u(z)$ at height z provides a measure of the dispersion of the wind around its mean value $V_m(z)$ and is used as a measure of the turbulence intensity $I_u(z)$, given by

$$I_u(z) = \sigma_u(z)/V_m(z) \quad (5.38)$$

The spectral density function or power spectrum. Power spectra describe the energy distribution of the wind with respect to its harmonic components, determined through Fourier transformation. This allows the engineer to undertake dynamic analysis within the frequency domain. Spectra are normally expressed in a non-dimensional form, thus:

$$\frac{n \cdot S_u(n)}{\sigma_u^2} \quad (5.39)$$

Davenport (1961) suggested the following formulation of wind spectral density function:

$$\frac{n \cdot S_u(n)}{\sigma_u^2} = 4800 v_m \kappa \frac{\beta \omega}{(1 + \beta^2 \omega^2)^{1.333}} \quad (5.40)$$

where $\beta = (600/\pi v_m)$, and κ is the surface drag coefficient, obtained from the roughness of the terrain as

$$\kappa = \frac{1}{\left(2.5 \ln\left(\frac{z}{z_0}\right)\right)^2} \quad (5.41)$$

A number of modifications were subsequently made to Davenport's formula, the most significant of which was to recognise the variation of spectral densities with respect to height above the ground. This was most conveniently described by Von Karman as

$$\frac{n \cdot S_u(n)}{\sigma_u^2} = \frac{4x}{(1 + 70.8x^2)^{5/4}} \quad (5.42)$$

while Eurocode 1 adopts the turbulence spectrum proposed by Solari (1993):

$$\frac{n \cdot S_u(n)}{\sigma_u^2} = \frac{6.868x}{(1 + 10.32x)^{5/4}} \quad (5.43)$$

where, in both Von Karman and Solari spectra

$$x = L_u^C \frac{n}{V_{(z)}} \quad (5.44)$$

where x is a non-dimensional frequency determined by the natural frequency of the structure in hertz, $n = n_{1,x}$, the mean velocity $v_m(z)$ and the turbulence length scale L_u^C , which represents the average gust size for natural winds, and

$$L_u^C = 300 (z/200)^{(0.67-0.05 \ln z_0)} \quad (5.45)$$

- The coherence or normalised co-spectrum $R(\lambda, n)$. Coherence functions describe the statistical dependency between the turbulence components of points separated in space by a distance λ . $R(\lambda, n)$ is a non-dimensional quantity, unity for small separations, with a roughly exponential decay as λ increases. It may be demonstrated that the correlation between two histories is negligible when the distance between two stations is greater than approximately 5 m. Thus the implication for many bridges is that cross-correlation is ignored when undertaking analysis in the frequency domain:

$$\sqrt{R(\lambda, n)} = \exp\left(-C \cdot \frac{\lambda \cdot f}{V(z)}\right) \quad (5.45)$$

where the parameter C is the decay factor with magnitudes as

	Lateral separation	Longitudinal separation	Vertical separation
Longitudinal turbulence component	8	2	8
Cross-wind turbulence component	4	4	4
Vertical turbulence component	8	4	4

Table 5.3 Coherence decay factor, C

shown in Table 5.3 and f is the frequency.

Aerodynamic motion

The effect of wind acting on structures is to induce deflections. These deflections have the potential to alter the boundary conditions of the incident wind to the extent that the flow pattern around the structure is changed. This gives rise to oscillatory deflections of the structure, a phenomenon commonly known as aeroelastic instability. All aerodynamic instabilities result from aerodynamic forces that are coupled to the motion of the structure. These instabilities are grouped into two broad categories.

Divergent amplitude responses include phenomena, such as galloping, stall flutter and classical flutter, that result in rapidly increasing oscillations, which must be avoided if structural failure is to be prevented. Galloping instabilities arise on certain types of bridge deck cross-section, owing to the variations of wind drag lift and pitching moments with wind incidence angle and time. The collapse of the first Tacoma Narrows Bridge, sometimes known as 'Galloping Gertie', in 1940 was due to classical flutter. Flutter occurs when a torsional disturbance in the structure increases the angle of wind incidence on a bridge deck. The deck responds by twisting further. Eventually, the angle of attack increases to the point of stall, and the bridge begins to twist in the opposite direction. In the case of the Tacoma Narrows Bridge, this mode was negatively damped (or had positive feedback), meaning that it increased in amplitude with each cycle because the wind pumped in more energy than the flexing of the structure dissipated. Eventually, the amplitude of the motion caused structural failure of the hanger cables, precipitating the rapid onset of progressive collapse.

Limited amplitude responses include phenomena, such as vortex-induced oscillations and turbulent response, that, although not divergent in nature, may result in overstress, fatigue damage to the structure or create motions affecting the serviceability of the bridge. Vortex-induced oscillations result from the alternate shedding of vortices from the upper and lower surfaces of the bridge deck. The shedding of vortices occurs at critical wind speeds; if these are close to the natural frequency of the structure in bending or torsion, a resonant response will occur. Vortex shedding most commonly occurs on bridge decks, single leg pylons and stay cables. Oscillation of a bridge, owing to a turbulent response, will occur if the energy present over a particular frequency band is sufficient to excite the natural frequencies of the structure.

Section 8 of Eurocode 1 Part 1-4 (BSI, 2005a) provides guidance on determining the susceptibility of bridges to aerodynamic behaviour. In general, only long-span bridges and footbridges with very flexible decks are prone to aerodynamic instability; where this is the case, it is usual to arrange wind tunnel testing. Instability can often be eliminated by careful design of the bridge deck cross-section, sometimes requiring fairings or other aerodynamic modifications.

Oscillation of cables and stays

Oscillations of cables and stays occurs for a number of reasons, including wind-induced vibrations, rain–wind-induced vibrations, structure resonance (deck or supporting structure) and traffic vibrations.

Wind-induced vibrations may be caused by vortex excitation, buffeting, wake galloping and rain–wind interaction.

Vortex excitation

The critical wind speed for vortex excitation is based on the Strouhal number:

$$S = fd/V \quad (5.47)$$

where

f = frequency of vortex shedding

d = diameter of stay

V = wind speed.

For a cylindrical cable, S is equal to 0.2, which gives the critical wind speed $V_c = 5\omega d$, where ω is the natural frequency of the cable.

The risk of significant vibrations occurring is a function of the Scruton number, Sc . When Sc is greater than 20, there is little risk of significant vibrations due to vortex excitation.

$$Sc = (2m_c \delta_s) / (\rho_a d^2) \quad (5.48)$$

where

m_c = mass per unit length (in kg/m)

δ_s = structural damping, typically between 0.005 and 0.01

ρ_a = air density, = 1.225 kg/m³ in the UK

d = diameter of stay (in m).

Wind buffeting

Buffeting occurs through the wind turbulence as it passes the stay. This effect is dependent on the wind characteristics as well as the stay details. Vibration of stay cables due to buffeting from the wind is rare.

Wake galloping

Caused by turbulent wake from adjacent cables, this is usually only significant if the stay spacing is less than 5× the stay diameter.

Rain–wind-induced oscillations

This is a phenomenon experienced on a number of cable-stayed and arch bridges, where vibrations have been witnessed when low wind speeds combine with light rain. Stays within polyethylene ducts are known to be the most susceptible to this effect.

Rain–wind oscillations of the stays usually occur in the wind speed range of 7–20 m/s. Below this range, rain rivulets do not form, while above this range the rivulets tend to be blown off by the wind before excitations occur. It has been suggested that there is negligible risk of rain–wind oscillation when $Sc > 10$, while rain–wind vibrations have been observed at frequencies between 1 and 3 Hz.

Encasing the cables with high-density polyethylene (HDPE) ducts ‘roughened’ with ribs or dimples has proven to be effective in preventing rain–wind vibrations. Figure 5.12 shows some examples of the types of surface finish that are commonly adopted.



Figure 5.12 Surface finishes on stay cables: (a) ribbed HDPE ducting; (b) roughened HDPE ducting

Stay natural frequency

The natural frequency of a stay cable depends on the direction of motion. For in-plane vibrations, the angular frequency, ω , is given by

$$\omega = (n\pi/L) \sqrt{(gT/w)} \quad (5.49)$$

where

$n = 1, 2, 3, 4$, etc., for ‘taut’ mode and 2, 2.86, 4, 4.92, etc., for sagging mode

L = ‘straight’ length of stay

T = tension force in stay

w = stay cable dead load per unit length.

If $(8\delta/L)^3 (EA/wL) < 1$, then the taut mode governs

$(8\delta/L)^3 (EA/wL) > 4\pi^2$, then the sagging mode governs,

where

δ = vertical sag of cable at midspan.

For stay vibrations perpendicular to the cable plane, the angular frequency, ω , is given by

$$\omega = (n\pi/L)\sqrt{gH/w} \quad (5.50)$$

where

$n = 1, 2, 3, 4$, etc.

Damping

The susceptibility of individual stays to vibration depends on many factors; it is notoriously difficult to predict whether the stays will suffer from vibration problems in the final structure based on design alone. It is common practice with many structures to fit damping to the stay system to prevent significant vibrations from occurring. In addition, stays will often include provisions for adding further damping at a later stage, should this prove necessary once the bridge has been completed and its performance in service has been monitored.



Figure 5.13 Examples of stay friction dampers

Dampers may be installed at both ends of the stay and should be easy to inspect and replace if necessary. Examples are shown in Figure 5.13.

Another method to counteract undesired oscillations is to increase the in-plane stiffness of stays by connecting them together with a set of transverse secondary cables, known as cross-ties. Cross-ties effectively reduce the free length of cables, thus increasing their frequency. The connections transform individual cables into a more complex cable network. Analysis demonstrates that careful consideration of dynamic behaviour of the overall cable network must be given for the design of cross-ties. The increase in frequency in the fundamental modes that is usually attained by adding cross-ties must be balanced with potentially undesirable local mode behaviour. This aspect has the potential of reducing the overall benefit of enhancing the performance of individual stay behaviour by a network and must be carefully studied when planning this method of cable damping.

Assessment of bridge susceptibility to aerodynamic instability

Computer analysis of buffeting response

The dynamic response of aerodynamically stable structures may be predicted using either frequency- or time-domain methods. The latter method requires the generation of spatially correlated wind histories consistent with the design wind power spectrum. A large number of sets of artificial time histories must be processed, to ensure that the analysis is statistically representative.

The frequency-domain method is based directly on the wind power spectra and thus produces a computationally efficient upper bound assessment of structural response. The aim of the method originally proposed by Davenport (1961) is to predict the statistical properties of the structural response, starting from knowledge of the statistical properties of the forces due to wind. The frequency-domain method is limited to the analysis of structures whose dynamic response is linear. Consider an equivalent SDOF system subject to along-wind buffeting, as shown in Figure 5.14.

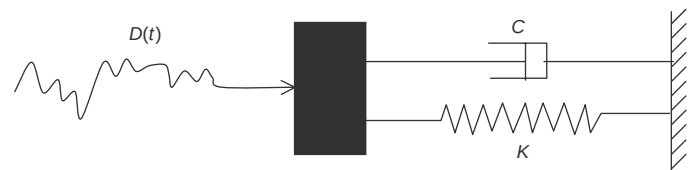


Figure 5.14 Equivalent SDOF system subjected to wind buffeting

The along-wind response x of the SDOF system is given by

$$x = x_s + x_d \quad (5.51)$$

where x_s is the static response component

$$x_s = \frac{\frac{1}{2}\rho C_d A V^2}{K} \quad (5.52)$$

in which

ρ = specific density of air

C_d = drag coefficient

A = area on which the wind load acts

V = mean wind velocity (averaged over 10 min)

K = stiffness

and x_d is the dynamic response component. Moreover,

$$x_s = \kappa \sigma_x \quad (5.53)$$

in which κ is a peak factor for weakly damped structures determined from

$$\kappa = \sqrt{[2 \ln(nT)]} + \frac{0.577}{\sqrt{[2 \ln(nT)]}} \quad (5.54)$$

where

n = natural frequency of SDOF system ($\sqrt{K/M}$)/ 2π

T = storm duration (taken as 3600 s)

and σ_x is the standard deviation of the turbulent component of response, with the variance

$$\sigma_x^2 = \frac{x_s^2}{V^2} \cdot \frac{\pi n}{\xi} \cdot A(n) \cdot S_u(n) \quad (5.55)$$

in which ξ is the total damping comprising structural (ξ_s) and aerodynamic (ξ_a) components. Aerodynamic damping is due to the relative movement of the structure relative to the wind and often of significant benefit when added to the structural damping:

$$\xi = \xi_s + \xi_a = \xi_s + \frac{F_d}{2\pi n VM} \quad (5.56)$$

$A(n)$ is an aerodynamic admittance function, which takes into account the spatial variation of the wind velocity and the frequency dependence of the drag coefficient. Guidance on a suitable value is found in specialist literature and design codes, such as Eurocode 1 Part 1-4 (BSI, 2005a). $S_u(n)$ is the design power spectrum.

The response of multi-DOF systems can be calculated in a similar manner to that of the SDOF system by first decoupling the equations of motion using eigenvalues and eigenvectors, then considering each modal equation as the equation of motion of a SDOF system. This is a computationally efficient method, commonly incorporated in the modal dynamics capabilities of modern structural analysis software.

Computational fluid dynamics

Computational fluid dynamics (CFD) is a computer modelling technique for simulating fluid flow by predicting air speed and direction, including the presence of vortex shedding; it is a tool that is gaining in popularity among bridge engineers and is useful for refining the shape of bridge decks prior to wind tunnel testing in order to eliminate features that instigate excessive vortex shedding. An example of a CFD analysis for the cable-stayed deck section of the Vasco de Gama Bridge in Portugal is shown in Figure 5.15.

There have been a number of instances to the author's knowledge where the use of CFD analysis as the sole means of confirming the stability of a particular deck cross-section to divergent responses has proven to be unreliable. It is therefore recommended that it is used with caution until such time as more reliable bridge-specific CFD methods are developed.

Wind tunnel testing

Wind tunnel testing is commonly undertaken for bridges with long or unusually flexible decks. Both CD 363 (Highways England, 2020a) and published document PD 6688-1-4 (BSI, 2015a) provide guidance as to when the use of wind tunnel testing is likely to be required in order to properly understand the structure's aerodynamic behaviour. There are three main objectives for the use of wind tunnel testing. First, where bridge decks or piers do not conform to the standard cross-sections contained within design codes, wind tunnel testing is used to determine structure-specific drag and lift coefficients. Static wind loads are measured in terms of the mean lateral force (F_x), mean normal force (F_z) and pitching moment (M_y) about the centre of the deck and are reported as static wind load coefficients. Measurements are made in both smooth and turbulent flow conditions, as appropriate to the bridge site location.

The second objective is to check for structure susceptibility to divergent amplitude responses, such as galloping and stall flutter or classical flutter and non-divergent responses to vortex shedding. In addition, flexible bridge decks must be checked for non-oscillatory divergence, which can occur if the aerodynamic torsional stiffness (i.e. the rate of change of pitching moment with rotation) is negative. At a critical wind speed, the negative aerodynamic stiffness becomes numerically equal to the structural torsional stiffness, resulting in zero total stiffness. Such analyses are usually undertaken in smooth flow conditions over a range of test wind incidence angles.

The third objective is to examine the effect of the surrounding topography or buildings on the dynamic response of the

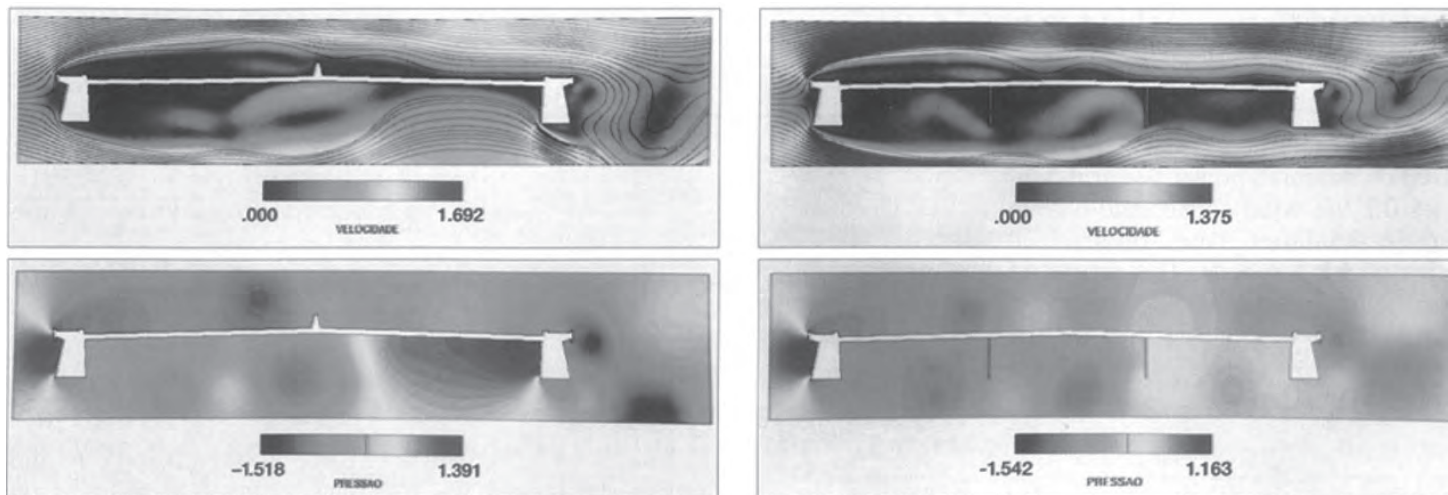


Figure 5.15 CFD analysis of the deck section of Vasco de Gama Bridge, Portugal

structure to turbulence. The most significant effect of such obstacles is to alter the angle of attack of the wind on the structure. The wind environment within built-up areas is often very complex; it is possible for wind to be funnelled around adjacent structures, leading to an increase in mean velocity on areas of the bridge under consideration.

Section model studies are conducted for prismatic (two-dimensional) structures, such as long-span bridge decks. A rigid model is mounted on a dynamic test rig consisting of a system of springs to model the properties of the structure. Measurements are made to determine the dynamic response due to such mechanisms as vortex shedding and the wind speeds for the onset of aerodynamic instabilities, such as galloping and flutter. The section model itself is designed using such materials as balsa wood, carbon fibre sheet and steel sections so that it accurately represents the scaled mass in conjunction with the lowest torsional and bending frequencies of the prototype. Sectional models should include handrails, parapets, guide vanes, maintenance gantry rails, stay pipes and all other non-structural appurtenances that have the potential to influence the aerodynamic behaviour of the deck. Where these items are too small to be modelled to scale, aerodynamically equivalent representations may be used. Dimensional accuracy is usually required to be within ± 0.05 mm. Where turbulent atmospheric conditions are required, they must be closely matched to the target spectral density function. Turbulent conditions within low-speed aeronautical wind tunnels are often generated by placing a two-dimensional grid consisting of vertical and horizontal strips at the entrance of the wind tunnel test section (see Figure 5.16). By changing the spacing of the grid, it is possible to simulate the desired turbulent conditions. Prior to installation of the test section, the wind properties just upstream of its location are measured in terms of mean wind speed, turbulence intensity and wind spectra, using hot-wire anemometry. These turbulence properties are measured over a range of wind

speeds representative of the expected experimental conditions. Results of the turbulence simulation are presented in terms of u - and w -component spectra and longitudinal and vertical turbulence intensities. The measured and target spectra should be compared with accepted industry standards such as those produced by the Engineering Sciences Data Unit (ESDU). During tests to examine the vortex shedding response, it is possible to use smoke visualisation to view the air flow over the structure.

Static load coefficients in the wind axis system are derived from the model forces and moments measured in the wind tunnel:

$$\text{drag coefficient} \quad C_D = \frac{D}{\frac{1}{2}\mu V_m^2 B} \quad (5.57)$$

$$\text{lift coefficient} \quad C_L = \frac{L}{\frac{1}{2}\mu V_m^2 B} \quad (5.58)$$

$$\text{pitching coefficient} \quad C_M = \frac{M}{\frac{1}{2}\mu V_m^2 B^2} \quad (5.59)$$

where D , L and M are, respectively, the wind axes along and across the wind force and the pitching moment; V_m is the mean wind speed; μ is the density of air (in the UK, taken as 1.225 kg/m^3) and B is a reference dimension, which is usually the deck width. Force coefficients must be corrected to account for blockage effects due to the confining effects of the wind tunnel. A number of methods for calculating correction factors have been proposed; reference to specialist literature, such as ESDU 80024 (ESDU, 1980), is recommended in order to establish the approach suited to the particular blockage ratio under consideration. Sections that comprise circular section members or curved surfaces are likely to be sensitive to the Reynolds number (Re); hence, adjustments based on full-scale data or theoretical considerations may be necessary.

An aerodynamic model of a full bridge is commonly referred to as an aeroelastic model. Aeroelastic tests are conducted for structures whose wind-induced motions are large, having a significant effect on the aerodynamic forces and hence the dynamic response. These are commonly structures that exhibit three-dimensional behaviour. Models are designed to reproduce the key dynamic properties by modelling the mass and stiffness distribution of the structure. In particular, the fundamental modes that govern the dynamic response of the structure must be accurately represented at the model-scale frequency. Modal responses are measured directly using accelerometers or displacement transducers. Dynamic loads are determined from the modal responses or, in the case of vertical structures, measured directly. Models for aeroelastic tests are typically produced at either 1:200 or 1:100 scales; therefore, the models for long-span bridges may be of considerable size. As a result, large-section boundary layer wind tunnels (BLWTs) are required for the tests. For the Akashi Kaikyo bridge in Japan, a 1:100 scale aeroelastic model was built with an overall length of approximately 40m. An example of an aeroelastic model in a BLWT is shown in Figure 5.17.

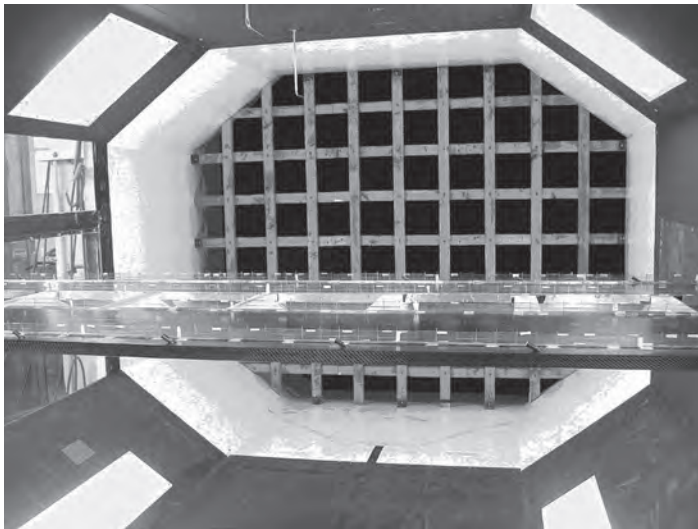


Figure 5.16 Closed-section low-speed aeronautical wind tunnel set-up for turbulent conditions (note grid upwind of model)

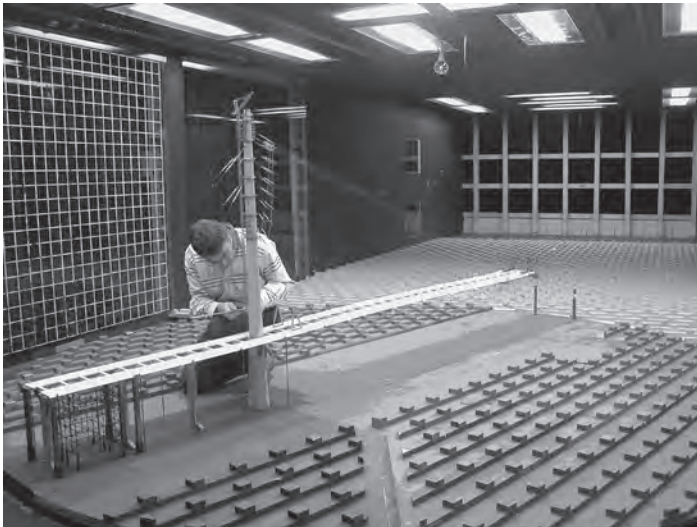


Figure 5.17 Boundary layer wind tunnel model (note turbulence grids in background and roughness elements on floor)

Turbulent boundary conditions are generated in a BLWT using an arrangement of roughness elements distributed over the floor of the wind tunnel and a two-dimensional barrier, with square vortex generating posts, placed at the entrance to the test section. The size, shape and distribution of these elements are designed to generate the particular turbulence characteristic required in the tests. Similar to the section model tests, calibration of the wind tunnel generated turbulence properties should be undertaken with reference to the target spectrum.

During the tests, displacement and acceleration time histories are measured at key points on the structure using displacement transducers and accelerometers. From the displacement time histories, mean, root mean square (RMS) background and RMS resonant components are derived. From acceleration time histories, RMS resonant components of the movement are derived. Time histories are recorded at fine wind speed increments in order to provide good coverage of the design wind speed range representative of the bridge site. Each time history is processed using Fourier transformation to determine the spectrum, thereby permitting the isolation of narrow-band responses that are characteristic of structural resonance.

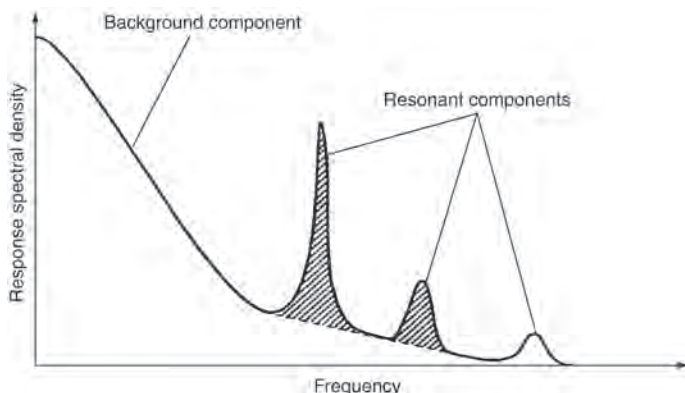


Figure 5.18 Spectral density of a response to wind

Background components are determined from the displacement spectra by filtering out the peaks corresponding to the resonant components of displacement. Figure 5.18 shows the components of a typical measured wind spectrum.

Peak displacements are then calculated as the mean displacements, plus or minus a combination of the standard deviation of the background and resonant components of displacement due to each mode. The following expression is commonly used for calculating the measured peak displacements:

$$D_{\text{peak}} = D_{\text{mean}} \pm \sqrt{(g_0 D_{\text{sdev}})^2 + \sum_k (g_k D_{\text{mode } k})^2} \quad (5.60)$$

where

D_{peak} = peak displacement

D_{mean} = mean displacement

D_{sdev} = standard deviation of background displacement

$D_{\text{mode } k}$ = RMS inertial displacement due to response in mode k

g_0 = peak factor to apply to background standard deviation (typically 3.4–3.5)

g_k = peak factor to apply to narrow-band displacement due to motion in mode k .

For the modal or narrow-band displacements, the requisite peak factors across the modal responses are calculated using the Davenport gust factor, given as

$$g_k = \sqrt{2 \ln(f_k \cdot T_0)} + \frac{0.577}{\sqrt{2 \ln(f_k \cdot T_0)}} \quad (5.61)$$

where

f_k = natural frequency of k th mode (in Hz)

T_0 = storm duration (typically 3600 s).

Correction factors may also be required within the calculation of the peak displacements, in order to account for scaling inaccuracies necessitated by virtue of the model scale, e.g. cable diameters.

Stay cable testing

Wind tunnel testing of stay cables is undertaken for the same principle reasons as the static testing of deck sections, namely the derivation of drag coefficient and examination for potential divergent instabilities. To ensure Reynolds number similarity between model and prototype, stay cables are usually tested at full scale. The cable models should accurately reflect the surface features of the prototype, including any helical fillets or dimple patterns. Cable models are dynamically mounted, using calibrated spring balances, on turntables in the ceiling and floor of the test section. This permits the cable model to be tested in a range of directions and inclinations to the prevailing wind. In some specialised wind tunnels, spray heads are mounted in the ceiling of the tunnel to simulate a range of rain conditions and thus enable the study of rain- and wind-induced cable vibrations. Figure 5.19 shows a typical cable test set-up.



Figure 5.19 Wind tunnel testing of stay cable

Earthquake-induced vibration

Earthquakes are one of the most destructive natural forces on the planet. Engineers must therefore plan the design of structures in seismically active regions of the world with great care to mitigate against collapse causing human and economic losses. Over the years, seismic analysis techniques have been developed that are common to many of the international codes of practice. Chapter 6 describes in some detail the principle mechanism of bridge collapses during earthquakes, with the preventative design and detailing measures adopted by the major international seismic codes. The purpose of this section is to give a brief overview of the propagation, measurement and characterisation of earthquakes. This is important background to understanding the most common methods for evaluating structural response to earthquakes in order to determine suitable design forces and displacements.

Propagation and measurement of seismic waves

Earthquakes are normally experienced as a series of cyclic movements of the earth's surface resulting from fault slips

within the earth's crust. When the strain energy stored within these faults is released, energy is propagated from the source as two types of waves, pressure or primary (P) waves and shear or secondary (S) waves. Given that P waves travel faster than S waves and knowing the wave velocities measured at a receiver it is possible to calculate the epicentre of any particular seismic event. As shown in Figure 5.20, P waves shake the ground in the direction they are propagating, while S waves shake perpendicularly or transverse to the direction of propagation.

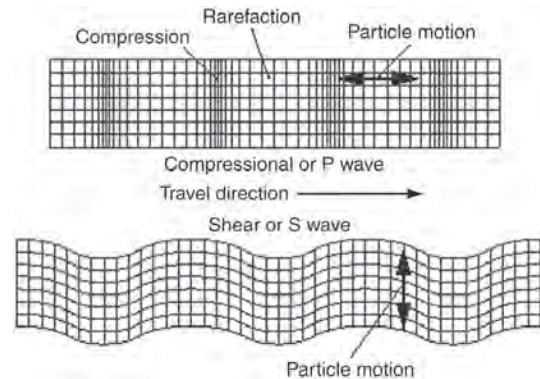


Figure 5.20 P-wave and S-wave propagation

Ground motion is measured by seismometers, which are sensitive to very small vibrations and thus go off the scale during major seismic events. Strong-motion seismometers are designed to be triggered when vibrations exceed a threshold that would indicate an impending earthquake. The data from seismometers throughout the world form the basis of our understanding of the properties of earthquake-induced ground vibrations and also the raw data for the development of seismic design spectra that are included in design codes.

Earthquake disturbance is most commonly understood in terms of the Richter scale, which ranges from 0 to 9.5, the upper value being the largest measurement to date (registered in Chile, 1960). The Richter scale is logarithmic and is a



Figure 5.21 Freeway bridge collapse due to earthquake, October 17, 1989, Loma Prieta, California, (courtesy of US Geological Survey)

measure of the strain energy released at an earthquake source. A magnitude 5 event would be a relatively minor event and not expected to cause significant structural damage, whereas the 1994 Northridge earthquake, with a magnitude of 6.6, resulted in an estimated 72 fatalities and \$12.5 billion damage. It is sobering to note that major loss of life in the Northridge earthquake occurred due to the collapse of highway bridges (see Figure 5.21).

Development of design response spectra

Information about ground motion is recorded by seismographs in the time domain in the form of displacement, velocity and acceleration histories (see Figure 5.22). These can readily be transformed into response spectra for use in design by time history analysis of idealised single degree of freedom systems,

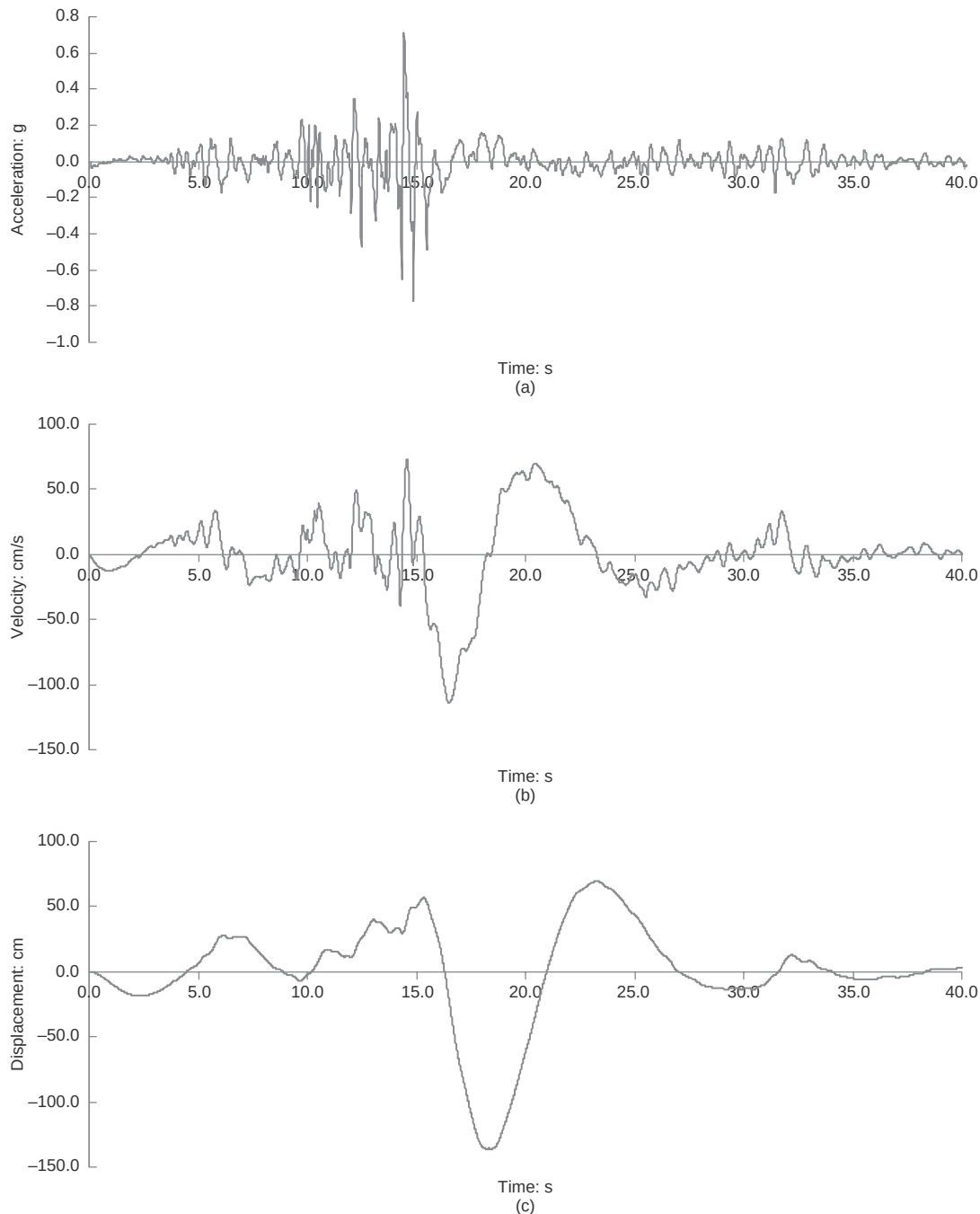


Figure 5.22 (a) Acceleration, (b) velocity and (c) displacement time histories recorded during Jiji earthquake, Taiwan, 21 September, 1999

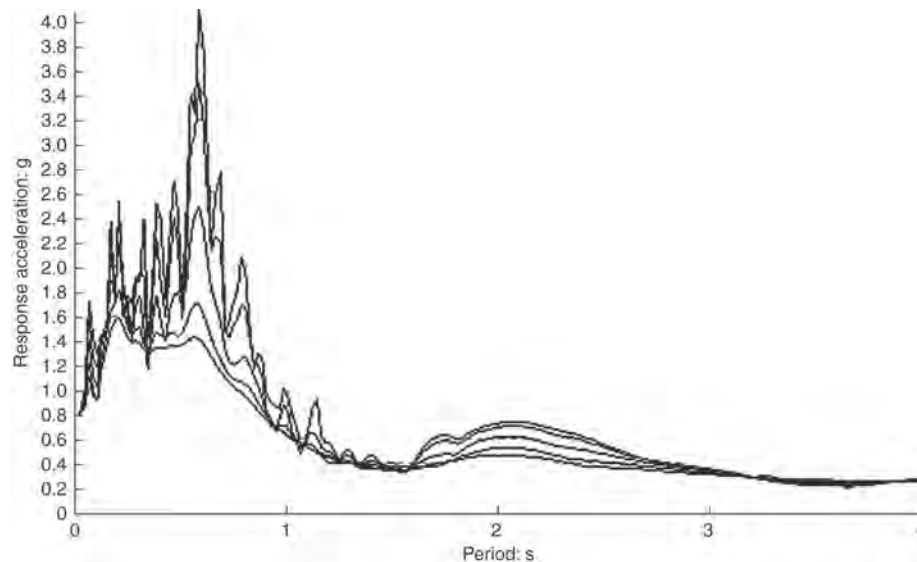


Figure 5.23 Elastic acceleration response spectrum for range of damping values derived from acceleration time history for Jiji earthquake, Taiwan, 21 September, 1999

tuned to a wide range of potential structure natural frequencies. The resulting plot of peak response against oscillator natural frequency provides an immediate indication of peak elastic response of a real structure of known natural frequency and damping (see Figure 5.23).

Seismologists and engineers use observational data from earthquakes in particular regions of the world to develop elastic response spectra suitable for adoption in design codes. This process requires Fourier analysis of large numbers of ground motion time histories to identify response spectrum, peak displacement, velocity and acceleration and strong-motion duration. Statistical analysis of the observational data is used to produce design response spectra with an acceptably low probability of exceedance within the specified return period. Design response spectra are usually normalised to 1.0g.

Influence of surface geology on ground motions

The properties of the soil overlying a site have a significant influence on the amplitude of seismic vibrations experienced at ground level. Where soft soils are present, the effect is akin to jelly on a shaking table, with the latter being the rock stratum. For example, the soft mud in San Francisco Bay has been recorded to produce an amplification of over 20 times the bedrock excitation amplitude. In conjunction with the amplification effect, soil strata also modify the predominant period of seismic excitation. Some weak granular soils may experience liquefaction during an earthquake; this phenomenon is discussed further in Chapter 6.

To cater for different soil conditions, seismic codes usually

Groundtype	Descriptionofstratigraphicprofile	Parameters		
		$v_{s,30}$:m/s	N_{SPT} :blows/30cm	c_u :kPa
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface	>800	–	–
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth	360–800	>50	>250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres	180–360	15–50	70–250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $v_s > 800$ m/s			
S1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index ($PI > 40$) and high water content	<100 (indicative)	–	10–20
S2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A–E or S1			

Table 5.4 Classification of soil profiles from Eurocode 8

include a soil factor within the design response spectra definition. Eurocode 8 (2005b) defines seven soil types, as detailed in Table 5.4. Where data are available, sites are classified on the basis of the shear wave velocity, $v_{s,30}$, which represents the average value of propagation velocity of S waves in the upper 30m of the soil profile at a shear strain of 10^{-5} or less. Where such geophysical data are not available, the soil profile is determined from standard penetration test (SPT) N values, as usually determined during site investigations.

Ground profile Type A represents a reference condition where the surface response is equal to the response at bedrock level. Ground profiles B to E represent increasing depths and softnesses of soil strata. Ground types S1 and S2 are profiles with the potential for liquefaction and thus require additional study in order to determine an appropriate level of amplification. Design response spectra defined for the five ground types A to E are shown in Figure 5.24. The peak acceleration for the softest ground Type E is 40% greater than that for Type A; this highlights the influence of soil strata on the amplitude of ground surface motions.

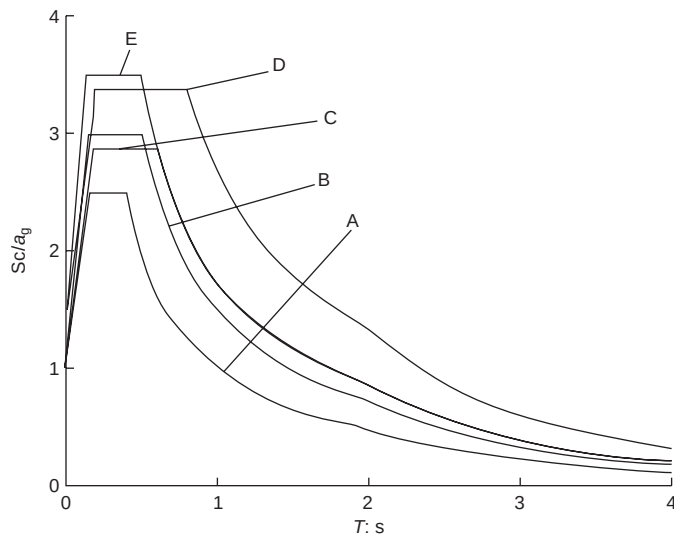


Figure 5.24 Type 1 design response spectra for ground types A to E (5% damping) in accordance with Eurocode 8-1 (BSI, 2004)

Methods of seismic analysis for bridges

A considerable number of methods of varying complexity are available to determine the seismic forces in structures. The method to be adopted in a particular instance will usually be guided by the requirements of the particular seismic code being followed. The most commonly adopted methods are described in the following sections in order of increasing complexity and computational demand. Further details on numerical modelling aspects are provided in NAFEMS texts.

Single-mode analysis

In seismic design codes, SDOF analysis is often termed equivalent static analysis. It is the most basic form of analysis and is

appropriate for a bridge frame whose response is captured by a predominant translational mode of vibration. The predominant natural frequency of the SDOF system, shown in Figure 5.25, is determined using the methods described earlier in this chapter. In undertaking this calculation, it is critical to ensure that account is made for all the relevant mass from structural, non-structural and live loading. In addition, the cracked flexural stiffness of columns should be taken into account, to avoid overestimates of the natural frequency.

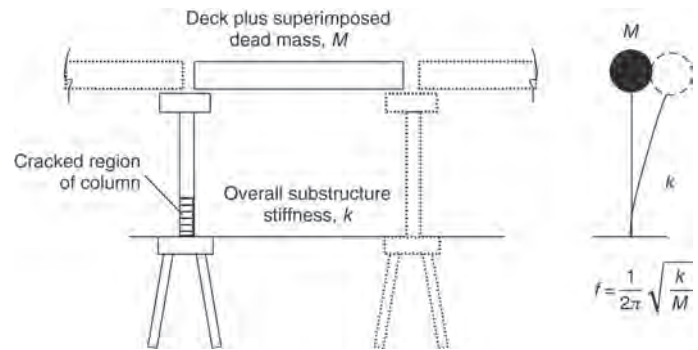


Figure 5.25 Bridge frame represented as a SDOF system

Having found the predominant natural frequency, f , the normalised spectral acceleration $S_a(T)$ is determined from the design response spectrum. The equivalent static force, $F_d(T)$ applied at the location of the mass M is logically

$$F_d(T) = MS_a(T)A_d \quad (5.62)$$

where A_d is the design peak ground acceleration.

Thus, the seismic forces in the substructure elements can be determined and combined with the other static forces acting on the structure.

Multimodal response spectrum analysis (RSA)

Multi-degree of freedom (MDOF) analysis is appropriate to most structures that exhibit predominantly linear behaviour. When used with design response spectra, as opposed to a single event spectrum, the method produces force and displacement maxima in the structure. One disadvantage of this approach is that the nature of the method means that RSA is unable to produce coexistent forces in a member and thus may result in undue conservatism in design.

The technique of RSA is to decouple the modal responses within the structure and to determine the spectral acceleration for each mode. The first step in the analysis of a MDOF system is to find its free vibration frequencies and mode shapes. These are readily obtained through eigenvalue analysis in most structural analysis programs by solving the eigenvalue equation:

$$K\hat{u} - \omega^2 M\hat{u} = 0 \quad (5.63)$$

where $\hat{u}$ represents the amplitude of vibration. Its solution for a system having N degrees of freedom yields a vibration frequency ω_n and a mode shape vector ϕ_n , which represents the

relative amplitude of motion for each of the displacement components in mode n . The dynamic analysis of MDOF systems can therefore be simplified to the solution of Equation 5.63 for each mode. For a structure having N degrees of freedom, the maximum earthquake force, $F_{n\max}$, for mode n is

$$F_{n\max} = \mathbf{M} \phi_n \frac{L_n}{\phi_n^2 \mathbf{M} \phi_n} S_{an}(T) \quad (5.64)$$

where

$\mathbf{M}$ is the mass matrix of the structure with N degrees of freedom
 ϕ_n is the mode shape vector representing the relative amplitudes of motion for each displacement component in mode n
 L_n is the earthquake participation factor and represents the proportion of the total structure mass that is participating in mode n
 $S_{an}(T)$ is the spectral acceleration for mode n .

As the modal maxima do not necessarily occur at the same time, some form of probabilistic combination of the modal responses is required. The most commonly adopted superposition formulae are the square root of the sum of the squares (SRSS) and the complete quadratic combination (CQC). The forces in an N DOF structure using the SRSS rule would be

$$F_{a\max} = \sqrt{u_{a1\max}^2 + u_{a2\max}^2 + \dots + u_{an\max}^2} \quad (5.65)$$

This approximation assumes that all modes are statistically independent; however, for most real structures, coupling of modes makes the SRSS procedure unduly conservative. The CQC method is based on random vibration theories and has found wide acceptance by most engineers; it has been incorporated as an option in most modern computer programs for seismic analysis. The CQC method addresses the superposition of modes at close frequencies through the introduction of cross-modal coefficients, such that the maximum forces in the N DOF structure are given by the double summation

$$F_{a\max} = \sqrt{\sum_i \sum_j F_{ki} \rho_{ij} F_{kj}} \quad (5.66)$$

where F_{ki} is the modal force associated with mode i . The double summation is conducted over all modes. Similar equations can be applied to node displacements, relative displacements, base shears and overturning moments. The cross-modal coefficients, ρ_{ij} , are functions of the duration and frequency content of the loading and of the modal frequencies and damping ratios of the structure. For a structure with constant modal damping, ξ , the cross-modal coefficient ρ_{ij} may be approximated as

$$\rho_{ij} = \frac{8\xi^2(1+r)r^{32}}{(1-r^2)^2 + 4\xi^2r(1+r)^2} \quad (5.67)$$

where r is the frequency ratio of adjacent modes (<1).

When there is little coupling between modes (i.e. low damping or well-separated modal frequencies), the CQC method reduces to the SRSS method.

To ensure that the structural response is adequately captured within a multimodal analysis, it is normal to ensure a minimum of 90% mass participation in the three orthogonal directions.

The number of modes considered in the analysis should be increased until sufficient mass participation is achieved. This criterion is stated in most seismic codes and is based on the total mass of the analysis model, which may include masses representing superimposed dead and live loads on the structure. It is sometimes difficult to achieve 90% mass participation in the vertical direction when large footing or pile caps on stiff foundation springs are included. In this situation, it may be appropriate to accept lower vertical mass participation if it can be proven that the 'missing' mass does not contribute to the response of the columns or bridge superstructure.

Combining orthogonal effects

Bridges must be capable of resisting seismic excitation occurring in any axis; however, peak excitations in the three orthogonal axes are not concurrent. It is reasonable to assume that motions that take place during an earthquake have one principal direction or, during a finite period of time, when maximum ground acceleration occurs, a principal direction exists. For most structures this direction is not known and, for most geographical locations, cannot be estimated. Therefore, the only rational earthquake design criterion is that the structure must resist an earthquake of a given magnitude from any possible direction. In addition to the motion in the principal direction, a probability exists that motions normal to that direction will occur simultaneously. In addition, because of the complex nature of three-dimensional wave propagation, it is valid to assume that these normal motions are statistically independent.

A practical treatment adopted by most codes of practice is to prescribe orthogonal combinations in terms of percentage contribution in each axis, the most popular of which is the 100/30/30 rule. In other words, the full peak response in the principal direction is combined with 30% of the response in the other two orthogonal directions. Thus, it is usually necessary to determine three seismic combinations where each orthogonal axis in turn is considered as the principal direction of excitation. In some codes, such as Eurocode 8 (BSI, 2004; 2005b), the SRSS rule is applied to the combination of orthogonal effects, as it yields approximately the same results as the 100/30/30 percentage combination.

Menun and Der Kiureghian (1998) have proposed the CQC3 method for the combination of the effects of orthogonal spectra. While not widely adopted in bridge codes, the CQC3 method produces results that are not a function of the user-selected axis reference system.

Limitations of RSA and some refinements

While multimode RSA is a very powerful and widely adopted method of seismic analysis, it is inherently linear and thus cannot be used where structural elements exhibit significantly non-linear behaviour. In particular, the analysis of structures that incorporate seismic isolation or damping devices must be undertaken using non-linear time history analysis.

The use of Ritz vectors in preference to traditional modal analysis was proposed by Wilson (1984) and is incorporated in an increasing number of structural analysis packages. This frequency-domain procedure is able to cope with limited non-linearity, such as that produced by tension- or compression-only elements. Ritz vector analysis therefore allows some non-linear problems to be analysed in a fraction of the time that would be required for time-domain analysis.

Care should be taken when using RSAs on long bridges or multispan viaducts, as the method assumes that all foundations experience identical ground motions. This will not be the case for long structures, as seismic wave propagation effects will cause differential motion of column bases. Time history methods, in which a unique ground motion time history is applied to each base, are normally used in this instance.

Time history analysis

The mode superposition methods described here are necessarily limited to the analysis of linear or nearly linear structures. The absorption of earthquake energy during strong-motion events is most effectively achieved through the hysteretic behaviour of certain ductile elements in the structure, usually the columns. Where retrofitting is being undertaken, structure response can be reduced through the addition of dampers or isolators, both of which exhibit significantly non-linear characteristics. To determine the response of a structure during such events, a step-by-step time history analysis is required.

Time history analysis requires the direct numerical integration of the dynamic equilibrium equations. This involves, after the solution is defined at time zero, the attempt to satisfy dynamic equilibrium at discrete points in time. Most methods use equal time intervals at Δt , $2\Delta t$, $3\Delta t$, ..., $n\Delta t$. Many different numerical techniques are available; however, all approaches can be classified as either explicit or implicit integration methods. The most commonly adopted explicit and implicit integration schemes have previously been described in this chapter.

Definition of earthquake time histories

To undertake time history analysis, it is necessary to derive a set of representative accelerograms. Two options are available: to use strong-motion records from previous earthquake events or to generate a series of design response spectrum compatible artificial time histories.

The major source of accelerograms is a worldwide collection of strong-motion records that are available in various forms to the scientific and engineering community. Many earthquake records are available for download over the internet. Some useful sources are:

- US Geological Survey (USGS, 2021)
- Engineering Strong Motion Database (INGV, 2021)
- PEER Strong Motion Database (PEER, 2021).

When searching strong-motion databases, it is necessary to match as nearly as possible the proposed design magnitude, epicentral distance, focal depth, source mechanism and soil with that of the real earthquake records. Most internet databases have the facility to search a vast catalogue of records for these key criteria in order to identify a set of records that fall within the matching parameters specified. Given that each earthquake has unique characteristics, it is important that time history analysis is undertaken for a family of records, so as to be representative. Even with this proviso, the use of historical strong-motion records does not ensure that the seismic analysis will capture the most severe potential response.

Simulated or artificial accelerograms may be generated from the design spectra. White noise or wavelet theory is commonly adopted in their derivation; however, a detailed treatment is beyond the scope of this chapter. Given that the earthquake waves will reach the piers of a bridge at different times, coherent simulated accelerograms must be generated for each pier location, taking into account their relative epicentral distance. There is considerable debate within the engineering community whether artificial accelerograms provide a sufficiently representative set of ground motions to capture peak responses of a structure to a design event earthquake. They should therefore be used with care and with a clear appreciation of their limitations.

Large mass method

The 'large mass method' is a convenient technique for applying earthquake time history to a bridge structural model. A very large point mass, M , is located below the foundations of the bridge piers and attached to it with 6 DOF springs representing the foundation stiffness. The point mass itself is several orders of magnitude larger than the total mass of the rest of the model (typically 10^6). The acceleration time history, $\ddot{x}(t)$, is applied as a time-varying force function to the very large point mass such that $F(t) = M\ddot{x}(t)$. NAFEMS' dynamics primer (Hitchings, 1992) provides a more detailed description of the method.

Performance-based seismic design

A number of seismic codes, including AASHTO LRFD (AASHTO, 2020) and Eurocode 8 (BSI, 2004; 2005b), promote the adoption of performance-based seismic design for bridge structures. Performance-based design involves ensuring that the displacement demand under the design seismic event does not exceed the structure's displacement capacity on the basis of the non-linear moment curvature capacity of the ductile elements of the structure, principally the columns. Given that the technique is mainly a design procedure reliant on the non-linear pushover behaviour, the assessment of displacement demand can usually be undertaken using the frequency and time-domain methods described previously.

Time history analysis incorporating the non-linear behaviour of the plastic hinges may be useful to study the sequence of development and the extent of inelastic behaviour through the structure, and to determine the force demands in non-ductile elements that must continue to perform elastically in the design earthquake. In the regions of plastic hinges, the stress–strain diagrams for both concrete and reinforcement or structural steel should reflect the probable post-yield behaviour, taking into account confinement of concrete, when relevant, and strain hardening or local buckling effects for steel. The shape of hysteresis loops should be properly modelled, taking into account strength and stiffness degradation and hysteretic pinching, if indicated by appropriate laboratory tests.

Background and guidance on analysis methods for performance-based design is provided in FEMA-440 (FEMA, 2005) and the LRFD seismic design guidelines (MCEER, 2001).

Dampers and seismic isolators

For low levels of mechanical damping, it is sometimes acceptable to approximate the non-linear force–deformation relationship as a single linearised stiffness. In this situation, the stiffness of the seismic isolator units is represented by a secant stiffness consistent with the maximum deformation, as shown in Figure 5.26. The structure can therefore be analysed using linear elastic frequency-domain methods, with the obvious advantages in terms of the adoption of design response spectra and computational efficiency.

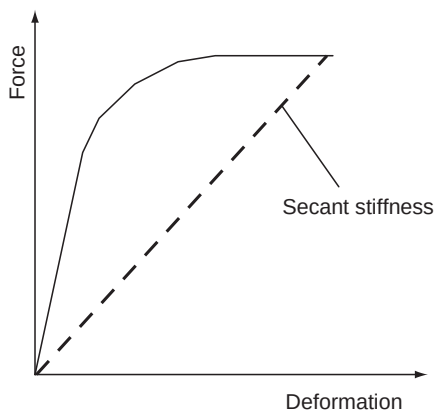


Figure 5.26 Damper equivalent linear secant stiffness

Seismically isolated structures with very long periods or large damping ratios require a non-linear dynamic analysis because the analysis procedures using an effective stiffness and damping might not properly represent the effect of isolation units on the response of the structure. The model for non-linear analysis must represent the actual hysteretic relationships for the isolator units and should account for temperature dependence in the case of fluid viscous dampers.

Human- and vehicle-induced vibration

Human-induced vibration may be a significant consideration for some pedestrian bridges, particularly for the long-span lightweight decks that are increasingly being designed as signature structures. The phenomenon of ‘synchronous pedestrian lock-in’ excitation has been observed on a number of bridges, including the much publicised Millennium Bridge across the River Thames in London.

The passage of highway vehicles or railway rolling stock over bridges can also induce resonant responses that are detrimental to ride quality or track stability, or result in fatigue of structural members over time.

Design codes are increasingly providing guidance on mitigating these undesirable vibrations. The following sections summarise both the theoretical background and the practical implications of these codes to the bridge engineer.

Pedestrian traffic

The motion of walking and running may give rise to considerable dynamic loading on pedestrian structures such as footbridges. These forces can be characterised by the pacing rate, the forward speed and, specifically, by the time function of the loading.

The pacing rate dominates the resulting dynamic load. It is sometimes given as footfalls per second, but its nature as loading frequency is more usefully expressed in hertz. For normal walking on a horizontal surface, the pacing rate of an adult ranges between 1.5 and 2.5 Hz. Assuming a Gaussian normal distribution around a mean of 2.0 Hz yields a standard deviation of 0.13–0.18 Hz. For jogging, the mean pace rate varies from 2.4 to 2.7 Hz and for sprinting it may be as high as 5.0 Hz. Observation suggests that it is rare for pacing rates above 3.5 Hz to be experienced on public pedestrian bridges.

	Pacing rate: Hz	Forward speed: m/s	Stride length: m
Slow walk	~1.7	1.1	0.60
Normal walk	~2.0	1.5	0.75
Fast walk	~2.3	2.2	1.00
Slow running (jog)	~2.5	3.3	1.30
Fast running (sprint)	>3.2	5.5	1.75

Table 5.5 Interrelationship between human pacing rate, stride length and forward speed

The speed or velocity of pedestrian propagation is coupled with the pacing rate through the stride length. Naturally, different people may possess quite different stride lengths and paces for the same forward speed. Table 5.5 summarises the interrelationship between pacing rate, stride length and forward velocity based on a number of studies.

When walking, a person exerts a vertical and horizontal time-varying force, the characteristics of which depend on pacing rate, the person's weight, gait and sex, the type of footwear being worn and the characteristics of the walking surface. An analytical treatment of the stochastic nature of pedestrian dynamic loads in a reliability based design approach is described by Živanović *et al.* (2007).

Time function of the vertical load

Given the multitude of parameters, the results of different investigations vary greatly, and are also influenced by the test procedure and measuring technique adopted. The influence of pacing rate, footwear and floor surface on the development of the dynamic vertical load was thoroughly examined by Galbraith and Barton (1970), who revealed that the latter parameters are of minor importance compared with the pacing rate. The shape of the load time function for walking at a medium pacing rate is more or less that of a saddle; the two observable load maxima result from stepping with the heel and pushing off with the ball of the foot. This feature disappears with increasing pacing rate and degenerates to a single maximum of sharp rise and descent when the person is running. From low to high pacing rates, the width of the signal decreases, and the load maximum increases. While for strolling with a frequency below 1 Hz the maximum load hardly exceeds the weight of the person, it increases by a quarter to a third for 2 Hz and by a half at around 2.5 Hz; at about 3.5 Hz, the maximum reaches about double the weight of the test person. For relatively high pacing rates (above 2.5 Hz), somewhat larger maxima are indicated, and for fast running the maximum load can increase to three times the runner's weight. There is a clear relation between pacing rate and the duration of foot contact with the ground. Vertical pedestrian loading with continuous ground contact can be taken as

$$F_p(t) = W + \Delta W_1 \sin(2\pi f_p t) + \Delta W_2 \sin(4\pi f_p t - \phi_2) + \Delta W_3 \sin(6\pi f_p t - \phi_3) \quad (5.68)$$

where

W = weight of the person (generally assumed to be 800 N)

f_p = pacing rate

ΔW_1 = load component (amplitude) of 1st harmonic, taken as 0.4W for $f_p = 2.0$ Hz

0.5W for $f_p = 2.4$ Hz with interpolation for intermediate values

ΔW_2 and ΔW_3 are the load components (amplitude) of the 2nd and 3rd harmonics, taken to be approximately 0.1W for $f_p = 2.0$ Hz

ϕ_2 and ϕ_3 are the phase angles of the 2nd and 3rd harmonics relative to the 1st and 2nd harmonics ($\approx \pi/2$ for $f_p = 2.0$ Hz)

This is shown graphically in Figure 5.27.

Consequently the analysis of the vibration effects of a single pedestrian on a structure is ideally suited to modal analysis using the frequency response function method previously described.

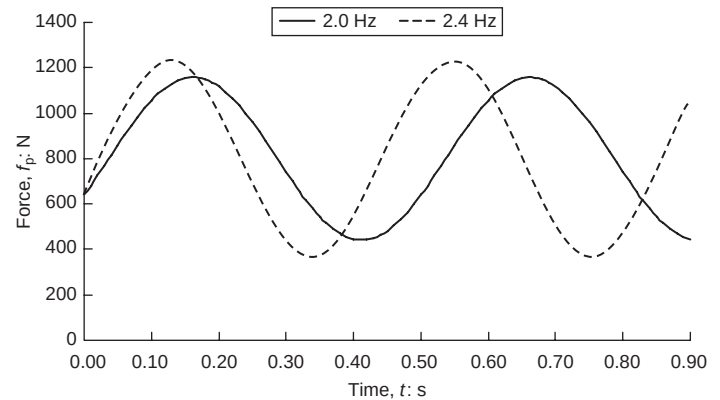


Figure 5.27 Idealised vertical forcing function $F_p(t)$ due to walking

Time function of the horizontal load

The horizontal loading from human walking is much smaller than in the vertical direction; however, for some laterally flexible structures, it might still present a critical condition. The lateral excitation frequency due to the sway of a person's centre of gravity occurs at half the pacing rate in the range 0.75–1.5 Hz.

The effect of groups of people

Pedestrian dynamic loading must account for the effect of groups of people on a structure; indeed, many footbridges are subjected to crowd loading, for example, due to large sporting events. In this situation, it is necessary to characterise the effect of group behaviour on the dynamic loads that are experienced by the structure. Based on data obtained by both observation and through experiment, a number of key behaviours are exhibited by pedestrian groups.

The magnitude of crowd loading on a structure is limited by the density of uninhibited walking. Research suggests that this density is approximately 1.6–1.8 persons/m², equating to a static load of 1.1–1.4 kN/m². A more realistic limit of 1 person/m² is necessary to obtain the maximum dynamic effects on pedestrian bridges.

It is difficult to derive a mathematical description for the magnitude of vertical loading due to groups of pedestrians. Matsumoto *et al.* (1978) suggested that, where the arrival probability of pedestrians takes the form of a Poisson's distribution, an enhancement factor for group loading, m , can be applied to the forcing function for a single pedestrian, such that

$$m = \sqrt{\lambda \cdot T_0} \quad (5.69)$$

where λ is the mean flow rate over the width of the bridge at a particular point in time and T_0 is the time required for a single pedestrian to cross the bridge. Therefore, $\lambda \cdot T_0$ represents the total number of pedestrians on the bridge at the assumed mean flow rate. This relationship assumes that there is no interaction or positive feedback of the pedestrian group to bridge vibrations.

Synchronous lateral excitation

On a number of pedestrian bridges with lateral and torsional deck natural frequencies below 1.5 Hz, substantial vibration amplitudes have been observed. The loading effect has been found to be due to the synchronisation of lateral footfall within large crowds of pedestrians on the bridge. This so-called ‘synchronous lateral excitation’ occurs because pedestrians find it more comfortable to walk in synchronisation with any initial lateral vibrations to maintain their balance. In doing so, they ‘lock in’ to the natural frequency of the bridge and their footfalls provide positive feedback to the initial sway, further increasing the vibration amplitude. As the amplitude of vibration increases, more individuals correlate their pacing rate to the natural frequency of the motion.

Studies by Blanchard *et al.* (1977) of the lateral vibration problems experienced on the Millennium Bridge in London suggested that, for a given level of damping, the limiting number of pedestrians on a bridge to avoid instability, N_L , is

$$N_L = \frac{8\pi c f M}{k} \quad (5.69)$$

where

c = critical damping ratio

f = natural frequency of bridge

M = modal mass.

The lateral walking force coefficient, k , is the correlated force per person normalised to the local velocity of the structure during synchronous excitation. On the Millennium Bridge, k was found to be of the order of 300 Ns/m over the frequency range 0.5–1.0 Hz. Knowing the limiting number of pedestrians on the bridge, N_L , and assuming a maximum pedestrian density as described previously, it is possible to determine whether additional structural damping is required in order to limit effects of synchronous lateral excitation.

Vandal loading

In addition to the vibrations that can be excited by normal pedestrian group behaviour, it is unfortunately also necessary to consider the possibility of deliberate attempts to excite the bridge’s natural frequencies through the action of groups of vandals. This type of loading is caused in the vertical direction by marching or skipping in time and in the horizontal direction by rhythmic shifting of body centre of gravity by a group of people. It is now common for lightweight or particularly long-span footbridges to be subject to commissioning tests to evaluate the structure’s response to so-called ‘vandal’ loading.

Existing design recommendations

The design basis section of the Eurocodes, BS EN 1990 (BSI, 2002), makes the following recommendations in terms of maximum accelerations to ensure satisfactory dynamic behaviour of footbridges:

0.7 m/s² for vertical vibrations

0.2 m/s² for horizontal vibrations due to normal use

0.4 m/s² for exceptional crowd conditions.

A verification of the comfort criteria is also recommended if the fundamental frequency of the deck is less than

5 Hz for vertical vibrations

2.5 Hz for horizontal (lateral) and torsional vibrations.

The UK National Annex to Eurocode 1 Part 2 (BSI, 2020) provides further guidance on defining the dynamic actions on structures due to pedestrian walking, jogging and the effects of crowded conditions. It also recommends maximum accelerations at the serviceability limit state for use in design, depending on site usage, route redundancy and structure height in the range 0.5–2.0 m/s².

The guidance also provides an empirical method for checking that structures do not exhibit unstable lateral responses due to synchronous lateral excitation. This check involves the calculation of a pedestrian mass damping parameter, D , given by:

$$D = \frac{m_{\text{bridge}} \zeta}{m_{\text{pedestrian}}} \quad (5.71)$$

where

m_{bridge} is the mass per unit length of the bridge

$m_{\text{pedestrian}}$ is the mass per unit length of pedestrians for the relevant crowd density (depending on site usage), assuming that each pedestrian weighs 70 kg

ζ is the structural damping expressed as a damping ratio.

The calculated pedestrian mass damping parameter can then be compared with a stability boundary line on a graph plotted against the lateral mode frequency. It is worth noting, however, that this stability boundary is based on a limited number of test measurements for footbridges with a lateral frequency in the range 0.5–1.1 Hz. Therefore, engineers should exercise caution in its use and undertake further studies, particularly where the bridge lateral frequency is below 0.5 Hz.

Steel Construction Institute Advisory Note AD 428 (Hendy C.) highlights a particular issue for narrow half-through footbridges. In a number of instances it has been found that such bridges having a low torsional stiffness can exhibit significant excitation in a lateral-torsional mode when pedestrians walk eccentric to the deck centreline and even more significantly when they are deliberately shaken. In a number of cases this gap in the standards has led to the need to retrofit dampers and/or provide additional stiffening. As a result of this experience the 2020 amendment to UK National Annex to EN 1991-2 recommends that for decks with a lateral or torsional frequency less than 2.5 Hz horizontal accelerations measured at deck level should be limited to 0.2 m/s² for the walking and jogging load models. It is expected that with these loads be positioned on the deck to give the highest accelerations which is likely to be offset from the bridge centreline as far as is practically possible given the deck cross-section geometry.

In view of the inherent uncertainties in predicting how human users will react to footbridge vibrations, it is recommended

that consideration should be given in the design for the future installation of dampers where the vertical and lateral natural frequencies are less than 1.5 Hz and 3.0 Hz, respectively.

A number of codes of practice detail the physiological effects of vibrations on humans themselves. These include ISO 2631 (ISO, 2003) and BS 6472 (BSI, 1992), albeit they specifically relate to the use of buildings. In both these standards, exposure limits to vibrations are defined in terms of RMS accelerations in the vertical and horizontal directions. In some situations, for example, during the construction of cable-stayed bridge cantilevers, it may be necessary to consider whether workers' exposure to structural vibrations are likely to result in motion sickness, leading to health concerns or reduced productivity.

Damping

It is recommended that the addition of mechanical damping devices be considered a last resort when determining a strategy for reducing resonant response in a new structure. Owing to the ongoing maintenance liability that mechanical dampers attract, it is far preferable to ensure that the predominant natural frequencies of the structure are outside the pedestrian excitation frequency range in both vertical and lateral directions. Where this is not a feasible approach, or where an existing bridge is experiencing unacceptable levels of vibration, the addition of dampers can be very effective.

Railway traffic

The dynamic response of bridges under the passage of rail rolling stock is becoming an increasingly important consideration, particularly for structures on high-speed routes. Historically, the dynamic effects of rolling stock have been accounted for by means of impact factors applied to the static wheel loading in combination with limiting deflections and structure frequency, as prescribed in the relevant codes of practice. This treatment has been perfectly adequate for structural design purposes; however, it provides no quantitative measure of bridge deck accelerations, which ultimately have an influence on ride quality and the long-term stability of ballasted permanent way.

Furthermore, application of the 'impact factor' approach to long-span or more unusual forms of rail bridge may not be appropriate, given that the studies that have produced such guidance hitherto have focused on the most common structural forms. Given that fatigue is a significant factor in the design of steel railway bridges, rigorous assessment of the stress regime at fatigue-critical details is only possible through dynamic analysis.

The principal factors affecting the dynamic response of a rail bridge are the speed of traffic across the bridge, the natural frequencies of the whole structure and relevant elements of the structure, including the associated mode shapes along the line of the track, the number of axles, axle loads and the spacing of axles, the damping of the structure, vertical irregularities in the track and the unsprung or sprung mass and

suspension characteristics of the rolling stock. Given that most rolling stock suspension systems are relatively stiff and that high-speed track is usually maintained to eliminate significant track irregularity, it is often sufficient to ignore these factors within dynamic response calculations. Track roughness and train-structure coupling effects can certainly be disregarded when checking for severe resonance due to axle-group passing frequency matching the vertical bending frequency of a span.

Analysis methods

The passage of rolling stock over a bridge may be analysed in the time domain as a transient load history. This method requires significant computational effort, given the number of analyses needed to adequately assess the full range of vehicle speeds.

A more efficient analysis method adopts modal superposition techniques, calculating the total structure response as the linear combination of modal responses computed using single degree of freedom methods. The inputs to the calculation are the modal force history for each mode in conjunction with the damping and frequency of each mode. Key to the technique is transformation of the structural system from physical to modal coordinates, achieved using standard eigenvalue analysis methods. Truncation of this modal series to the low-frequency modes leads to a greatly reduced set of equations, which can be solved independently by making the assumption that modes are uncoupled. The modal responses are easily computed using single degree of freedom vibration response formulae, such as Duhamel's integral, as described in this chapter. Finally, the modal displacements are transformed back to physical motions and forces through the eigenvectors, or mode shapes. This leads to an evaluation of modal displacement of each mode at each time step. The structural response is then calculated at each time step by modal superposition. This method has been effectively implemented as a post-processing procedure to a number of software packages capable of eigenvalue analysis.

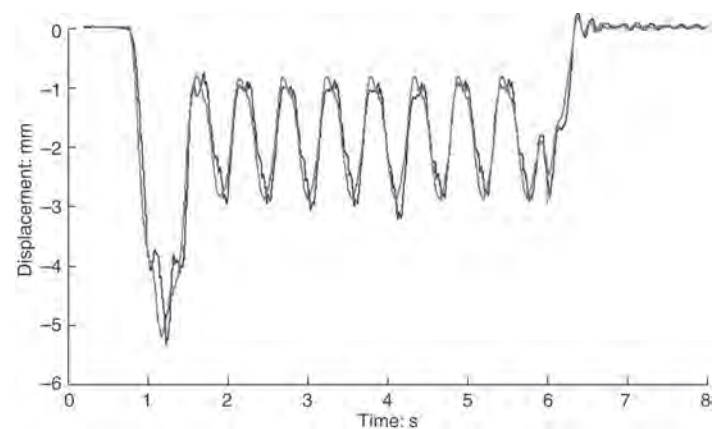


Figure 5.28 Comparison of measured and calculated displacement histories at midspan of a railway bridge span

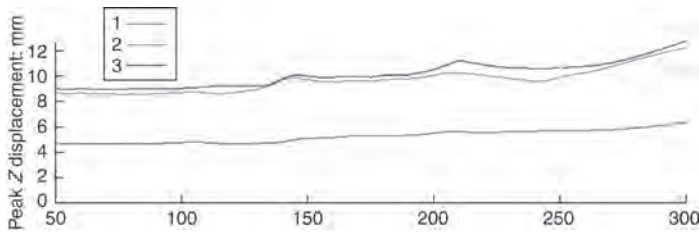


Figure 5.29 Peak response at midspan of a bridge deck for a range of train speeds

The method is powerful in that it uses modal analysis to determine both peak structural responses for the range of train speeds under consideration and time histories at particular speeds. Figure 5.28 shows a comparison of the measured and calculated peak vertical displacement history and Figure 5.29 shows the peak calculated responses for a range of train speeds.

Using these dynamic methods, it is possible to calculate structure-specific dynamic magnification factors for subsequent structural design.

Design recommendations

Eurocode 1 Part 2 (BSI, 2020) provides a procedure for evaluating the need for dynamic analysis of rolling stock on railway bridges. This procedure is reproduced in Figure 5.30. The Eurocode defines upper and lower bound thresholds of structure natural bending frequency beyond which dynamic analysis is required. These thresholds are reproduced in Figure 5.31 for ease of reference.

The limiting design values for dynamic response of rail bridges are usually defined in terms of peak vertical accelerations, rate of deck twist and relative angular deformation between adjacent decks. The design basis section of the Eurocode gives the following guidelines for ensuring traffic safety and passenger comfort.

Traffic safety

Peak values of bridge deck acceleration at the track positions should not exceed 3.5 m/s^2 for ballasted track and 5.0 m/s^2

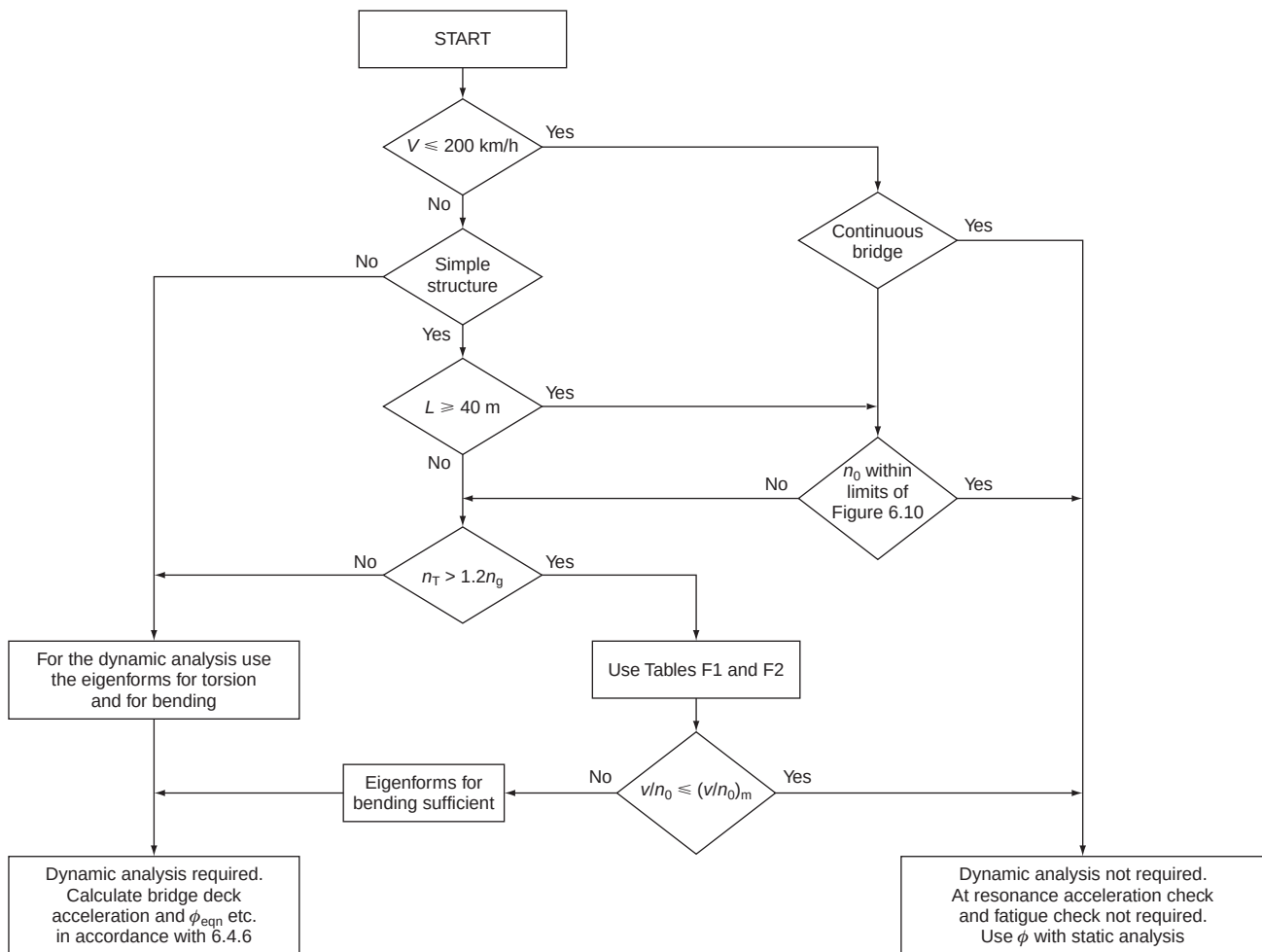


Figure 5.30 Procedure given in Eurocode 1-2 (BSI, 2020) for determining whether dynamic analysis of rolling stock loading is required

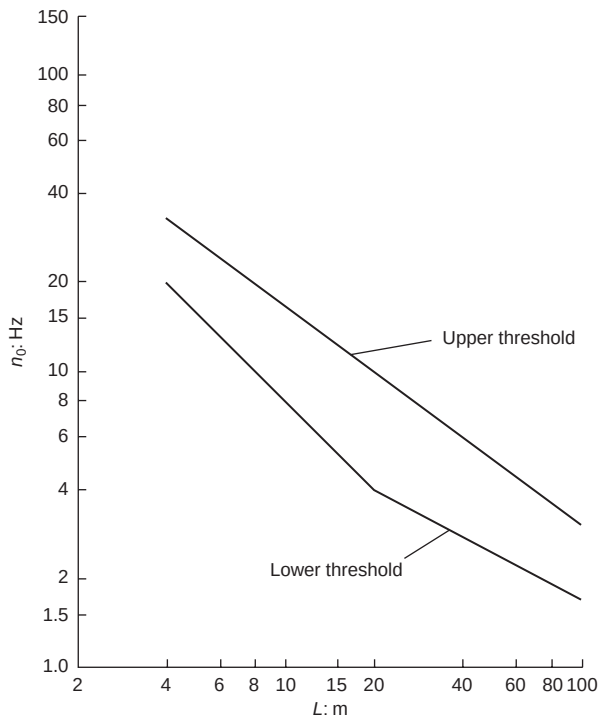


Figure 5.31 Range of structure predominant natural frequency, n_0 , for which a dynamic analysis is not required (with $V \leq 200$ km/h), given in Eurocode 1-2 (BSI, 2020, Figure 6.10)

for slab track. The former limitation is designed to prevent dynamic ballast instability.

Deck twist, t , is defined in terms of the maximum twist over standard rail gauge, s (1435 mm), measured over a length of 3 m. Table 5.6 gives the limits to deck twist recommended by the Eurocode.

Speed range V : km/h	Maximum twist t : mm/3m
$V \leq 120$	$t \leq 4.5$
$120 < V \leq 200$	$t \leq 3.0$
$V > 200$	$t \leq 1.5$

Table 5.6 Eurocode 0 (BSI, 2002) limits on maximum deck twist, t during the passage of rolling stock

The relative angular rotations at movement joints between adjacent decks or at abutments are limited in UIC 776-3R (UIC, 1989) to 0.01 rad for ballasted track and 0.005 rad for slab track.

Passenger comfort

To ensure passenger comfort, vertical acceleration within the coach crossing the structure must be limited. This acceleration is a function of the bridge deck behaviour and the design of the rolling stock, in terms of its suspension system. Other factors, such as rail or wheel irregularities, will also have an

effect on the level of passenger comfort experienced within a coach. It is possible to undertake a full dynamic analysis of the complete coach and structure system using software, such as VAMPIRE; however, this is specialist analysis more usually applied to the design of rolling stock than the design of bridges. The Eurocode (BSI, 2002) provides rule-of-thumb guidance for achieving ‘very good’ comfort levels in terms of limits on vertical deck deflection with respect to structure span, as shown in Figure 5.32. Guidance is given within the Eurocode for calculating the vertical deflection, δ in conjunction with modification factors for other structure forms and levels of ride comfort.

Noise mitigation

A significant consideration for urban railway systems is the environmental impact of structure-borne noise, owing to the passage of trains. The magnitude of the noise can be increased by the vibrations of the structure and its components. It is now common for detailed studies to be undertaken to evaluate the impact of different structural forms on structure-borne noise. For the West Rail Viaducts in Hong Kong, it was found that low-frequency noise radiation could be significantly attenuated by placing the bridge deck webs directly beneath the line of the rails. Such studies are informative at the conceptual stages of a project to enable the form of structure to be tailored to the project environmental constraints.

Highway traffic

Typical highway bridges are seldom vulnerable to detrimental vibrations, owing to their configuration and mass, but it is occasionally necessary to check the response of a structure to vibrations caused by passing cars and trucks. The frequency of vibrations generated by highway traffic extends over a wide range, depending on the vehicle configuration, speed and maintenance, as well as the smoothness and state of the running surface. Highway traffic have been found to generally create vibrations with a frequency above 5 Hz, which may excite individual elements on bridges, such as stay cables, external tendons or parts of the deck furniture, but are seldom critical for the global behaviour of the bridge. A possible exception is the combination of severe running surface defects with heavy traffic, which can generate significant impact loading.

Collision

Accidental collision with bridges occurs mainly from highway vehicles and ships. Bridge designers must consider accidental collision scenarios to ensure that such events do not precipitate collapse. The evaluation of impact is fundamentally based around conservation of energy; however, the interaction of a vehicle or ship with a bridge structure during a collision is a complex dynamic problem.

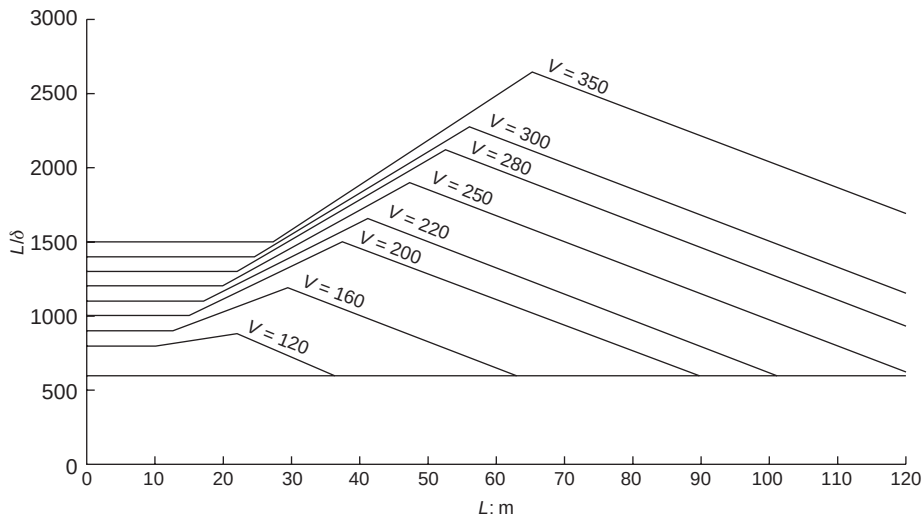


Figure 5.32 Eurocode 0 (BSI, 2002) recommendations for achieving 'very good' passenger comfort by limiting deck deflection, δ , for bridges with three or more successive simply supported spans, L

Vehicle impact

It is not usually necessary to undertake dynamic analysis of vehicle impacts on bridges, as most international bridge codes prescribe quasistatic collision forces based on full-scale testing. The UK National Annex to Eurocode 1 Part 1-7 (BSI, 2015b) details requirements for the application of equivalent static impact forces to bridge substructures and superstructures.

Where retrofitting of older bridges is required to provide enhanced collision protection, the adoption of dynamic impact analysis may be appropriate. Such analysis requires detailed consideration of the non-linear behaviour of both the bridge and the vehicle during a collision event and is thus time-consuming and expensive to undertake. A study by the UK Highways Agency (Ove Arup and Partners, 1995) demonstrated that the impact of heavy goods vehicles on bridge supports is typically characterised by two main pulses. In the tanker impact studied (a 30t tanker at 40 miles per hour), a short pulse of force (~ 10 ms, 7500 kN peak) occurred when the engine, gearbox and drive train struck the column and then a longer pulse (~ 90 ms, 5800 kN peak) was measured when the rear part of the vehicle struck. Furthermore, tests have shown that up to 80% of the vehicle impact energy is absorbed by crushing of the vehicle itself, with the remainder being absorbed by deformation and crushing of the bridge supports. The UK *Design Manual for Roads and Bridges* Standard CS 453 (Highways England, 2020b) provides guidance on the assessment and strengthening of existing bridge supports for vehicle collision.

Ship and barge collision

Where bridges are located in navigable waters, they constitute a hazard to shipping and are thus vulnerable to damage or even destruction in the event of vessel collision. While ship collisions

with bridges are relatively infrequent, they are generally serious, having significant human, economic and environmental consequences. On 9 May, 1980, the *Summit Venture*, a 35 000 DWT bulk carrier, struck the Sunshine Skyway Bridge in Tampa, Florida, causing the collapse of three spans and resulting in 35 fatalities. The IABSE guide by Larsen (1993) provides detailed guidance and useful background information on designing bridges for ship and barge collision.

The evaluation of ship impact forces on bridges during a collision is a complex matter as it depends on both the vessel and bridge characteristics. The most significant parameters are vessel speed and impact angle, hydrodynamic mass, water depth, vessel type and hull-crushing characteristics, and bridge size, shape and stiffness.

Kinetic energy from the vessel during an impact is dissipated through crushing of the vessel hull in combination with deformation of the bridge structure. Where the collision occurs at an angle, energy is also dissipated through contact friction between the vessel hull and the bridge.

Equivalent static impact forces are recommended by the AASHTO Guide Specification (AASHTO, 2020) or by Eurocode 1 Part 1-7 (BSI, 2015b) and are commonly adopted in bridge design. The empirical relationships proposed are based on the results of both physical and mathematical modelling. It is usually adequate to follow these recommendations without recourse to more detailed dynamic analysis.

To undertake dynamic analysis of bridge response during a ship collision event, it is necessary to establish load-indentation relationships for the particular vessel types under evaluation. These relationships describe the variation of impact forces with respect to deformation of the vessel hull and are

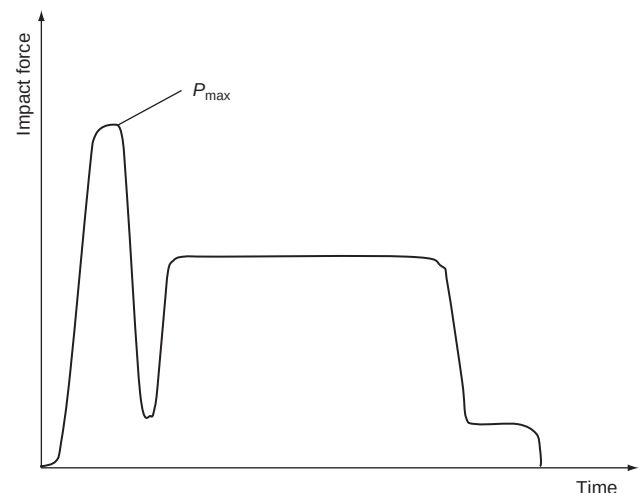


Figure 5.33 Ship impact force time history derived from collision tests conducted by Woisin and Gerlach (1970)

described in specialist literature, the most useful of which are summarised in the IABSE guide (Larsen, 1993). Force time histories of ship collisions have also been investigated (Woisin and Gerlach, 1970), as shown in Figure 5.33.

Having defined a suitable force time history through model testing or by reference to the literature, dynamic analysis can be undertaken using the incremental techniques discussed in this chapter.

Where required, additional protection to the bridge structure can be provided in the form of timber, rubber steel or concrete fender systems, pile-supported fenders, dolphins, protection islands or tensioned cable systems. The commentary to the AASHTO Guide Specification (AASHTO, 2020) provides examples of each of these systems. The design of collision protection systems is based on conservation of energy principles, where the kinetic energy of the vessel is absorbed by the work done through the deformation resistance of the protective structure. Given the magnitude of the forces involved, plastic deformations are inevitable; therefore, the inelastic load–deformation behaviour of the structure must be established through physical testing or inelastic analysis. The area under the load–deformation curve is the energy capacity of the system. A typical pile-supported fender system and its associated load–deformation characteristics are shown in Figure 5.34.

Where the structure's force resistance is higher than the vessel impact force, crushing of the vessel's hull will occur. If, however, the impact force is greater than the protection structure's resistance, energy will primarily be absorbed through deformation of the protection system. Given that the interaction between vessel and structure is a complex matter, it is often assumed that the protection structure must absorb the majority of the impact energy.

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This chapter has briefly described a number of fundamental concepts and applications of dynamic analysis relevant to bridge engineering. More detailed information can be found in the following references.

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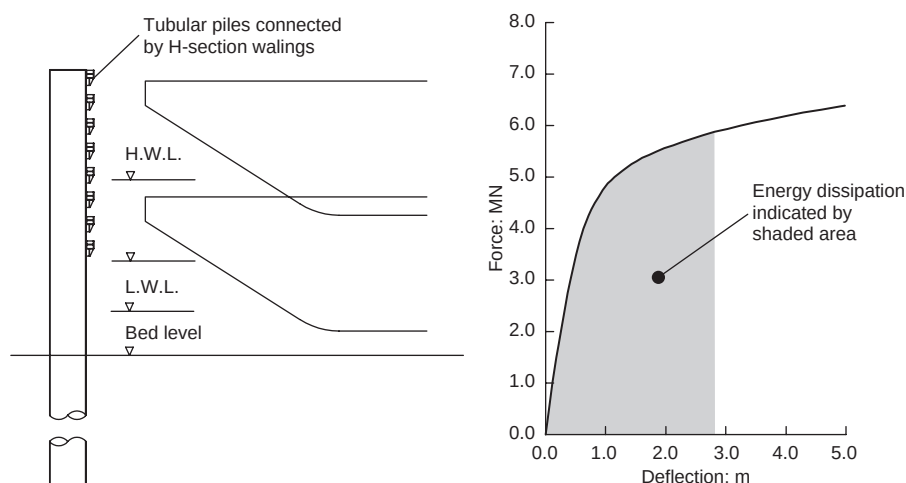


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Chapter 6

Seismic response and design

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Bridges are the transportation network component most vulnerable to damage from natural disasters, compared with roads and railway lines. It is therefore of priority to adequately design new bridge structures and reassess the response of existing bridges in areas subjected to earthquake hazard. This chapter briefly addresses a number of topics related to seismic response and design of bridges, namely damage observations in previous earthquakes, conceptual design and modern seismic codes. Commonly observed bridge failure modes following damaging earthquakes are presented. This shows that despite the advancement in seismic design practice, there are repetitive damage patterns, owing to the increased number of bridges of complex configurations and the heightened consequences of bridge damage in developed societies. Features of layout and configuration that are favourable to controlled and predictable seismic response of bridges are also discussed. Various options available, from foundations through to the superstructure, and connections between various components, are presented and their likely effects on the response are discussed. Finally, a brief review of seismic design codes in Europe, the USA and Japan is presented. The review highlights the differences and their origin, which is an important step towards improved understanding of seismic design procedures.

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Introduction

An efficient transportation system plays a vital role in the development of a modern society, mainly owing to the inter-reliance of various industries and the increased trend for outsourcing various necessary ingredients within a single activity. Hence, transportation networks are referred to as lifelines, the integrity of which has to be protected, alongside water supply, electricity and gas networks. While roads are a most important component of transportation networks, bridges are both more important and sensitive to damage from natural disasters, since roads are more easily repairable and may also be readily bypassed. The closure of a bridge that represents the only or most important link between two areas separated by water or some geological feature (e.g. gorges) would potentially cause very severe consequences for industry, commerce and society as a whole. Recent examples abound as to the effects of earthquake damage to bridges, as discussed in subsequent sections. Two examples are quoted herein of the consequences of the closure of the Oakland Bay Bridge on traffic between San Francisco and Oakland (Loma Prieta earthquake, 1989) and the closure of several of the crossings between Kobe and Port Island (Hogoken-Nambu earthquake, 1995), among several others. Not only did such closures affect the communities in the immediate vicinity of the bridge, but they also had knock-on effects on many other communities,

owing to loss of business and delays in the delivery of essential goods. Table 6.1 gives estimates of economic loss as the result of bridge damage in three major earthquakes. These do not include indirect loss arising from business interruption and lost revenue; however, they serve to confirm the economic significance of bridge damage.

If the economic loss arising from closure of a main arterial bridge is assessed alongside the cost of seismic retrofitting of the structure, the case for the assessment and redesign of bridge structures in seismic areas will be immediately apparent. To emphasise this point, the effect of the San Fernando earthquake of 1971 is considered. Many of the cases of collapse of spans were attributed to the short seating length allowed at seismic joints. The cost of design and installation of restrainers (assuming that other failure modes would not be triggered) would have been a very small fraction of the direct cost of repair, and an even smaller proportion of the total cost including business interruption and loss of revenue. It is therefore of priority to reassess bridge structures in areas subjected to seismic hazard with a view to minimising earthquake damage.

One serious problem facing the earthquake engineering community in reducing the earthquake risk to bridges is that whereas, in general, engineers tend to have a feel for vulnerable parts in buildings and frequently encountered failure modes are common knowledge, they are often less familiar with bridge structures. Therefore, increasing bridge designers'

and earthquake engineers' awareness of the observed repetitive damage patterns following damaging earthquakes is a worthy cause. Moreover, adopting an appropriate concept for the bridge, including sub- and superstructures, is certainly an effective means of reducing drastically complications that may arise at the detailed design stage. Consequently, presenting conceptual design issues in a transparent manner and exploring their relationship with anticipated seismic behaviour would lead to reduced earthquake damage to bridges. Finally, there are considerable discrepancies between the leading international codes for seismic design of bridges. Hence, comparative assessment of codes for the design of bridges under earthquake motion, highlighting the differences and their origin, is an important step towards improving the understanding of the engineering community of the seismic design procedure.

Earthquake	Date	Magnitude: M_w^a	Cost: \$ billion ^b
Loma Prieta	17 October 1989	6.9	1.8
Northridge	17 January 1994	6.7	0.2
Kobe	17 January 1995	6.9	6.5

^aUSGS (2019); ^bBasoz *et al.* (1999) and USGS (2019)

Table 6.1 Economic loss due to bridge damage in selected earthquakes

The objectives of this chapter are limited, and the audience well defined. It is intended for non-specialists but those well aware of issues of dynamic response of structures, inelastic deformation and energy absorption and generalities of earthquake ground motion causes and effects. The scope is confined to fixed reinforced concrete (RC) bridges comprising decks, which may be steel or composite, supported on RC piers. Other configurations are beyond the scope of this short chapter, and the reader is referred to the extensive literature.

Modes of failure in previous earthquakes

Worldwide bridge damage observations

One of the earliest detailed and pictorial accounts of bridge failure is that of Milne (1892), who described the effects of the 'Great Earthquake of Japan, 1891'. This earthquake hit on 28 October 1891 and affected the prefectures of Gifu and Aichi and was felt over an area of approximately 90 000 square miles. The earthquake caused severe damage and some cases of collapse to bridges, such as the collapse of the masonry piers of the Kiso Gawa Bridge and the total collapse of the Nagara Gawa steel truss bridge (Figure 6.1). Several other bridges suffered varying degrees of damage. Since then, many earthquakes caused severe damage to bridges, with very serious consequences, usually more economic than human. Among such earthquakes in Japan are the Great Kanto earthquake of 1923 ($M=7.9$), Nankai in 1946 ($M_L=8.1$), Fukui in 1948 ($M_L=7.3$), Imaichi in 1949 ($M_L=6.4$ and 6.7 ; multiple event), Tokachi-oki in 1952 ($M_L=8.1$), Northern Miyagi in 1962 ($M_L=6.5$), Niigata in 1964 ($M_L=7.5$), Ebino in 1968 ($M_L=6.1$), Tokachi-oki in 1968 ($M_L=7.9$), Miyagi-ken-oki in 1978 ($M_L=7.4$), Nihonkai-Chubu in 1983 ($M_L=7.7$), Kushiro-Oki in 1993 ($M_L=7.8$), Hokkaido Nansei Oki in 1993 ($M_L=7.8$), Hyogo-ken Nanbu in 1995 ($M_L=6.9$) and Tohoku 'Great East Japan' earthquake in 2011 ($M=9.1$).

The Hyogo-ken Nanbu earthquake was particularly damaging to bridge structures in the area, knocking off vital services, such as the Shinkansen line and Route 3 of the Hanshin Expressway (Figure 6.2), and very seriously affecting all other lines in the Kobe area (Elnashai *et al.*, 1995; Kawashima and Unjoh, 1997; Seible *et al.*, 1995).



Figure 6.1 Damage to the Nagara Gawa Railway Bridge, Gifu (Japan) earthquake of 28 October 1891 (courtesy of INGV – National Institute of Geophysics and Volcanology, Italy)



Figure 6.2 Collapse of the Hanshin Expressway elevated road, Hyogo-ken Nanbu (Japan) earthquake of 17 January 1995 (courtesy of National Information Service for Earthquake Engineering, University of California, Berkeley)

In the USA, widespread damage was reported to bridges in the earthquakes of San Fernando of 1971 ($M_S=6.6$; Figure 6.3), Whittier Narrows of 1987 ($M_S=6.3$), Loma Prieta of 1989 ($M_S=7.1$) and Northridge of 1994 ($M_S=6.9$). In particular, the Northridge-inflicted damage on bridges was one of the major sources of financial loss, owing to the severe disruption of transportation lifelines (Broderick *et al.*, 1994; EERI, 1995).

Damage to bridges was also reported from other areas of the world, such as the Philippines earthquake of 1990 ($M_S=7.8$), the Talamanca earthquake of 1991 in Costa Rica, the Guam earthquake of 1993 ($M_S=8.1$), the Adana-Ceyhan earthquake of 1998 in Turkey ($M_S=7.8$), the Kashmir earthquake of 2005

in Pakistan ($M_W=7.6$) and the Maule earthquake of 2010 in Chile ($M_W=8.8$) (Durrani *et al.*, 2005; Elnashai, 1998; Elnashai *et al.*, 2010; Elnashai *et al.*, 2012).

Observed damage patterns

An appreciation of the possible areas of vulnerability in bridges is essential for the development of conceptual designs in seismic areas. Hence, the following is a brief account of the most commonly observed failure patterns from various earthquakes.

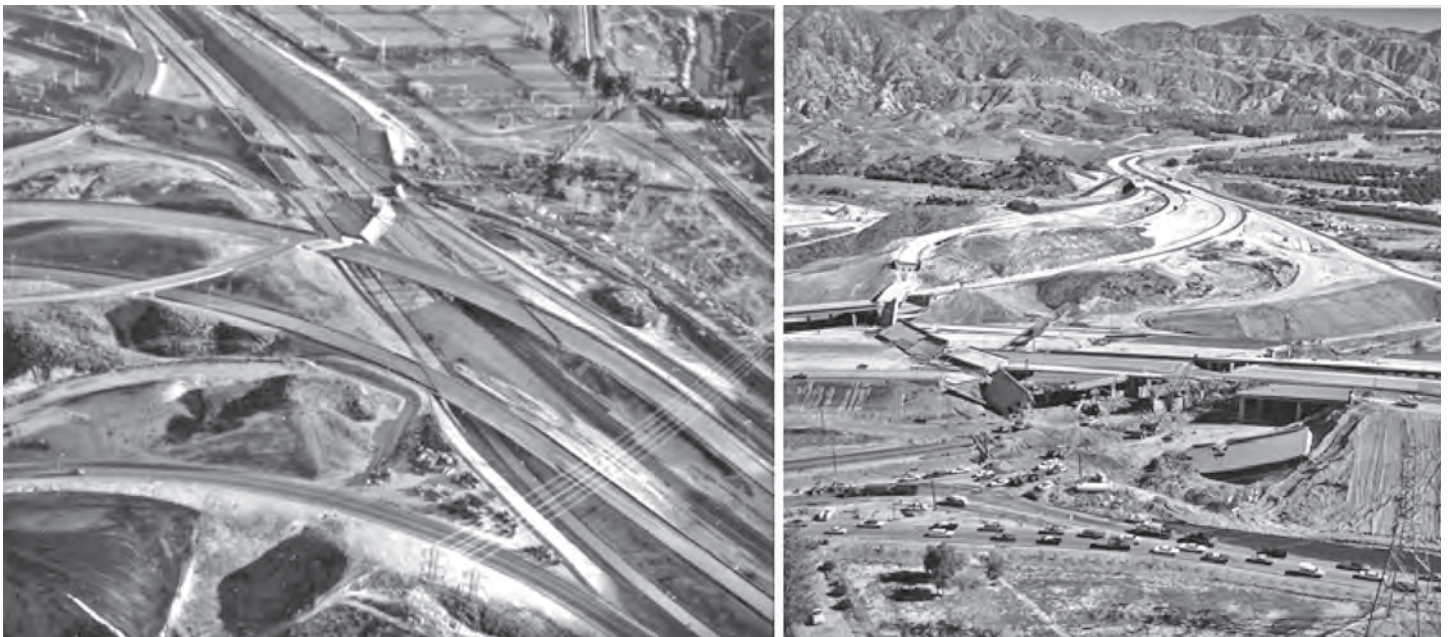


Figure 6.3 Aerial view of the Golden State–Foothill Interchange, San Fernando (USA) earthquake of 9 February 1971 (courtesy of USGS)



Figure 6.4 Soil boils and cracks at pier foundations of Nishinomiya-ko Bridge in the 1995 Kobe (Japan) earthquake (courtesy of USGS)

Foundation soil

Soil lateral spreading or liquefaction imposes large deformation demand on bridge components (Figure 6.4), such as piles, abutment walls or simply supported deck spans. Some bridges founded on soft ground in the Kobe area suffered damage to piles, owing to negative skin friction resulting from soil failure. Also, approach structures and abutments have suffered substantial movement, owing to soil slumping. Liquefaction was widespread in the Niigata earthquake, especially in the alluvial plains of the Shinano and Agano rivers. This caused significant damage, owing to large movements of pier and abutment foundations. Also, railway and highway bridges were affected by large ground displacement in the Costa Rica earthquake, where caisson and pier movements of 2.0 and 0.8m, respectively, were observed.

Foundations and piles

Footings and piles are sometimes underdesigned for earthquake loading, since the overstrength of piers they support would not have been taken into account. In the Kanto earthquake, tilting of foundations of mass concrete was observed, indicating inadequate consideration of overturning. In Kobe, several investigated cases showed damage to footings, which cracked, mainly in shear. Several piles were also damaged. It is relatively difficult to ascertain the cause of failure of subgrade structures, but it is likely that such failures are due to unconservative estimates of the actions transmitted from the piers to the foundations. Also, the point of contraflexure of the pile–footing–pier system is often misplaced; hence, the critical sections are not treated as such.

Substructure

Probably the most commonly observed failure is to the piers of bridges. Three modes of failure are possible, namely flexure,

shear and axial distress (Lee and Elnashai, 2001, 2002), as well as their combinations. The single-pier substructure of the Hanshin Expressway collapsed in the Hyogo-ken Nanbu earthquake, owing to the failure of gas welds on main reinforcing bars (Figure 6.2). Several cases of symmetric buckling of reinforcement and compressive failure of piers may, at least in part, be attributable to high vertical earthquake forces, both in Kobe and Northridge. The three piers supporting the Interstate 10 (Santa Monica freeway) Collector-Distributor 36 suffered varying degrees of shear failure, owing to the short shear span that resulted from on-site modification of the original design. Inadequate confinement in many piers caused premature failure of RC piers in recent earthquakes. Also, misinterpretation of the deformed profile of piers, with respect to the connection between pier and deck and pier and foundation, led to critical sections developing at points of reduced longitudinal reinforcement. Several cases of damage in Kobe are attributed to the latter effect.

Unzipping of corner welds in steel box piers in Kobe led to complete collapse of a number of piers, which were then squashed by the weight of the heavy deck (Figure 6.5). Also, a number of columns suffered extensive local buckling in the Kobe area. In several cases, this coincided with the termination of concrete infilling, which is used to protect the piers from vehicle impact.

Multicolumn substructures have also suffered damage in previous earthquakes. Examples of these are the Cypress Street Viaduct in the Loma Prieta earthquake; frames collapsed along a distance of more than a mile, owing to shear failure of the RC section at the base of the top level column (Figure 6.6), and the Embarcadero RC elevated road, which sustained heavy cracking, especially at the beam–column connections. Piers of the Struve Slough Bridge also failed in the Loma Prieta earthquake, and punched through the deck slab (Figure 6.6). There are many more cases of damage to piers in recent earthquakes, and more details are available in the literature (e.g. Broderick and Elnashai, 1995; EERI, 1995; Kawashima, 2000a, 2012; Kawashima *et al.*, 2011).

Superstructure

This includes pier–deck connections. Numerous cases of damage to or dislocation of bearings were observed in Kobe, with the ball or cylinder bearings sometimes found several metres away from their intended locations.

Damage to the superstructure is mainly not due to overstressing, since decks are normally designed to remain near elastic in earthquakes. Many cases of collapse were observed in San Fernando, Loma Prieta and Northridge, owing to unseating at seismic or expansion joints. This is due to defects in the bridge elsewhere, with the consequence of deck failure. Notable examples are the Oakland Bay Bridge steel truss (Figure 6.7), which lost a full span, owing to the inadequate size of support angles, and several high-level crossings in the San Fernando Valley. Also, a simply supported span of the Nishinomiya-ko bridge collapsed through unseating (Figure 6.7). Where

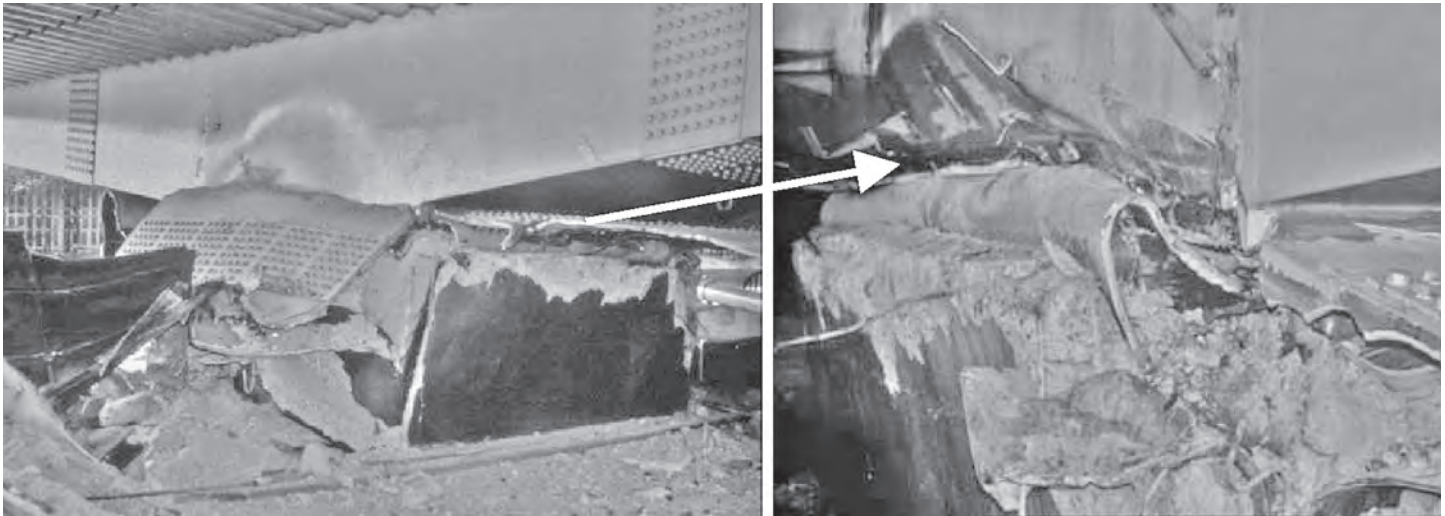


Figure 6.5 Total collapse of steel box girder pier, owing to weld failure (courtesy of Kawashima)

seismic restrainers were provided, damage to diaphragms occurred, owing to the very high local demands imposed at the restrainer anchorage point. Such effects are further aggravated in asymmetric or skew bridges, which are difficult to analyse at the design stage (Figure 6.8).

Impact damage between abutment and deck has also been observed, owing to inadequate displacement tolerance there (Figure 6.9). In a few cases, large inelastic deformation demands were imposed on decks, which were not designed for ductility. However, this is invariably due to the failure of an intermediate pier, thus imposing large flexural demand on a much-increased deck span.

Remarks

It is indeed noticeable that more recent earthquakes cause more damage to structures, notwithstanding the advancement in seismic design practice. This is due to the increased number of bridges of complex configurations, coupled with the heightened consequences of bridge damage in developed societies.

This quick overview of damage patterns sets the scene for considerations of layout and configuration that go a long way towards ensuring the seismic safety of bridge structures. If the general guidelines presented here are adhered to, it is to be expected that final design confirmation by analysis would



Figure 6.6 Damage to substructure resulting from the Loma Prieta (USA) earthquake of 17 October 1989: left, the Cypress Street Viaduct (I-880); right, failure of the piers of the Struve Slough Bridge (courtesy of USGS)



Figure 6.7 Damage by unseating at joints: left, the Oakland Bay Bridge failed span in the Loma Prieta (USA) earthquake; right, the Nishinomiya bridge collapse during the Hyogo-ken Nanbu (Japan) earthquake (courtesy of USGS)

be straightforward and little additional provisions would be required, especially in areas of low to medium seismic hazard.

Conceptual design issues

Layout

Many design problems that would lead to unsatisfactory seismic performance and damage could have been anticipated by an earthquake engineer at the conceptual design phase. It is therefore of importance to discuss the commonly observed layout defects and their implications on seismic behaviour, following from the field observations made in this chapter. Hereafter, a non-exhaustive review is given.

Plan layout

It is often necessary to construct skew or curved bridges. It should be noted, however, that curvature in bridges complicates the design and analysis and leads to difficulties in uniformly

distributing ductility demand (Burdette and Elnashai, 2008; Mwafy and Elnashai, 2007). It is often unconservative to analyse bridges with tight curvatures in two dimensions; hence, a full three-dimensional analysis is needed (Elnashai, 1996, 2004). Moreover, it is very difficult to quantify the degree of irregularity of a curved bridge, owing to complicated mode shapes of response and interaction between pier location and height and mode shapes of most significance. An example of a multispan curved bridge assessed using detailed modelling approaches and state-of-the-art assessment tools is shown in Figure 6.10 (Mwafy *et al.*, 2007).

Skew bridges also pose design and analysis problems additional to those faced for straight bridges (Kwon and Jeong, 2013). Vibrations along the axis of a skew bridge cause a torsional response that imposes large rotation demands on pier heads and out-of-plane movements on diaphragms. It was

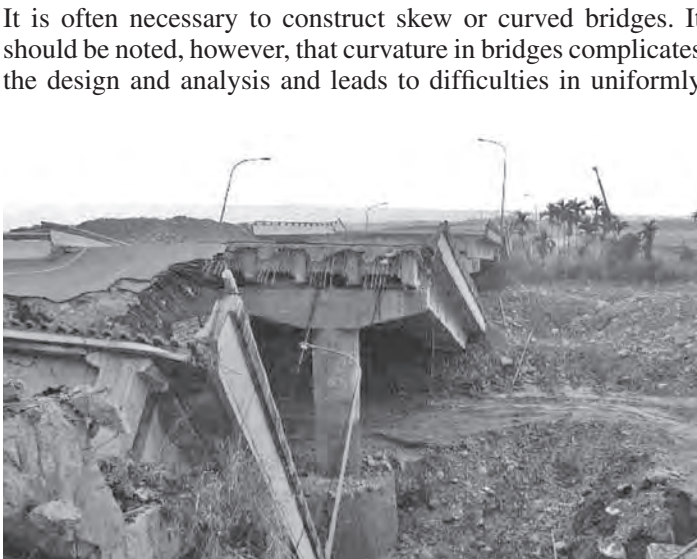


Figure 6.8 Unseating owing to bridge skew during the 1999 Chi-Chi earthquake in Taiwan (courtesy of National Information Service for Earthquake Engineering, University of California, Berkeley)



Figure 6.9 Failure of the abutment of the Highway 3 North-East out of Fengyuan during the 1999 Chi-Chi earthquake in Taiwan (courtesy of National Information Service for Earthquake Engineering, University of California, Berkeley)

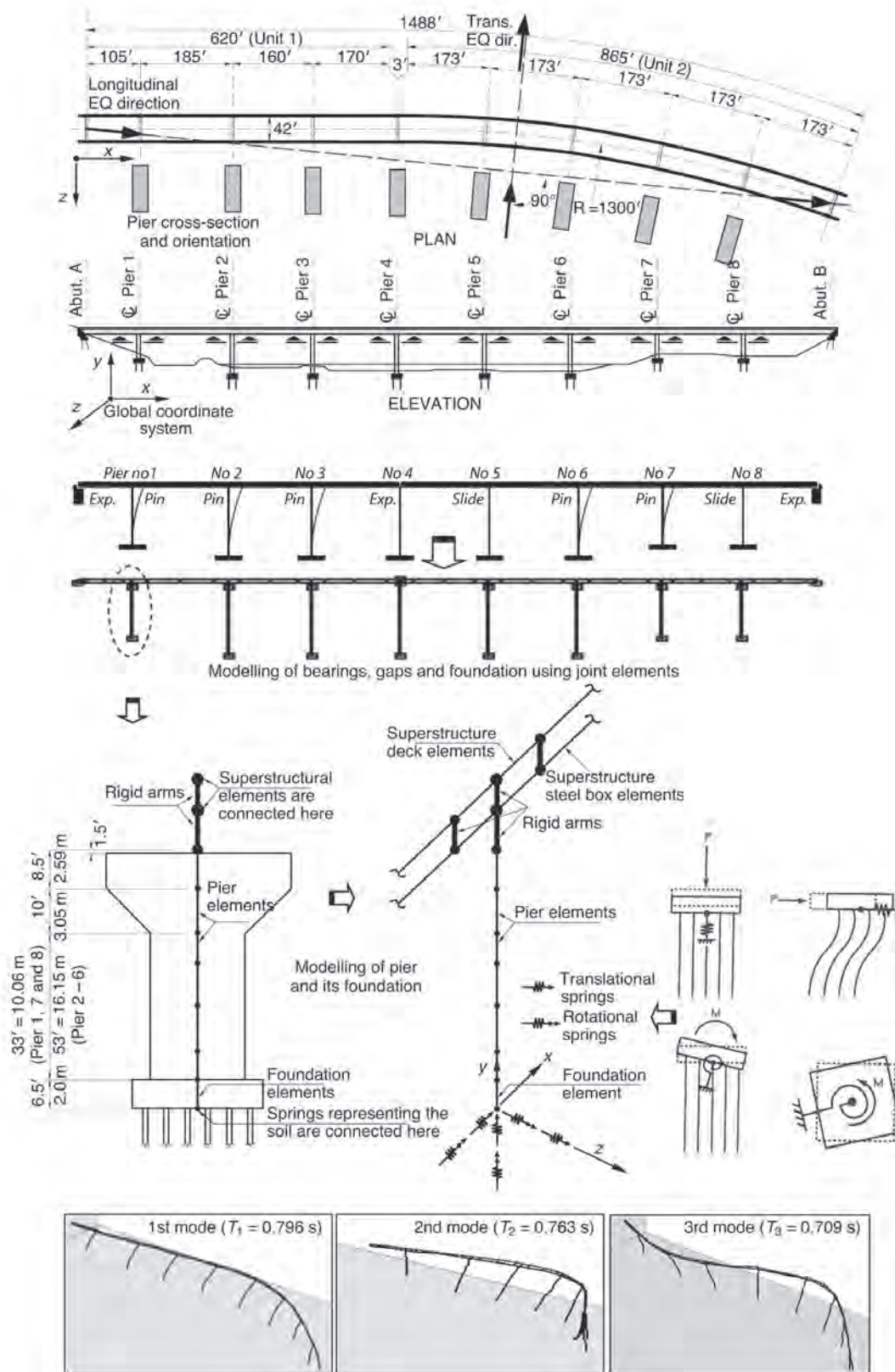


Figure 6.10 Fibre-based modelling of a nine-span steel-girder curved bridge, along with its dynamic characteristics

observed (Astaneh *et al.*, 1994) that diaphragms in steel decks would be subjected to lower demands if they were parallel to the axis of the pier, as opposed to normal to the axis of the connected girders.

In single-pier bridges, an eccentricity between the deck axis (horizontal) and pier axis (vertical) would also lead to torsional response and non-uniform distribution of deformation demand. It is therefore concluded (Calvi *et al.*, 1994) that the most effective layout is a straight bridge, with the axes of piers coinciding with the centre line of the deck in single-pier systems or with the support frame axis normal to the deck axis in multicolumn systems.

Layout in elevation

Contrary to common belief, the most regular bridge is not necessarily that with equal pier heights. This would be the case only in the special situation of full isolation of all piers and also abutments. The most regular response would be obtained from an elevation layout whereby the height of the pier is proportional to its distance from the abutment, where fixity against out-of-plane displacement is available, taking into account the mode shape most likely to influence the response. In general, bridges are long period structures that are likely to be affected by higher modes. These notes are pertinent only to RC bridges of moderate lengths, as mentioned before, where the fundamental mode would be predominant. Therefore, if the height of the various piers follows a half sine wave spanning between two abutments, where θ_y rotations are free and Z displacements are restrained, the most regular ductility demand will be imposed on the piers. In the case where the abutments allow lateral displacements, equal pier heights are the best solution. The relationship between pier stiffness and imposed displacement dictates the degree of regularity of the bridge. Conventionally, geometry considerations have been used to quantify bridge regularity measures, or indices. Expressions including mode shapes, hence the imposed demand, were proposed in the literature (e.g. Calvi and Pavese, 1997). Further developments of regularity indices that take into account instantaneous supply and demand, in the inelastic range, are currently underway. These are based on the concept that the pier causing the most irregular effect, owing to its stiffness, is likely to be damaged first. By sustaining damage, its stiffness drops; hence, the overall irregularity reduces.

Deck continuity

The majority of existing bridges have seismic or expansion joints. These were included in early design to reduce stresses from thermally induced deformations or to simplify the analysis of the deck–pier system. As mentioned already, such joints have caused a large number of failures in previous earthquakes. Therefore, it is recommended that decks are designed as continuous structures wherever possible to eliminate problems with unseating and pounding. The latter problems are compounded when incoherent motion is imposed (incoherence is defined

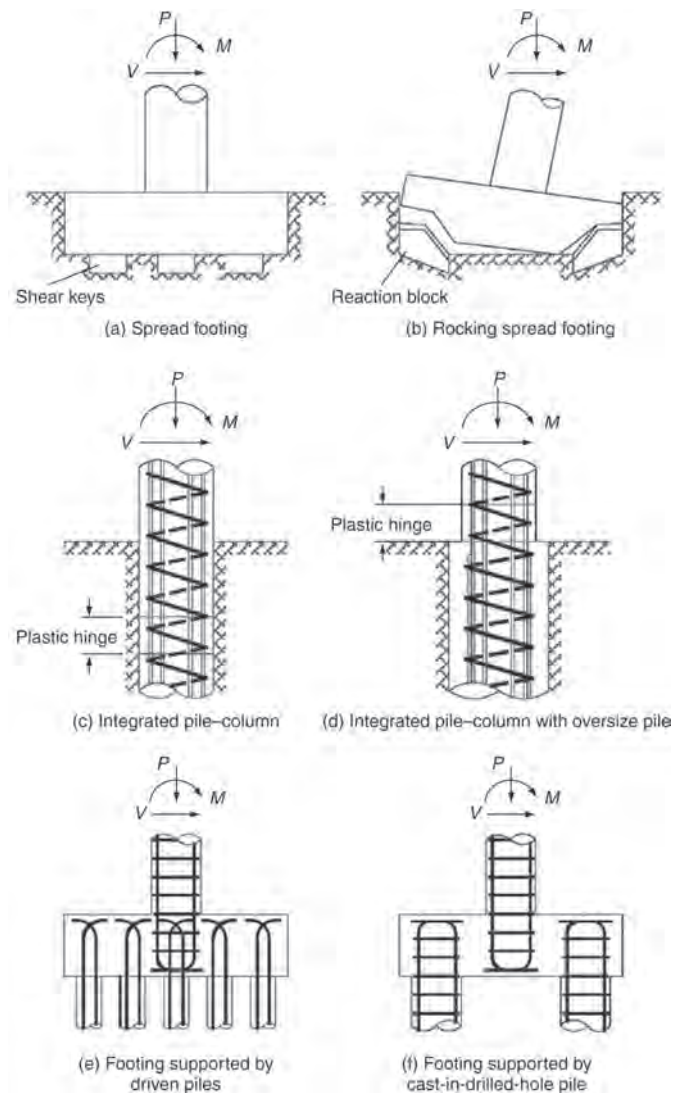


Figure 6.11 Options for foundation systems of RC bridges (Priestley *et al.*, 1996)

as significant differences in the input motion at different pier and abutment bases, owing to travelling waves and geometric incoherence of ground motion). The effect of incoherence on differential movement is negligible for most cases, compared with the effect of direct dynamic excitation (Burdette and Elnashai, 2008; Burdette *et al.*, 2008; Monti *et al.*, 1996). However, for complex or short-span bridges with stiff abutment conditions, the incoherent case may become dominant, and conventional estimates of design forces may be unconservative (Mwafy *et al.*, 2011; Tzanetos *et al.*, 2000).

Span length

In long-span structures, a large axial force is imposed on piers, owing to their tributary part of the deck. Under earthquake motion, horizontal and vertical excitations are imposed, with the distinct possibility that vertical modes of vibration of the

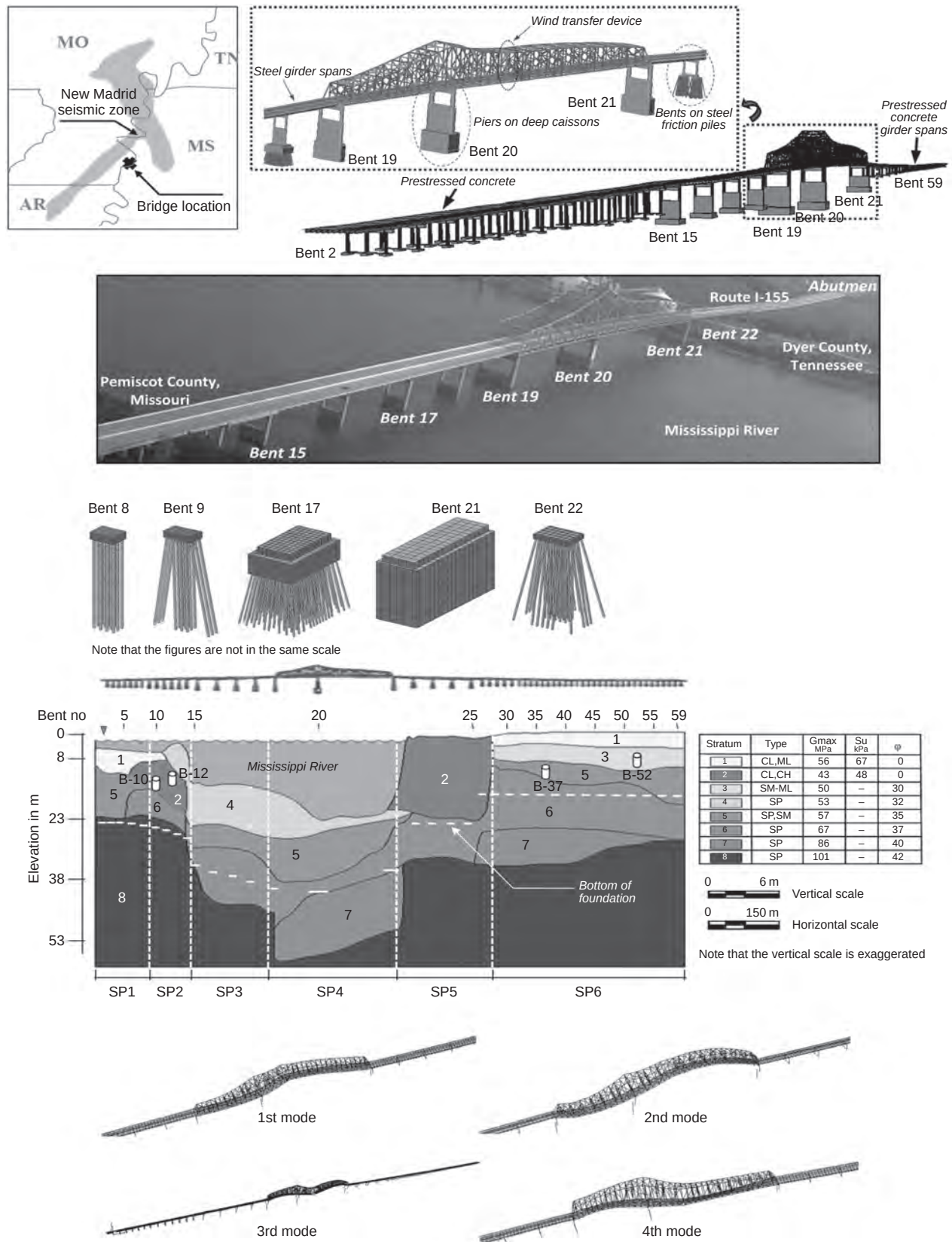


Figure 6.12 Structural and foundation systems and soil profiles along the length of the I-155 Bridge, along with its modes of vibration

deck (acting as a continuous beam) would be excited. This will impose very high axial forces (and variations in axial forces) on piers, thus reducing their flexural and shear capacities. It is therefore advisable to use short span lengths by increasing the number of piers. Otherwise, analysis under vertical earthquake ground motion should be undertaken (Elnashai and Papazoglou, 1997) and the effect of axial forces in piers accounted for.

Foundation materials

Bridges are especially susceptible to damage from large imposed displacements. Therefore, it is important to found them on rock or stiff soil wherever possible. In cases where the surface soil is soft or potentially liquefiable, use of piled foundations is recommended. When piles are driven through sloping ground, stability issues should be considered, since slumping may impose very large lateral forces on piles and piers.

Foundation systems

Foundations are the first points of contact between the ground transmitting seismic waves and the structure. Therefore, the foundation system has a most marked effect on the response characteristics of the bridge. Various alternatives are shown in Figure 6.11 (Priestley *et al.*, 1996).

The most commonly used foundation systems are RC footings. These should be designed to resist the gravity loading in addition to the seismic forces, appropriately scaled up by a factor to account for the force reduction used in the base shear calculation. Stability of the footing is provided by the contact with the foundation substrata, owing to gravity loading. Shear resistance is by friction on the horizontal plane under the footing and bearing on the vertical faces. Shear keys may also be provided. Plastic hinges will form in the column base first, and their location may be controlled by detailing of longitudinal and transverse reinforcement. The damage location is easily accessible, hence repair will not pose a problem. An alternative is shown in Figure 6.11(b), where the foundation is allowed to rock under earthquake motion, thus delimiting the column base force at the expense of increased top displacements (e.g. Mergos and Kawashima, 2005).

Figures 6.11(c) and 6.11(d) show integral pier–pile systems. These are rather economical and are often used in practice (e.g. the Jamuna bridge in Bangladesh, where concrete-filled steel circular tubes are used for piers and piles). In this case, the critical section is underground, hence it is difficult to inspect and repair. To alleviate this shortcoming, the pile section may be increased, to impose plastic hinging in the pier, as shown in Figure 6.11(d).

Footings supported on piles are becoming widespread in practice. The most economical system (Priestley *et al.*, 1996) is the cast-in-drilled-hole RC pile, shown in Figure 6.11(f). In cases of footings supported on piles, the objective is to keep

the piles elastic and concentrate the inelasticity in the piers. Consequently, and for cost considerations, the use of a small number of large-diameter piles is recommended.

Different foundation systems may be used in major bridges. Figure 6.12 shows the Caruthersville Bridge, which carries route I-155 over the Mississippi River between Pemiscot County, Missouri, and Dyer County, Tennessee. The foundation system of the 59-span 7100 ft bridge includes friction piles as well as deep caissons (Elnashai *et al.*, 2007; Mwafy *et al.*, 2010).

Foundation–pier connections

There are two options for this connection, namely pinned and fixed. In the pinned case, four bars are inserted at the centre of the pier and are anchored inside the footing, to resist nominal

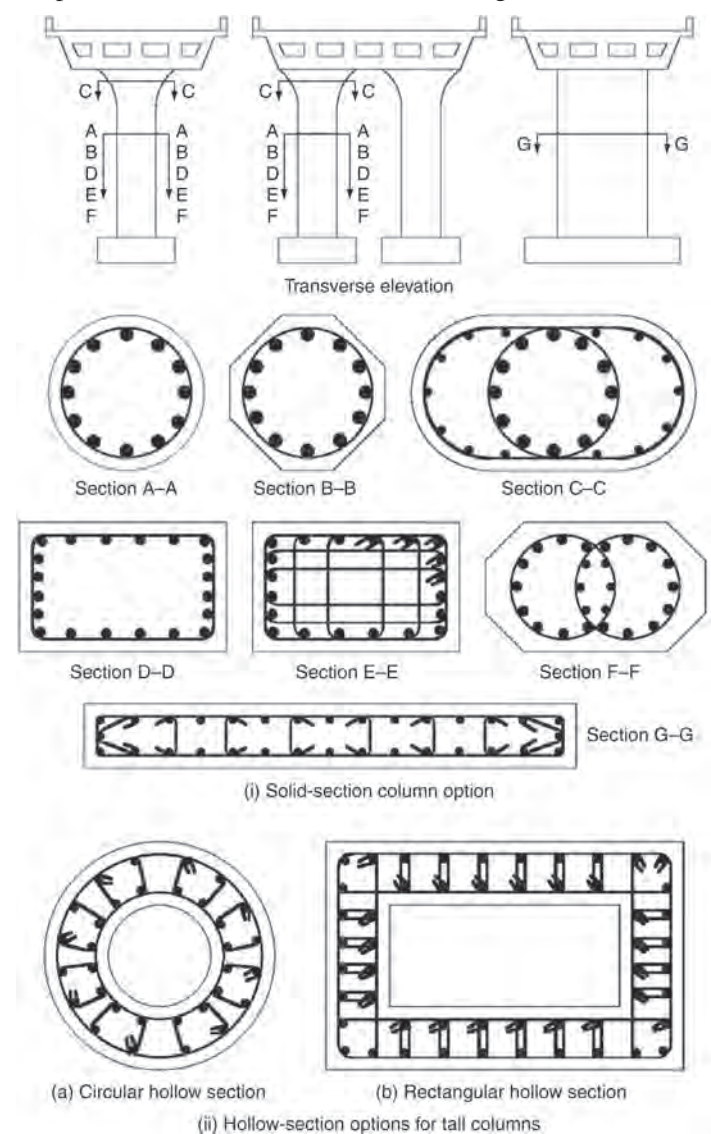


Figure 6.13 Options for pier sections and configuration (Priestley *et al.*, 1996)

sliding shear. The pier section is cast discontinuously from the footing and rubber or felt sheets are inserted between the two. In this case, large moments will develop at the pier head, acting as an inverted pendulum, but very low forces are exerted on the footing, which requires only nominal reinforcement. The second option is full moment connection between the footing and the pier. In this case, a full design is required, since possible uplift forces may exist. Axial, bending and shear actions on the footing are also much higher than in the pinned case.

Pier sections

A wide variety of RC and steel sections, as well as composite steel–concrete, is used for bridge piers, some of which are shown in Figure 6.13 (Priestley *et al.*, 1996). The circular section is most desirable, especially in cases where the longitudinal and transverse demands are similar. One of its main advantages is that it provides uniform confinement (in contrast to rectangular sections) and adequately restrains the longitudinal bars from buckling. To provide torsional resistance and stability at the pier–deck intersection, a flare is commonly used, as shown in Figure 6.13(i) (section C–C). The additional bars shown are preferable to bending of the main bars into the flare, for reasons of construction ease and also because of undesirable outward acting forces at the bending location.

The rectangular section D–D shown in Figure 6.13(i) has the disadvantage of inadequate protection of longitudinal bars against buckling. Also, the core is inadequately confined. These problems are mitigated in section E–E at the cost of a congested section and added workmanship. Section G–G, representing a shear wall type pier, is used where the longitudinal forces are carried mostly by the abutments, and only transverse stiffness and strength are required. It is noteworthy that bridge piers similar to those supporting the steel truss of the I-155 Bridge shown in Figure 6.12 are not recommended. The non-uniform distribution of stiffness and strength along the height of these piers results in large ductility demands at the base of the top columns (Elnashai *et al.*, 2007; Mwafy *et al.*, 2010).

Hollow sections (e.g. Figure 6.13(ii)) are used where the height of the piers is excessive; hence, the use of a solid section is not advisable (owing to the pier's large self-weight). It is most important, though, to check hollow sections against imploding, owing to the inward buckling of hoop inner layer of reinforcement.

The use of composite sections, constructed from concrete-filled steel tubes, is also popular, owing to their high ductility capacity and ease of construction. The method of load transfer from deck to pier has an effect on the characteristics of the response, owing to load sharing between the two materials. It is important that the steel tube does not separate

from the concrete core, otherwise local buckling may occur (e.g. Fam *et al.*, 2004; Hajjar, 2000; Tort and Hajjar, 2004). Sometimes, shear connectors are welded to the inside of the steel tube to ensure adequate interaction.

Lateral force-resisting system

The frame action resisting earthquake motion may be either single-column or multicolumn structures, as shown in Figure 6.14 (Priestley *et al.*, 1996). Single-column structures are easy to design and construct and are most suitable for situations where the demand along and across the bridge is similar. Also, since there is only one plastic hinge, response prediction is straightforward. There are several disadvantages though, such as low redundancy, high moment demand at the base, large seismic actions imposed on the foundation (owing to the necessity of fixing the base) and large deck displacements.

Multicolumn structures offer the option of fixed- or pinned-base solutions, hence drastic reductions in top section moments are availed of. Also, in general, displacements at the deck level are reduced, especially in the transverse direction. Moreover, one of the most important advantages of this configuration is the degree of redundancy, since redistribution of action can occur between the various columns.

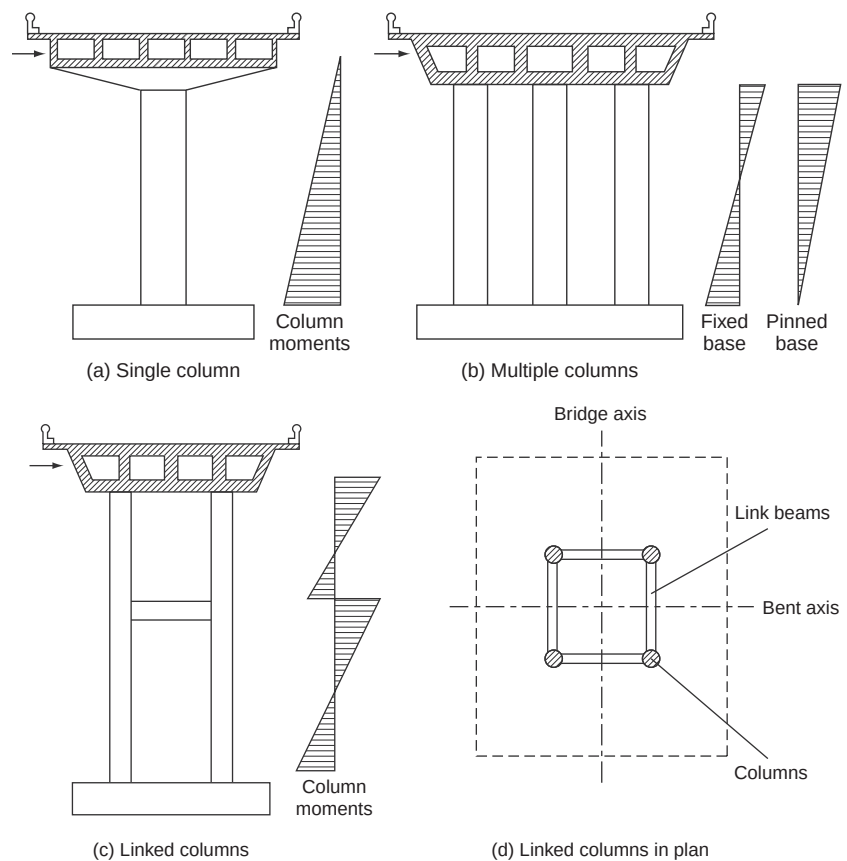


Figure 6.14 Options for lateral force-resisting systems (Priestley *et al.*, 1996)

Finally, load sharing between deck and columns, in monolithic structures, is better distributed than in the case of a single-column system. The main disadvantages of this type of structure are that its response is more complicated, hence less predictable in the inelastic range, and that there are more detailed connections than in the case of a single column. The demand imposed on columns will be non-uniform, owing to variations in axial forces and cap beam flexibility at various locations. As a consequence, the ductility demand in one column will be higher than that of the overall structure. In cases of very tall piers, linking of the columns may be used, as shown in Figures 6.14(c) and 6.14(d). In this case, care should be exercised in shear design where the link creates a column of height shorter than about $6\times$ its plan dimension.

Connection between deck and piers

This may be either monolithic or bearing-supported, as shown in Figure 6.15 (Priestley *et al.*, 1996). Monolithic construction is normally used for slender columns and small bridges. The energy absorption capacity of this configuration is, in general, larger than in bearing-supported systems (Elnashai and McClure, 1996), owing to the double curvature of the former, leading to potentially two plastic hinging zones, instead of one in the case of bearings.

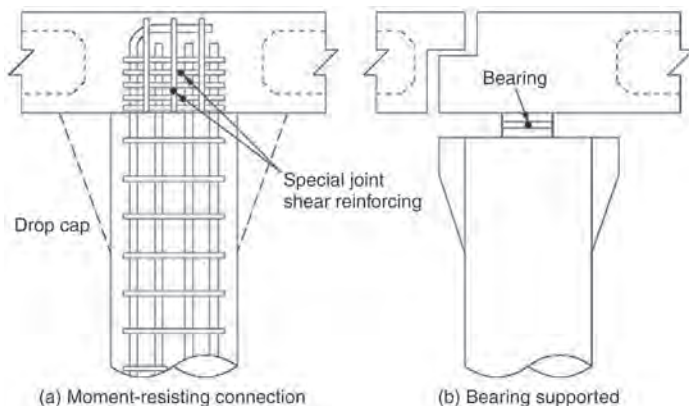


Figure 6.15 Options for pier-deck connection (Priestley *et al.*, 1996)

For multicolumn configurations, the use of monolithic systems enables the stiffnesses in the longitudinal and transverse directions (when using circular and square columns) to be equal, thus eliminating the potential for a preferential response direction. Two more advantages of monolithic construction are that it allows consideration of using a fixed or pinned column-foundation condition (see earlier discussion of foundation-pier connections) and that it leads to higher redundancy of the lateral response system.

A major disadvantage of monolithic deck-pier systems is that large moments are transmitted to the deck that add to the moments from gravity forces, thus creating critical conditions

there. This is especially true for single-column systems with wide decks, since the effective deck section resisting these high moments is relatively small.

Another problem with this type of connection is the difficulties of ensuring that capacity design is respected, in terms of connection overstrength. Large-diameter bars from the pier should be adequately anchored in the relatively shallow cap beam; thus, reinforcement congestion may become a problem.

Another problem is the imbalance between the longitudinal and transverse directions for single-pier systems. In the transverse direction, the torsional stiffness of the deck only is acting against the pier behaving as a cantilever; thus, the behaviour is close to such a condition. Longitudinally, the deck is monolithic with all piers, hence the restraint it applies on pier heads is much larger than in the transverse direction. Therefore, the pier is normally stiffer longitudinally than transversely. If a balanced design is sought, a rectangular section will be required.

Thermal expansion imposes large displacement demand on monolithic systems longitudinally and therefore requires short spans between expansion joints. This is an unfavourable feature of monolithic pier-deck construction.

The second option is the provision of bearing support between pier and deck, allowing one or more translational or rotational degrees of freedom. The most significant advantage of bearing supports is that the deck is not subjected to seismic forces; hence, configurations not amenable to high moment resistance may be utilised. Also, the period of the bridge is elongated, as compared with monolithic bridges. This may be advantageous when the bridge is founded on rock or stiff soil, but is not suitable for soft sites. Another important advantage of bearing supports is that, by suitable adjustment of bearing characteristics, stiff bearings may be placed on top of flexible piers, and vice versa. Hence, a more uniform distribution of stiffness and strength than for the case of monolithic structures would be easy to achieve.

Bearing supports have serious disadvantages, such as the effect of period elongation of the structure in areas of soft site conditions subjected to large distant earthquakes. Also, the bridge is subjected in general to larger displacements than its monolithic counterpart. For multicolumn structures, the piers are placed in double curvature transversely while the longitudinal response is that of a cantilever. Since the option of a pinned pier foundation condition is no longer available, footings are subjected to large seismic forces and are susceptible to uplift. Also, owing to the existence of one potential plastic hinge in a single-column structure, the ductility demand on the single pier is much larger than that of the overall structure (a global displacement ductility of 5 may correspond to a pier ductility demand of 14; Priestley *et al.* (1996).

Finally, bearing systems may undergo large inelastic displacements that are not restored at the end of the earthquake, hence impairing the use of the bridge. This drawback can be offset by using self-restoring bearing systems.

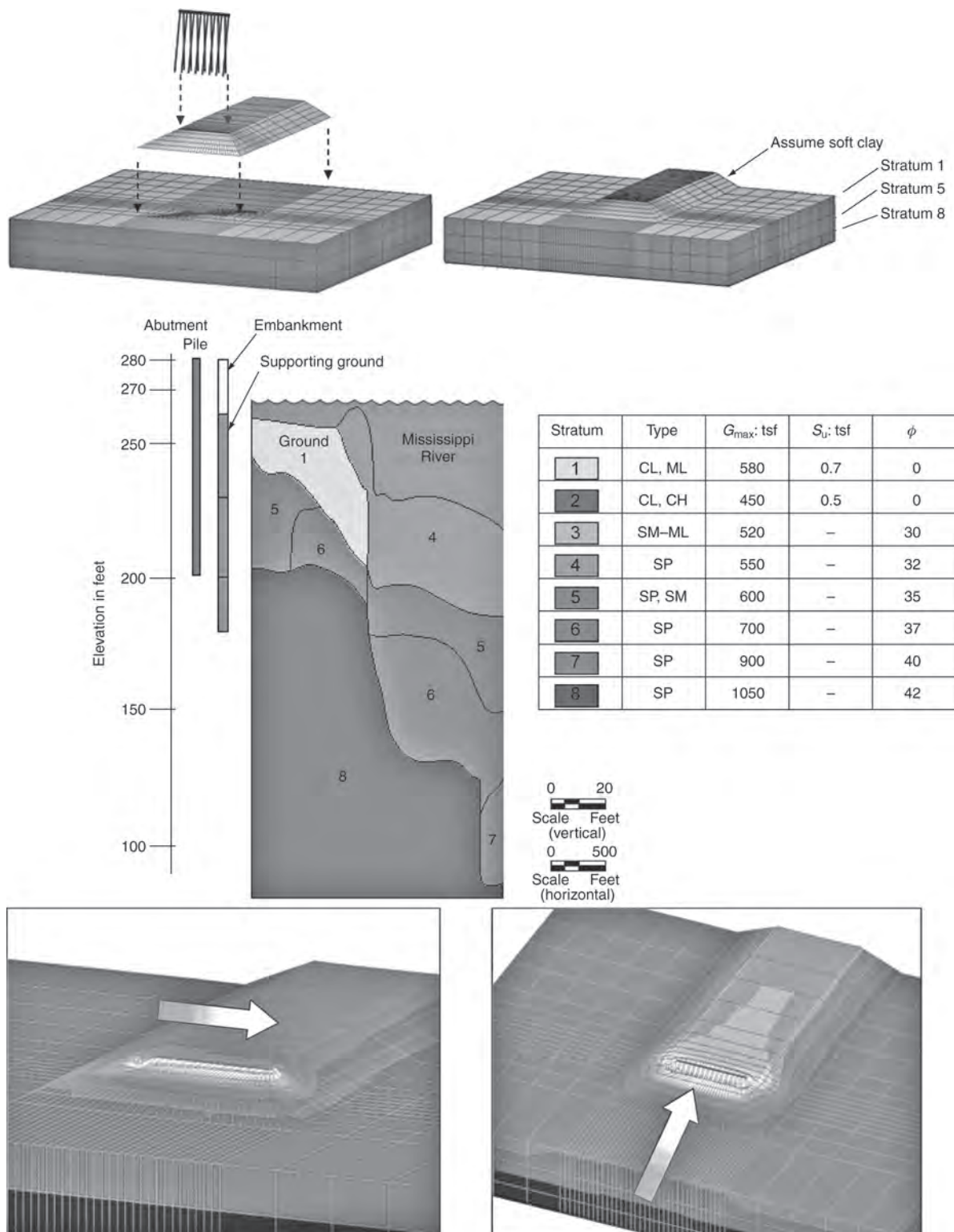


Figure 6.16 Advanced finite-element modelling and sample analysis results of bridge abutment considering soil–structure interaction

Connection between deck and abutment

It is often the case that abutment design is not given due consideration, leading to poor seismic response. This is partly due to the misconception that abutments are less sensitive to earthquake destruction than piers. Also, accurate representation of abutment seismic response is difficult, owing to soil–structure interaction representation. Figure 6.16 shows advanced finite-element modelling and sample analysis results of the I-155 Bridge abutment (see also Figure 6.12), considering soil–structure interaction. However, site studies indicate that abutment failure is frequently observed to warrant serious consideration in design. Moreover, studies on bridge models with various abutment conditions (e.g. Dodd *et al.*, 1996) concluded that boundary conditions at the abutment have a significant influence on dynamic response characteristics of RC bridges.

A further point is the influence of abutment modelling and restraints under asynchronous motion that was studied by Tzanetos *et al.* (2000), as mentioned in earlier sections of this chapter. A number of analyses were conducted for pinned and fixed abutment–deck connections, which led to the conclusion that the latter is more critical in the case of soft foundation material (low shear wave velocities). This was particularly noticeable and critical for the fixed abutment case, where the second mode contributed significantly for the asynchronous case, but not in the synchronous analysis. It is therefore important to ponder the effects of abutment configuration, stiffness in various planes and strength on the response of the bridge as a whole. The connection may be monolithic, bearing-supported or isolated, as described next.

Monolithic connections, such as those depicted in Figures 6.17(a) and 6.17(b) (Priestley *et al.*, 1996), are more commonly

used for small bridges. They are designed to resist the total seismic force, while intermediate piers are designed for gravity loads only but detailed for ductility as a precaution. In this case, the system shown in Figure 6.17(b) is more reliable than that of Figure 6.17(a), since the latter relies at least in part on soil bearing resistance behind the abutment wall. This system will also have unequal stiffness in the push–pull longitudinal direction. The rigid (monolithic) abutment is designed for the peak ground acceleration without amplification, since it will move as a rigid body with the ground.

For large bridges, it is extremely difficult to resist the total seismic force at one location. Therefore, a bearing support may be employed, with all other vertical members sharing in the seismic resistance. Bearing supports have many configurations, two of which are shown in Figures 6.17(c) and 6.17(d). The stiffness in the negative longitudinal direction is provided by the wing wall, and the soil bearing stiffness (and strength) follows the closure of the gap, which is designed for temperature changes.

In the positive direction, stiffness depends on the bearing characteristics. Ball and cylinder bearings resting on horizontal planes or dish receptacles (for self-righting) are used in practice. The alternative shown in Figure 6.17(d) utilises a knock-off detail, in which damage to the transverse wall is allowed in large earthquakes, hence a large tolerance for displacements is afforded. For both configurations shown, the bearings may be substituted by isolators, which normally double up as dissipaters, since the effect of period elongation of bridges on the seismic forces is less significant than the effect of damping by energy dissipation.

An issue of great significance is that of the transverse boundary conditions imposed on the deck by the abutment and the consequence of resulting forces imposed on the abutment. In general, it is more difficult to provide stiffness and strength

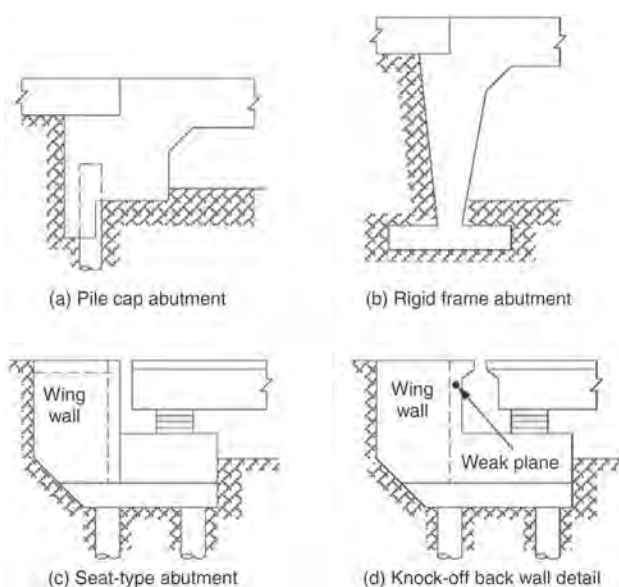


Figure 6.17 Options for abutment–deck connection (Priestley *et al.*, 1996)

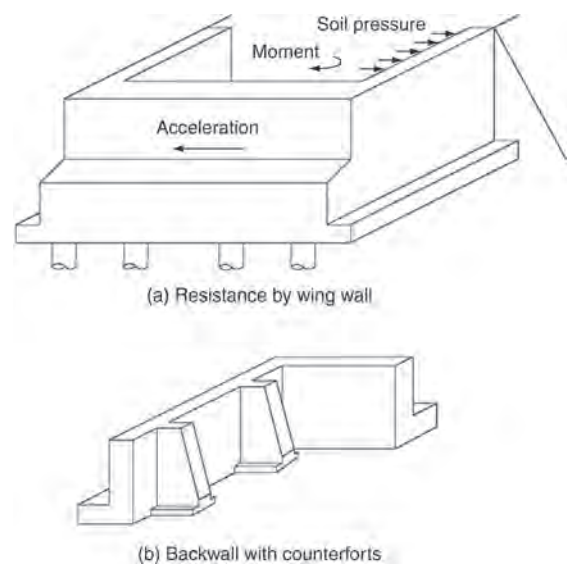


Figure 6.18 Mechanisms of resisting forces at the abutment (Priestley *et al.*, 1996).

laterally than for the longitudinal direction, as may be intimated from Figure 6.18(a) (Priestley *et al.*, 1996). Hence, buttresses, as shown in Figure 6.18(b), may be utilised to increase resistance in both directions, which is also contributed to by the piles. With regard to the boundary conditions imposed on the deck, consider Figure 18(a). The moment constraint will be a function of the bearings used, and their number and location. For a large number of bearings at some spacing, resting on a high-friction surface will provide a high degree of restraint, and vice versa. The force restraint depends on the existence and details of shear keys. Care should be exercised not to allow a very large difference in stiffness to develop between abutment and piers, since this will lead to a high concentration of force and deformation demand imposed on the former, which in turn will lead to severe distress.

Brief review of seismic design codes

Historical note on bridge codes in Europe, the USA and Japan

The history of European practice in seismic design is rather recent, and cases of bridge damage in European earthquakes are very few indeed. For Europe, the interest in seismic design of bridges arises from two main considerations: the potential for disastrous effects and the export market. With regard to the former, several thousand bridges in Italy are potentially subjected to considerable earthquake risk, while major projects for bridge construction have been completed in Greece, among several other European nations with a large bridge population and non-negligible exposure.

Whereas national codes in Europe included seismic provisions for bridges, these were rather minimal considerations and intended to support European consultants working abroad. It was through the development of Eurocode 8, Design of Structures for Earthquake Resistance (Eurocode 8 Part 1 (BSI, 2005a); Eurocode 8 Part 2 (BSI, 2005b)), that concerted efforts were dedicated to the development of seismic design guidelines for bridges in Europe. The first drafts, circulating in the late 1980s, were quite brief, but unevenly so, and reflected the interest of the individuals involved in the drafting panels, rather than the practice of design. With funding from national and European sources being directed towards research projects in the earthquake engineering field, extensive development work has been carried out in several academic and research institutions in Europe, including universities and research laboratories in Italy, Greece, Portugal, France, Belgium, Germany and the UK. Links with US, New Zealand and Japanese workers in the field also encouraged the development of a European approach to seismic bridge design. Subsequent versions of the Eurocode 8 draft, with Part 2 dedicated to bridges, seemed more robust and comprehensive, reflecting a somewhat different approach from that of other codes. It is perhaps the relative freedom of

drafting panels working on the draft, unhindered by practice (for better or for worse) that led to the current format of Eurocode 8 (Fardis *et al.*, 2005).

The current version of Eurocode 8 Part 2 (BSI, 2005b) reflects a recent view of the European earthquake engineering community. The approach is that of utilising ductility to reduce design forces, to detail potential plastic hinge zones for high levels of curvature ductility and to protect other bridge components by applying capacity design factors. It deals with both fixed and isolated bridges in steel and reinforced concrete. There are rather advanced topics in the code, such as models for spatial variability and conditions of acceptability of suites of artificial records for analysis. The code also attempts to give comprehensive guidance for modelling and analysis of bridges, statically and dynamically.

The situation is fundamentally different in the USA. The Association of American State Highways and Transportation Officials (AASHTO) has been issuing seismic provisions for many years. The current version is based on the Load and Resistance Factor Design (LRFD) philosophy (AASHTO, 2017). However, the status of bridges in California and the repeated relatively poor performance of bridges in Californian earthquakes led the California Department of Transportation (Caltrans) to develop its own design guidance. The Caltrans (2019a) Seismic Design Criteria have been affected by the patterns and type of damage to bridges in earthquakes in the state. In the San Fernando earthquake of 1971, the interchange between the Golden State and Antelope Valley Freeways suffered total collapse.

There were many other cases of heavy damage, partial and total collapse of bridges and elevated roads in the prosperous and heavily populated San Fernando Valley. This prompted major investment in research, particularly on the design and strengthening of RC bridges, and an extensive strengthening programme was initiated. The weakness of bridges in California was confirmed by the heavy damage sustained in the Loma Prieta earthquake of 1989. The catastrophic collapse of the Interstate 880 (Cypress Street Viaduct) caused heavy casualties as a result of the collapse of a mile-long stretch of the double-deck RC structures (Elnashai *et al.*, 1989). Heavy damage was also inflicted on two major structures at the heart of the San Francisco network. The Embarcadero Freeway suffered shear distress to many columns and girder-column connections alongside the terminal separation. These were replaced at a cost of \$120 million (at 1990 prices). Also, the Struve Slough Bridge outside Watsonville collapsed completely, with the piers punching through the deck. The state-funded research initiative for bridges was approved on 6 November 1989, less than 1 month after the earthquake.

Following the Governor of California's report on the earthquake, *Competing Against Time*, published in May 1990 (Housner and Thiel, 1990), state and federal funds were made available to a number of universities and research laboratories, in association with practising engineers. Interim guidance for repair and strengthening was issued, as well as new design procedures, with further emphasis on ductility and capacity design

principles. The next major disaster hit southern California in 1994. The Northridge earthquake of 17 January inflicted very heavy damage to bridges, intersections, overcrossings and even exit and entrance ramps (Basoz *et al.*, 1999; Broderick and Elnashai, 1995; Broderick *et al.*, 1994), showing that the battle against bridge damage in earthquakes was far from over. This sequence of events led to the continuing of state support for development of the Caltrans design and strengthening guidelines. It was therefore deemed appropriate that the subsequent comparison of bridge codes should include the Caltrans (2019a) *Seismic Design Criteria* as a US representative, as opposed to AASHTO (2017).

Following the Great Kanto earthquake of 1923, the first regulations for bridge design in Japan were introduced in 1926, as reported by Kawashima and Unjoh (1997) and Kawashima (2000a). These were updated in 1939, 1956 and 1964. These were rather simple guidelines, mainly recommending the application of a lateral force of 20% of the weight of the structure, without reference to earthquake, structure or site characteristics. Moreover, the regulations pertained only to steel structures, since very few bridges were constructed in RC.

The first comprehensive seismic design code for bridges was issued in 1971, introducing design forces dependent on importance, hazard zone and site condition, in the 'seismic coefficient method'. In a variant, termed 'modified seismic coefficient method', the structural response characteristics were also taken into account. And, owing to the experience of the Niigata earthquake of 1964, assessment of liquefaction potential became part of the 1971 code.

Parts of the 1971 Guide Specification that are common to the material sections (steel, concrete and composite) were further revised in 1980 to become the *Design Specification for Highway Bridges (1980 Specification)*. This mainly entailed revision to the liquefaction potential assessment method and guidance on foundation design in liquefiable soils. A further, but more substantial, revision was introduced in 1990 (JRA, 1990). The main revisions were (Kawashima, 2000a; Kawashima and Unjoh, 1997):

- unification of the seismic coefficient and modified seismic coefficient methods
- checks on the dynamic strength and ductility of piers
- introduction of a frame analysis approach to evaluate force distribution in multispan structures
- provision of response spectra for dynamic analysis.

Damage to bridge structures in the Hyogo-ken Nanbu earthquake of 17 January 1995 was significantly more than expected (e.g. Elnashai *et al.*, 1995; Kawashima and Unjoh, 1997; Seible *et al.*, 1995). Quick revisions of the design guidance, especially for repair, were therefore published in February 1995. These were applied to new structures as well, while the process of revision of the 1990 code was underway. The latter was issued in 1996, and included explicit reference to ductility-based design while retaining the old seismic coefficient method. The

performance-based design concept was then introduced in 2002 (JRA, 2002; Kawashima and Unjoh, 2004).

In the authors' opinion, Japanese design practice for bridges has been traditionally based on strength concepts, thus ignoring the deformational capacity of structures. This usually leads to stiff and strong structures that exhibit very low levels of ductility. This could be a consequence of the cost of ductile detailing, as opposed to that of additional concrete and reinforcement. The damage inflicted by the Hyogo-ken Nanbu earthquake, in particular, attests to this view. Contacts with Japanese experts working on code developments indicate that this approach has changed although, in practice, engineers were unaccustomed to applying large force reduction factors in design.

Comparison of provisions

A number of studies comparing seismic codes from the USA and other countries are available in the literature (e.g. Kawashima, 2000b; Rojahn *et al.*, 1997; Yen *et al.*, 2003). A comparative study between the seismic design specifications for highway bridges of the USA and Japan was made by Yen *et al.* (2003). In the comprehensive study of Rojahn *et al.* (1997), the USA codes, namely AASHTO, ATC and Caltrans, were compared. Also included in the latter study were the codes of New Zealand and Japan, as well as Eurocode 8. Kawashima (2000b) also presented a brief comparison of design philosophy, design force and ductility requirements for the European, New Zealand, Japanese, AASHTO and Caltrans (1996) ATC-32 codes. A number of studies also presented the development in certain seismic codes (e.g. Kawashima and Unjoh, 2004; Kuwabara *et al.*, 2013; Unjoh *et al.*, 2002). A comparison is given in Table 6.2 for a subset of the codes compared in the aforementioned references. The latest Caltrans provisions (Caltrans, 2019a) are selected to represent the US codes in preference to AASHTO, for the reasons outlined in the previous section. For brevity, only the provisions of the Japanese code and Eurocode 8 are given (BSI, 2005b; JRA, 2002). Table 6.2 is based on the aforementioned Caltrans, European and Japanese provisions. Moreover, several clauses were updated and altered to reflect new information available to the authors or different interpretation of code clauses.

It should be emphasised that all three codes discussed here are moving targets, since they are continuously under review and development. This is especially the case with the Japanese code, owing to the time lapse between issuing a new code version and its availability to an English-speaking audience.

Closure

This chapter gave a snapshot of bridge failure modes commonly observed in previous earthquakes, given to highlight that there are repetitive patterns caused by inherent weaknesses. This was followed by a discussion of features of layout that are favourable to controlled and predictable seismic response of bridges. This favours relatively regular structures in plan and elevation.

Provision	Caltrans	Eurocode 8	Japan
1. General			
(a) Performance criteria	Three main bridge categories are assigned different post-earthquake damage states and service levels, corresponding to various seismic hazard evaluation levels.	No collapse (ultimate limit state) under the design seismic event. Minor damage (serviceability limit state) under frequent earthquakes.	Two classes of bridges are assigned three levels of seismic performance, ranging from 'no structural damage' to 'prevent critical damage', under earthquakes with different probabilities of occurrence.
(b) Design philosophy	Sufficient strength and ductility to achieve the specified performance criteria. Ensure failure by inelastic flexural yielding while minimising the chance of non-ductile failure modes.	Sufficient ductility and strength to be provided according to the intended seismic behaviour. Brittle types of failure to be avoided in all structures.	Bridges are designed by both the seismic coefficient method and the ductility design method.
2. Seismic loading			
(a) Design spectrum	Design response spectrum based on USGS seismic hazard maps. Adjustment factors for near-fault effects and basin amplification are also applied.	Normalised elastic response spectra with variable corner periods. Site-dependent power spectrum representation is permitted.	Standard acceleration response spectra.
(b) Return period	975 year return period for safety evaluation earthquake (SEE); 225 year return period for functional evaluation earthquake (FEE).	The recommended value is 475 years.	Unknown. Level 1 and Level 2 ground motions with two types in Level 2, representing large-scale subduction-type and near-field earthquakes.
(c) Geographic variation	Based on Caltrans-approved USGS seismic hazard maps.	Set by national authorities. No agreed-on zonation maps for Europe.	Modification factor for zones.
(d) Importance considerations	Bridges are categorised as ordinary, recovery or important.	Recommended three categories (greater than average, average and less than average). Set by national authorities.	Bridges are categorised into two groups: standard bridges (Type A) and important bridges (Type B).
(e) Site effects	Soils are classified as either Class S1 (competent soil) or Class S2 (non-competent soil), depending on their properties.	Five basic types, and two additional profiles for soft soils. Spatial variability must be considered for bridges with total length >600m and where abrupt change in soil.	Types I, II and III, corresponding to stiff, medium and soft soil conditions, respectively.
(f) Damping	5% of critical.	Spectra normalised to 5%. Damping correction factor may be used.	5% of critical.
3. Analysis			
(a) Selection guidelines	Based on maximum bridge length, skew angle and bearing difference between supports.	Based on bridge regularity and anticipated performance.	Based on structure complexity.
(b) Equivalent static	Equivalent static horizontal force applied to individual frames. Suitable for short length regular bridges.	Fundamental mode method is applicable for simple and regular bridges.	Static analysis with seismic coefficient method or ductility design method. Applicable to bridges with simple behaviour.
(c) Elastic dynamic	Multimodal spectral analysis. Design spectrum is applied simultaneously in two orthogonal directions.	Multimodal response spectrum analysis. 3D lumped mass space frame.	Suitable analysis method, such as response spectrum analysis.

Table 6.2 Comparison of seismic codes

Provision	Caltrans	Eurocode 8	Japan
(d) Inelastic static	Used to determine the displacement capacity of a frame or bent, provided the seismic response is dominated by a single translational mode of vibration.	May be applied to the entire bridge structure or to individual components.	Unknown.
(e) Inelastic dynamic	Applicable to very long bridges with significant skew angle or bearing difference between supports.	Used only in combination with a standard response spectrum analysis to provide insight into the post-elastic response.	Applicable to complex bridges and for the verification of Seismic Performance Levels 2 and 3 for Level 2 earthquake ground motion.
(f) Directional combinations	Seismic loads are applied along the principal directions of the bridge. The resulting demands need not be combined.	Case 1 = $L + 0.3T + 0.3V$ Case 2 = $0.3L + T + 0.3V$ Case 3 = $0.3L + 0.3T + V$	Longitudinal and transverse loads examined separately.
(g) Vertical ground motion	Considered for multispan girder bridges located at sites with peak ground accelerations $\geq 0.6g$.	May be omitted in zones of low to moderate seismicity. Need to be investigated in exceptional cases in high seismicity zones.	Unknown.
4. Seismic effects			
(a) Design forces	Consider ductility and design seismic hazards.	Consider ductility class and ground motion.	Elastic forces for Seismic Performance Level 1. Forces are adjusted for ductility when ductility design method is required for the verification of Seismic Performance Levels 2 or 3.
(i) Ductile components	Seismic critical members are designed for energy dissipation or damping when the design seismic hazards occur.	Elastic force reduction factor (behaviour factor q) depends on structure type and component.	Force reduction factor is determined according to the mechanical properties of structural members, including their plastic characters.
(ii) Non-ductile components	Capacity protected members are designed to remain essentially elastic (capacity design).	Capacity design with protection factors.	Unknown.
(b) Displacements	Based on effective moment of inertia at first yield for seismic critical members.	Use secant stiffness at yield for members with plastic hinges	Definitions for yield and ultimate displacements for Type I and Type II earthquake ground motions.
(c) Minimum support length	Should accommodate relative longitudinal earthquake displacement.	States that unseating should be avoided.	Provisions based on relative displacement due to earthquake ground motion.
5. Concrete design			
(a) Columns (flexure)	Ductile design and detailing as seismic critical members.	Capacity design. Ultimate strength design.	Lateral strength and ductility capacity are verified for Seismic Performance Levels 2 and 3.
(b) Column (shear)	Shear capacity based on contribution from concrete and transverse reinforcement. Concrete contribution at plastic hinges is reduced to zero for members in tension.	Verification of shear resistance is carried out in accordance with Eurocode 8 Part 1, with additional safety factors.	Shear capacity based on contribution from concrete and transverse reinforcement.
(c) Footings	Capacity protected members that are designed to remain essentially elastic when the design seismic hazards occur.	Mainly covered in Eurocode 8 Part 5.	Foundation is verified to ensure that yielding will not generate. Controlled plastic behaviour may be allowed in certain cases (e.g. large liquefaction effects) if the function of a bridge can easily be recovered.

Table 6.2 Continued

Provision	Caltrans	Eurocode 8	Japan
(d) Superstructure and pier joints	Designed to transmit the maximum forces produced when the column has reached its overstrength capacity.	Capacity design in ductile structures.	Unknown.
(e) Caps	Capacity protected members.	Capacity design in ductile structures.	Unknown.
(f) Superstructure	Capacity protected members.	Capacity design in ductile structures.	Response of superstructural members should not exceed their allowable values, which depend on their types and properties.
(g) Shear keys	Designed as sacrificial elements that are prone to damage during design level earthquakes.	Capacity design in ductile structures.	Provisions for excessive displacement stoppers.
(h) Anchorage of column reinforcing steel	Provisions for anchorage length.	Mainly covered in Eurocode 8 Part 1.	Unknown.
(i) Splices of column reinforcing steel	Not allowed in plastic hinge zones.	Not permitted in plastic hinge regions.	Lapping of axial reinforcement is not be placed within the plastic zone.
6. <i>Steel design</i>	Based on Caltrans <i>Seismic Design Specifications for Steel Bridges</i> (Caltrans, 2016).	Covered for the substructure and superstructure.	Provisions for steel columns and steel superstructure.
7. <i>Foundation design</i>			
(a) Spread footings	Designed to resist seismic loading in accordance with the seismic performance criteria.	Mainly covered in Eurocode 8 Part 5.	Provisions for pier and abutment foundation.
(b) Pile footings	Provisions for pile foundations in Class S1 (competent soil) and Class S2 (non-competent soil).	Mainly covered in Eurocode 8 Part 5.	Provisions for pier and abutment foundation.
(c) Liquefaction	Considered associated effects of design seismic hazards (DSHs). Provisions for the analysis and design of bridges subject to liquefaction.	Mainly covered in Eurocode 8 Part 5.	Detailed provisions for foundations at sites prone to soil liquefaction.
8. <i>Miscellaneous design</i>			
(a) Restrainers	Designed to remain elastic and restrict movement; should not be used to reduce the required support length at hinge seats in new bridges.	Required if minimum seat length without restrainers not met in ductile structures. Required for limited ductility and isolated bridges.	Design provisions for unseating prevention system of superstructure.
(b) Base isolation	Considered non-standard bridge feature subject to the project-specific seismic design criteria and Caltrans (2019b) <i>Bridge Memo to Designers</i> .	Specific chapters devoted to seismic isolation and elastomeric bearings.	Provisions for seismically isolated bridges.
(c) Active or passive control	Non-standard bridge feature subject to project-specific seismic design criteria.	Under development.	Unknown.

Table 6.2 Continued

Finally, many of the options available for foundations through to the superstructure, and connections between various components, were presented and their likely effects on the response discussed. Simple general guidelines adhered to during the conceptual design stage would considerably facilitate final design confirmation, and would increase seismic safety, especially in areas of low to moderate seismic hazard. Other alternatives to the bridge configurations presented here, such as the use of isolation and dissipation devices and active control mechanisms, are also feasible and have been gaining ground recently.

A brief review of seismic design codes and general practice in Europe, the USA and Japan indicates that the situation is far from uniform. In general, US and European practice utilise ductile response and capacity design principles more than does Japanese practice. A number of issues require further development and this is, by and large, underway. In all three cases considered, seismic design codes are undergoing radical revisions; hence, the statements made here should be considered as time-qualified.

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Chapter 7

Substructures

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'Substructure' is a collective term comprising all elements below the deck, which transfer loads to the ground below. No two bridge sites share the same ground conditions, so by definition substructure design must be bespoke for each structure. In the substructure, structural and geotechnical engineering come together, with the nature of the ground influencing not just the selection of piles or footings, but the stiffness and behaviour of the supported elements and, ultimately, the bridge in its entirety. One cannot divorce the supports from the deck or vice versa; a successful bridge design must consider sub- and superstructure holistically. In a single chapter, it is not possible to do justice to all aspects of substructure design, or to cover the myriad of variations of abutments, piers, towers and pylons in use around the globe. The aim in this chapter is to describe the overarching principles governing selection of bridge substructures, provide an introduction to the parameters to be considered during design, and direct the reader to sources of further guidance.

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Introduction

Ask someone about a famous bridge and few people will describe its substructure to you. Yet the choice of substructure, its design and its detailing, have a disproportionate impact on the success of a bridge project, from construction through to the end of its design life. Within the context of a major project, the cost of the bridges may represent only a small part of the total contract value, but the construction of the bridge substructures can have a major impact on the overall contract programme, since this normally occurs at the same time as earth-moving and drainage operations and is inevitably on the critical path for the construction.

The cost of providing the substructure to a bridge deck often represents more than half of the total bridge price. In spite of this, current design practice and rules often require that as much as 90% of the total design time is spent on the analysis and refinement of the superstructure. Of course, decisions regarding the superstructure and substructure are intrinsically linked. Early-stage consideration of the bridge articulation affects the loading regime from deck level to foundations, and one bridge element cannot be considered without the other. For integral and semi-integral construction, which is quickly becoming the norm for 'standard' bridge structures, the relationship between the superstructure and substructure continues throughout detailed design.

The design and detailing of the substructure determine many of the maintenance and replacement requirements throughout the full life of the structure. Poorly designed substructures that cause unnecessary additional direct costs and consequential indirect costs will undoubtedly cause an overall increase in

the construction price. A simplification in design or detail that leads to a speeding up of the construction process should be welcomed, as the real saving may not necessarily be in the individual price of a bridge. It may, for example, be in a reduction of the operating costs involved in the hire of an earthworks fleet. Similarly, good detailing that reduces the requirement for maintenance will minimise whole-life costs, as well as those associated with construction.

Substructures for bridges fall into two distinct categories – end supports and intermediate supports. The end supports are normally described as 'abutments', while the intermediate supports of a multispan bridge are termed 'piers' (Figure 7.1). The abutments and piers are usually constructed from in situ concrete, but precast sections can be employed to speed up the construction process.

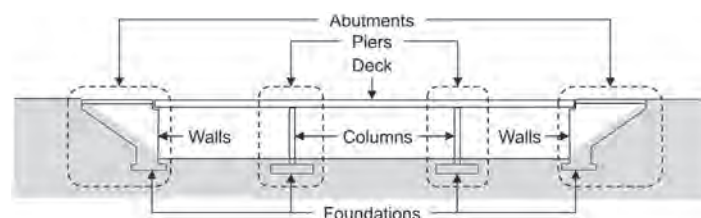


Figure 7.1 Abutments and piers

In integral bridges, the tops of the piers and abutments are built into the deck; semi-integral bridges are built in at piers only, with abutments on sliding bearings. Integral bridges have become the standard solution in recent years for bridges with overall length less than 60m, although recent projects

demonstrate that this form can be applied to longer structures too. This minimises whole-life costs and problems associated with maintenance and replacement of bearings, and inexorable leaking of the expansion joints at the ends of a deck.

Fundamental to the decision-making process when selecting substructure types is an understanding of the ground conditions. The type and strata of the soils are likely to be the most uncertain site conditions, particularly in the early design stages. Initial proposals are therefore reliant on information available at the concept stage; this is likely to be desk studies and publicly available data from historic local investigations.

As the project develops, comprehensive ground investigation is vital; specification of such works must ensure that this process returns high-quality information and is extensive enough for detailed design. Once detailed geotechnical information has been ascertained, it may be necessary to revisit the initial concept and reassess substructure types in the light of the new data. Specification and interpretation of geotechnical investigations is outside the remit of this publication, but extensive guidance is available elsewhere (e.g. BS EN 1997-2:2007 (BSI, 2010a), BS 8004:2015 (BSI, 2015a), *ICE Manual for Geotechnical Engineering* (Burland *et al.*, 2012a, 2012b), to name a few).

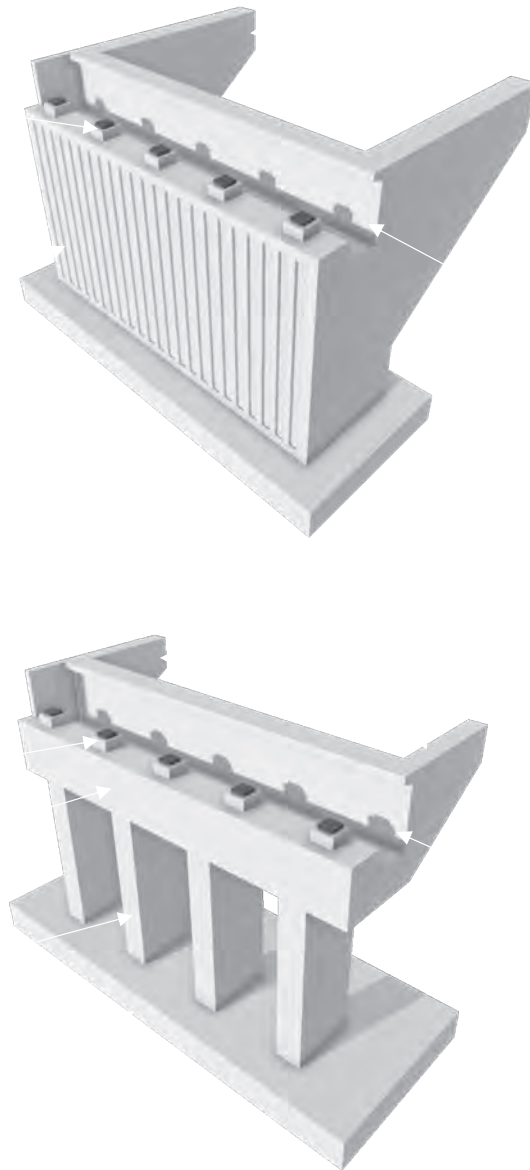


Figure 7.2 (a) Solid and (b) skeletal or 'spill-through' abutment

Types of abutment

The selection of appropriate abutments for a bridge should be made at the same stage as the choice of the deck superstructure. There are many types of abutment in use around the world, the most common of which are discussed herein.

Solid, or full-height, abutments (Figure 7.2(a)) are common but are often not favoured on the basis of aesthetics and can tend to restrict the line of sight of traffic on slip roads. In addition, they can produce a tunnel-like appearance on wide structures. Profiled finishes are commonly specified within the external concrete of solid abutments and adjoining wing walls in an attempt to break up this solid face and 'soften' their appearance.

Skeletal abutments and mass concrete bank seats are commonly employed where open side spans are required (Figure 7.2(b)). The bank seat is more suited to the top of a cutting slope, where simple footings can be used just below the existing ground level. However, it can also be applied to the embankment situation, where pile supports driven through the fill may be necessary (Figure 7.3). The cost of piling will influence the relative economy of this choice of end support.

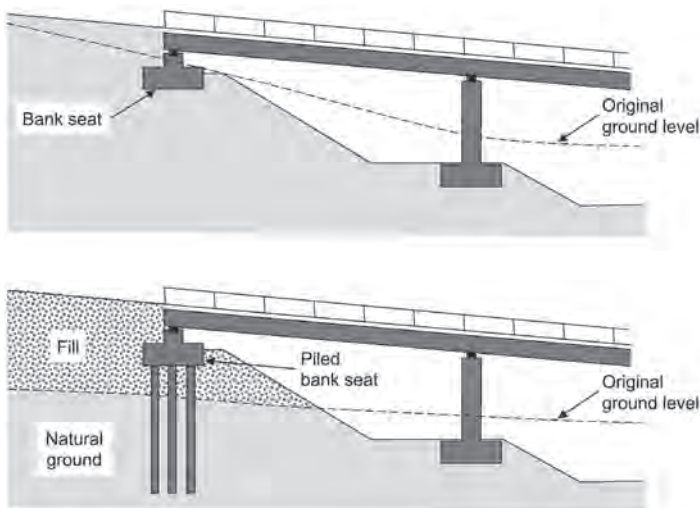


Figure 7.3 Bridge deck supported by piers and bank seats

In the embankment case, a skeletal abutment founded close to the previous existing ground level can be more economic, but there are complications in the construction of this type of structure. The contractor is faced with a severe restriction on backfilling operations and casting of the transverse capping beam causes further delay to the construction sequence. The magnitude and distribution of the earth pressures acting on the buried columns are particularly in doubt and limited experimental evidence is available to form the basis of a realistic theoretical analysis.

Despite the reservations expressed here, a solid wall abutment design (Figures 7.4 and 7.5) is often favoured in practice because it produces a minimum overall length for the

superstructure. An open side span solution produces a more attractive appearance and can be used to assist with farm access or provide additional areas for flood relief in a river crossing. In the case of bank seat and skeletal abutment forms of construction, the savings arising from smaller material content in the abutments and smaller amounts of granular backfill have to be offset against the cost of the additional deck area required.



Figure 7.4 Solid abutment using secant bored pile walling, A12, Suffolk, UK



Figure 7.5 Solid abutment using steel sheet pile walling, A12, Suffolk, UK

Cantilever abutment walls

The minimum headroom for new highway bridges is usually more than 5.3 m, so the overall height of a solid wall abutment is automatically in the region 7–9 m. This height is beyond the economic range of mass concrete walls and has encouraged the widespread use of reinforced cantilever abutment walls around the world. As integral bridge construction has become more prevalent, the popularity of cantilever abutments has decreased, but they remain common. The simplicity of this form of construction and the similarity with cantilever retaining walls also accounts for its economic success and popularity.

Free cantilever abutment walls

Plane cantilever abutment walls are the most common form of construction for heights of 6–9 m and, in spite of the size, the main concrete wall is often poured in one lift. The wall stem generally ranges from 0.9 to 1.2 m in width; the overall width of the base will generally be 0.4–0.6 times the height and the toe may project 1.0–2.0 m in front of the wall. However, the physical dimensions and proportions of the base will depend on the soil foundation conditions and the available resistance to sliding. A typical example of a cantilever abutment wall with horizontally cantilevered wing walls is illustrated in Figure 7.6.

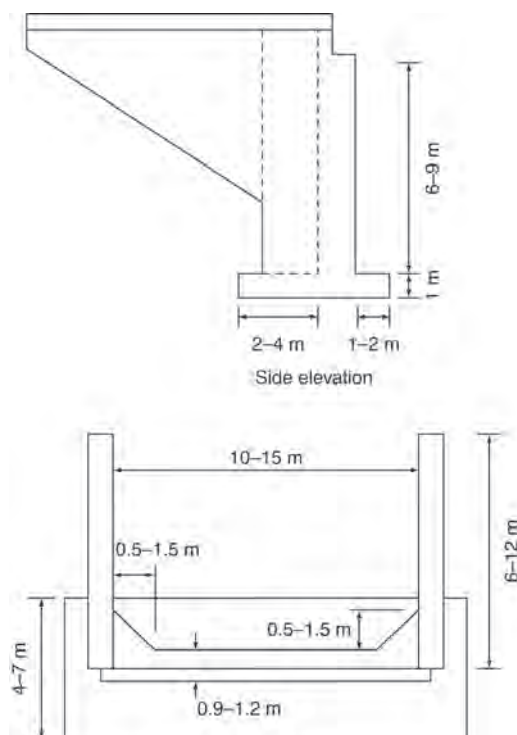


Figure 7.6 Typical cantilever abutment with wing walls (without abutment gallery)

Design forces acting on the abutment wall are often calculated on the basis of a metre-wide strip, assuming that the structure acts only as a vertical cantilever. If wing walls are attached to the rear of the abutment, as in Figure 7.6, there is a strong case for considering the three-dimensional behaviour of the structure. The combined effects of the wing-wall weight and substantial corner spalls can reduce the requirement for vertical reinforcement in the main abutment wall to a nominal percentage of the cross-sectional area.

An idealised system of vertical and horizontal forces acting on a simple cantilever wall is illustrated in Figure 7.7. Reliance on passive pressure at the front of the wall is unwise, since the soil may be completely removed during excavations for highway services along the toe of the foundations.

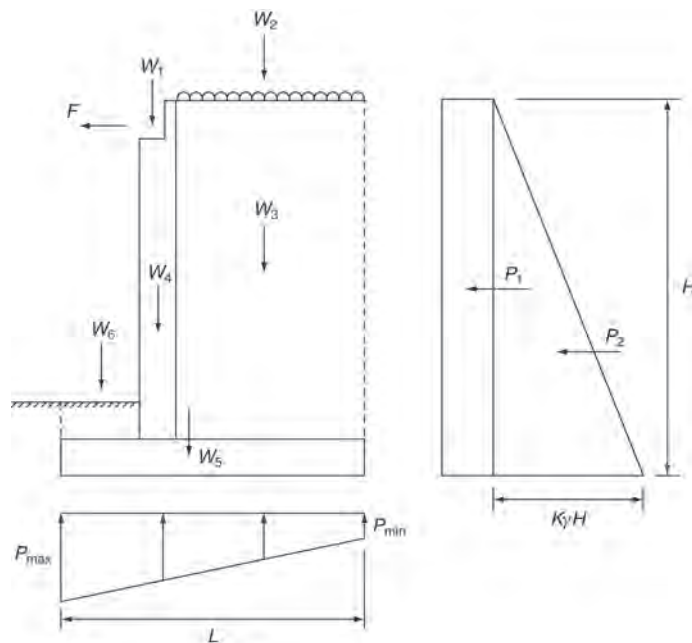


Figure 7.7 Idealised forces on an abutment wall

Counterfort abutment walls

Counterfort abutment walls become economic for heights greater than 10 m, where the percentage reinforcement in a free cantilever becomes very large. Triangular counterforts are added to the rear of the abutment wall slab to provide further flexural rigidity and resist lateral earth pressures developed by the depth of backfill material. The construction is complicated by the steel and formwork around the counterforts and physical compaction of the backfill is more difficult.

The counterforts are spaced at about one-half the height of the wall and are designed as vertical cantilevers. The wall slab may be treated as a slab clamped on three sides, although it naturally spans the shorter horizontal distance between the counterforts and the wall thickness can be reduced accordingly. The heel of the base slab also spans between the counterforts. However, there is usually little scope for reducing the thickness because the anchorage length for the main tensile reinforcement at the rear of the counterforts is a limiting factor.

Embedded abutment walls

Embedded abutment walls are an extension of free cantilever walls, where installed piles are used as both the foundation and the retaining structure. Solid walls can be formed using concrete secant or steel sheet piles (Figures 7.4 and 7.5, respectively). In the former case, the 'female' secant piles are typically shorter as they simply provide infill over the exposed section of the wall. Contiguous piles can alternatively be used, with nominal clear space in between, subject to the soil type being retained. In this case, an in situ facing layer is typically provided, or a drained cavity with facing panels in front.

Embedded wall types are generally applicable when the final bridge level is similar to the existing ground level, since piles must extend to the top of the abutment (less cross-beam and pile breakdown). They are typically, though not exclusively, used when bridges are constructed using top-down methods.

Propped cantilever abutments

There is little longitudinal movement in bridge decks up to 12m in span, and it is possible to use the deck as a strut for a square bridge or a bridge with a small angle of skew. In more recent years, the tendency has been to design in situ bridges of this scale as fully integral, idealised as buried box structures (following the guidance in PD 6694-1 (BSI, 2011)), instead of propping the abutment with an independent deck. This leads to better durability, as described in the next section. Nonetheless, propped abutments may be advantageous when speed of construction is a primary concern, and when parts of the structure are precast, since in situ stitches are not required at the corners.

The abutments can be designed as a propped cantilever but, owing to the rigid nature of the structure, at-rest earth pressures are normally assumed for the design of reinforcement in the rear of the wall and footing, and for both stability and bearing pressure calculations. Complete fixity of the base is unlikely, so the front face reinforcement in the abutment is often estimated by assuming that the wall is pinned at the deck and base levels.

A flexible packing is normally used to separate the deck from the top of the abutments and the rear curtain walls at the top of the abutment are designed to withstand the propping force. It is often necessary to specify that initial backfilling should be limited to 50% of the abutment height to avoid rotation and horizontal deflection of the abutments. Completion of the backfill behind the abutment walls is then delayed until the deck has been constructed.

Integral abutments

Expansion joints and bearings are traditionally installed between the superstructure and the abutments in conventional bridges to accommodate the relative movement and prevent temperature-induced stresses from developing between superstructure and abutments. However, these expansion joints and bearings can bring serious maintenance problems. In the UK, a survey of a random sample of 200 concrete bridges (Wallbank, 1989) has confirmed that the most serious source of damage is de-icing salts leaking through or penetrating deck joints on to substructure components. This process causes corrosion and immobilisation of movement joints and bearings located at abutments and between spans and represents a major element of conventional road bridge repair and maintenance costs.

A survey of bridge owners in Great Britain undertaken by the Steel Construction Institute's Steel Bridge Group in 2019 (Hodgkinson, 2019) identified similar results. By far the most costly maintenance schemes identified in the survey were those associated with bearing maintenance and repair (average £1.17 million per scheme), while movement joint replacement and

maintenance represented the second-highest average annual maintenance cost for steel bridges in particular (£1464/year per bridge). (Painting, fatigue repairs and deck waterproofing maintenance accounted for the other highest-value items.)

Because of the problems with conventional bridges with joints and bearings, and the associated whole-life costs, the concept of physically and structurally connecting the superstructure and abutments to create what is referred to as an integral bridge has become more popular. For integral bridges, all of the problems associated with joints and bearings, as discussed previously, are avoided.

In the UK, integral bridges have been encouraged for more than 20 years, especially by the publication of BD 57/95 (later updated via BD 57/01 (DoT, 2001a) and more recently superseded by CD 350), although this form of structure was adopted on many earlier bridges in the UK and overseas. Document CD 350 recommends that highway bridges up to 60m in length should be designed as integral by default, unless they are highly skewed or such construction is deemed inappropriate for other reasons.

Integral abutments can be solid cantilever walls, skeletal, embedded piled walls or bank seat arrangements. They can be combined with reinforced soil abutments, although the associated movement range is limited in this case. For longer bridges, lateral flexibility of the abutment minimises forces in the substructure, owing to longitudinal movement of the deck. This is commonly achieved by using single rows of piles, sleeved over the top few metres if necessary.

Owing to the integral connection between the superstructure and substructure, the abutments are forced to move away from the soil they retain when the temperature decreases and the deck contracts (e.g. in the winter), and towards the soil when the temperature rises and the deck expands (e.g. in the summer). This is commonly referred to as 'soil ratcheting'. As a consequence, the soil behind the abutment is subjected to temperature-induced cyclic loading from the abutment, which can generate much larger earth pressures than previously designed for.

Recognising the lack of standardised design guidance related to this effect, the Highways Agency published BA 42/96 on the design of integral bridges (DoT, 1996). Similar to the design standards before it, BS EN 1997-1 (BSI, 2010b) does not provide specific guidance for the design of integral bridges considering soil ratcheting. However, guidance has been published in the form of the non-contradictory complementary information (NCCI) document PD 6694-1:2011 (BSI, 2011). BA 42/96 (DoT, 1996) has been withdrawn but remains useful for reference.

Document PD 6694-1 (BSI, 2011) provides alternative methods for 'short' bridges, defined as having an expansion length (typically half the bridge length) of less than 7.5m. These are designed using the simplified process presented for buried concrete structures. Bridges with an expansion length above 10m are designed for the full effects of soil ratcheting. Bridges in between the two categories can be designed using the latter approach, or a hybrid method.

The type of soil being retained, and the type of foundation construction, affect the degree to which soil ratcheting occurs and the increase in pressures to be designed for. Document PD 6694-1 (BSI, 2011) provides guidance for different structural abutment arrangements, particularly for piled foundations, in various soil conditions. Soil–structure interaction analysis is recommended for abutments retaining granular fill and founded on a single row of piles. For piled or embedded integral abutments with cohesive soils, although daily and annual temperature will cause significant horizontal stress variations, a build-up of horizontal stress is not expected.

Open abutments

The term ‘open abutments’ is often used to denote the type of end supports required to extend the central span of a bridge to create adjacent ‘open side spans’. Two basic types of abutment are used in this situation, a mass concrete bank seat situated at the top of the slope containing a side span or a buried reinforced concrete skeletal or ‘spill-through’ abutment founded at previous existing ground level beneath an embankment slope.

In general, it is possible to choose between a single-span deck with solid cantilever abutments and a three-span deck with intermediate piers and end abutment supports. The relative costs of two large cantilever abutments, associated wing walls and selected granular backfill may therefore be compared with the price of two intermediate piers, two end abutments and two additional deck spans. However, other considerations are involved in choosing a three-span open structure, such as aesthetics, sight lines, flood relief and pedestrian safety.

Bank seats

Simple mass concrete or lightly reinforced sections may be used for abutment supports at the top of cuttings where the foundation level is close to the existing ground level (Figure 7.3). This type of structure is relatively small, and is usually ‘stepped out’ in section to reduce the foundation pressure and confine the resultant force on the bank seat within the middle third of its base. Small wing walls may be conveniently hung from the back of the bank seat to contain the immediate area of backfill behind the wall.

Bank seats may also be used on embankments, in which case they can either be allowed to settle with the fill or be supported directly on pile foundations. In the latter case, settlement of the embankment can cause down-drag on the piles and this reduces the payload of the pile group. Driving raking piles for a bank seat can be particularly awkward at the edges of an embankment and is not recommended should the embankment be expected to settle.

The cost of using a bank seat, intermediate pier and extra deck for the side span is often less than that of a solid abutment wall with large wing walls. This is particularly true for narrow bridges, but the closed abutment is generally more economic

for wide structures, since the cost of the wing walls is constant and represents a decreasing proportion of the solid abutment as the width increases.

Spill-through abutments

This form of abutment is illustrated in Figure 7.8 and consists of two or more buried columns supported on a common base slab and capped by a cill beam to carry the deck construction. The backfill spills between the legs and needs careful compaction around the columns to minimise long-term settlement. It is often used in an embankment situation where it is possible to obtain a suitable foundation at original ground level. In this case, it can form an economic alternative to a bank seat supported on piles driven through the embankment fill. A typical example of a spill-through abutment bridge is illustrated in Figure 7.9, although with the bridge completed it is not possible to see the legs of the abutment.

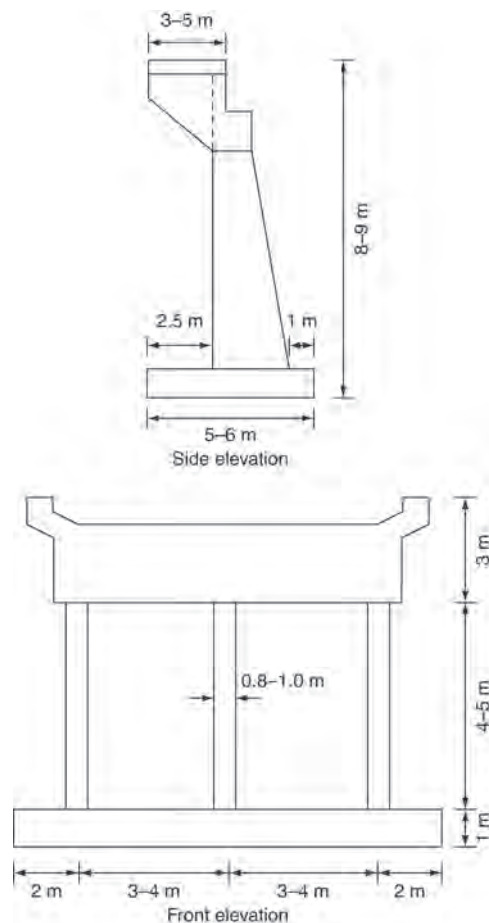


Figure 7.8 Typical spill-through abutment (without abutment gallery)

Design assumptions for this type of abutment vary widely, since very few field investigations have been undertaken to determine the long-term movements and earth pressures on the buried structure (Lindsell and Buchner, 1987, 1994). One con-



Figure 7.9 Spill-through abutment (in fill, to left) and piers, M25, Wisley, UK (before widening and central protection barrier)

servative simple design approach is to assume full active earth pressure across the entire width of the abutment, regardless of the soil that spills between the columns. Normally, the columns and cill beams are considered to be loaded by active earth pressure, but some arbitrary allowance is made for ‘drag effects’ or arching of the fill between the columns.

No additional calculations are really necessary for traction and braking forces acting towards the backfill. The total reaction from ‘at-rest’ earth pressure acting on the rear face of the capping beam and curtain wall is often sufficient to balance the longitudinal forces. If not, monitoring studies have demonstrated that the columns will rotate about a point above the base foundation and derive a very effective restoring moment from the lateral soil pressures with which to counteract horizontal loads at bearing level (Lindsell and Buchner, 1987).

Reinforced soil abutments

Reinforced soil, often referred to as mechanically stabilised earth (MSE), is a generic term, encompassing a number of systems employing a similar method for construction of retaining structures, including bridge abutments and wing walls. Each proprietary system differs, but all include these three key constituent parts, visible in Figure 7.10:

- hard facing panels, which are typically precast concrete but can be steel, wood or polymer
- earth backfill
- soil reinforcement consisting of either metallic strips or polymeric geogrids.

The abutment is built by repetition of a set of operations at successive levels: installing facing panels; placing and compacting backfill; laying soil reinforcement and further placing and compacting fill. This sequence is repeated until the desired height is reached. Reinforced soil abutments are usually com-

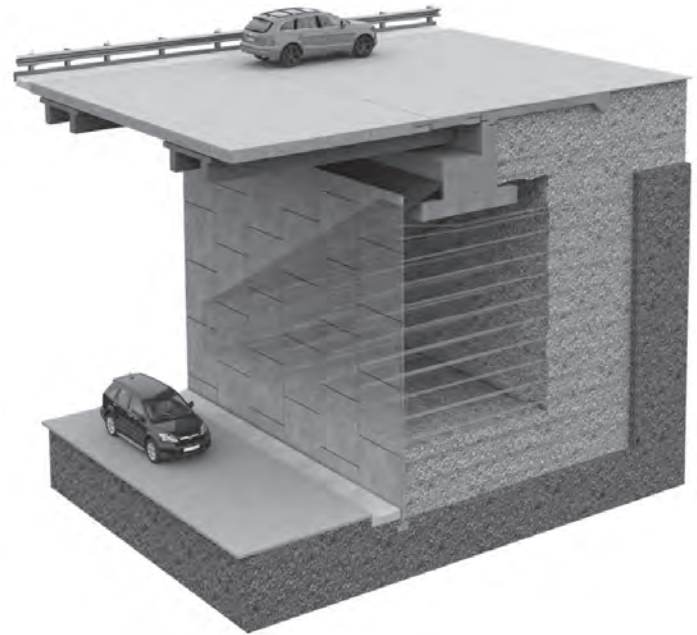


Figure 7.10 Reinforced soil abutment components (image courtesy of Reinforced Earth Co.)

bined with reinforced soil wing walls (as shown in Figure 7.11). In essence, reinforced soil walls are a form of gravity retaining wall, where a block of soil is tied together in order to satisfy the geotechnical limit states of the soil below. As such their design includes:

- checks for external stability, as with any gravity retaining wall
- checks for internal stability, that is, the reinforcement system within the soil.



Figure 7.11 Reinforced soil bridge abutment (image courtesy of Reinforced Earth Co.)

Reinforced soil can be used as part of an abutment when a bank seat is placed on top (Figure 7.10). In this case, the bank

seat is usually set back from the top of the wall to reduce any local loading effects that might cause local deformations on the face of the wall. Typically, reinforced soil is applicable for abutments with bearings permitting longitudinal movement, or for restrained or integral abutments where the longitudinal movement range transmitted into the fill is less than ± 20 mm. Another solution becoming increasingly common globally is to divide the abutment, with reinforced soil forming the approach embankments and earth-retaining structure, located behind structural columns, which independently support loading from the bridge superstructure alone.

This method of construction is typically faster than in situ concrete solutions, and often more cost-effective. The reinforced soil system can be utilised to reduce imposed loading on founding soils, owing to its wide base, while itself being capable of tolerating large settlements and supporting high loads.

Typically, granular fill is used (e.g. Class 61/6J, as defined in the UK *Specification for Highway Works Series 600* (DoT, 2016)). There is scope to use alternative fill types, which can provide environmental benefits, particularly if a surplus of site-won or local material is available. However, this should be considered early and with due consideration of the material's durability and compatibility with the reinforced soil system. Alternative fill types can potentially increase reinforcement quantities, testing requirements and potentially overall cost.

Reinforced soil walls are frequently combined with the other forms of abutment structure to provide cost-effective retaining structures around the approaches to a bridge. Where piling is required for abutments, steel or concrete piles can extend through the area of reinforced soil. For integral abutments combined with reinforced soil, sleeving of such piles over the top few metres is common to provide flexibility in the substructure, minimising forces exerted by longitudinal movements of the superstructure.

Reinforced soil wall design is outside the remit of BS EN 1997-1:2004 (BSI, 2010b), so design is generally undertaken in accordance with British Standard BS 8006-1:2010 (BSI, 2010c). Detailed guidance for the design of reinforced soil structures is included in Chapter 73 of the *ICE Manual of Geotechnical Engineering* (Burland *et al.*, 2012a, 2012b). Since the details of each proprietary system differ, it is common for a specialist reinforced soil subcontractor to undertake the detailed design. As a rule of thumb, for early-stage design, it is typically assumed that the length of reinforced soil block (measured parallel to the bridge deck) required is the larger of $(0.7H + 2\text{ m})$ or 7 m, where H is the height from the base of the soil block to the road surface level above.

Wing walls

The primary function of the wing walls on an abutment is to contain the backfill material at the rear of the abutment wall and minimise settlement of the carriageway. The combination of soil containment and compaction of the backfill material may lead to high lateral earth pressures. Consequently,

the horizontal forces acting on both the abutment wall and the adjacent wing walls can be similar and a very significant factor in selecting an appropriate design. The wing walls may be constructed as free-standing, independent structures or they can be designed as an integral part of the abutment wall construction.

Free-standing wing walls

A free-standing wing wall is designed as a nominal cantilever retaining wall with a separate foundation from the main abutment. Differential settlement and tilting between the abutment and wings may occur. Hence, construction joints between the two structures require careful design to both permit and conceal the relative movements.

To suit the local terrain, the wing walls can be arranged parallel to the abutment wall; this allows simple compaction of the backfill with no complications in the design, regardless of the skew angle of the deck. Alternatively, the wings can be designed to follow the direction of the over-road, and have the dual function of supporting the parapet fencing and the backfill. It is more difficult to place the backfill material with this configuration and larger earth pressures will result because of the restraint against sideways movement. Consequently, this form of design may be more expensive to construct; a cheaper arrangement can be to use wings splayed at 45° to the abutment and tapering in height.

Cantilevered wing walls

A second approach to the design of wing walls parallel to the over-road is to use horizontally cantilevered wings. This form of construction is practical for lengths up to 12 m from the abutment, but care must be taken in designing the junction between the wing and abutment wall. The structure has the advantage of being founded on a common base so that it settles as one unit, but compaction of the backfill may be difficult around the wings. The rigid nature of this type of design encourages high earth pressures; at least 'at-rest' earth pressures should be considered in a design (Ingold, 1979; Jones and Sims, 1975).

This type of abutment and wing-wall system forms a three-dimensional structure. A traditional metre-strip assumption is widely used but may not be an appropriate basis for a design (Lindsell and Buchner, 1994). The vertical and horizontal bending actions in the abutment are significantly modified by the presence of the wing walls; an overall reduction in the steel requirements is possible if the wings are used to their full advantage.

The self-weight of the wing walls should be considered, since they have a major influence on the stability and bending moments in the abutment wall. Horizontal forces on the wings are transmitted into the abutment corners and are distributed across the abutment wall. Therefore, the corner splays between the abutments and wing walls can be designed as vertical torsion blocks to carry the large torsional moments generated by the wing wall loading.

Abutment foundations

Most abutments are generally supported by either spread or piled foundations. Key issues to be considered in the choice of foundation include

- the available bearing capacity of the undisturbed natural soil at the site
- the settlement that the foundation will undergo (and impose on the superstructure)
- the tolerance of the abutments, deck, etc., to the expected differential settlements
- the provision of lateral flexibility where horizontal deformations are imposed on the abutment (i.e. integral bridges or in seismic locations).

In most cases where the bearing capacity of the soil is adequate to support a spread footing with limited settlements, this will be the most economic founding solution. Most dense sands and granular soils or stiff clays will provide suitable bearing capacities for spread footings; if rock is present at the founding level, this will inevitably lead to a spread footing solution. Piles are usually adopted where soft, compressible soils are present or when the abutment is located on the top of a high embankment. Piles can also be preferred to suit the construction methodology adopted. For bridges in a cutting, if a top-down approach is adopted, the use of piling can simplify the overall construction.

Often, if a suitable soil or rock is present within a reasonable depth of the founding level, it can be more economic to excavate any soft material and replace it with well-compacted granular material or mass concrete, rather than adopt a piling solution.

While spread foundations may be suitable at some sites, on the basis of bearing capacity, their size will often mean that total settlements will be relatively large. Typically, spread footings are sized based on a settlement of 25 mm under maximum serviceability loads, although this can be larger if the bridge structure as a whole is designed accordingly. By comparison, settlements likely to be experienced by a well-designed pile foundation are typically less than 10 mm at working load. The choice of founding method may then be governed by the capacity of the deck structure to absorb the differential settlements predicted.

It is difficult to estimate differential settlements with any accuracy, but experience suggests that they may be as large as 50%–70% of the maximum total expected settlement. Thus, for spread footings designed for a total settlement of 25 mm, assessment of differential settlement typically assumes 10–15 mm between piers. This is, of course, site-dependent and must be considered on a project basis.

While larger vertical stiffness is invariably preferable, for integral construction, or when seismic effects are significant, high lateral stiffness of abutments and piers can potentially be detrimental. Maintaining a level of flexibility in the abut-

ment system will reduce loads imposed on the structure, owing to deformations imposed from the deck or ground in these cases. For piled abutments, this tends towards a single row of larger-diameter piles, with the option of sleeving the top few metres if further flexibility is required.

Abutment run-on slabs and granular wedges

Bridge foundations and abutments are necessarily designed and constructed to restrict structural vertical movements, since these would be associated with differential movements, and would induce additional bending moments in the structure. Conversely, embankment fill adjacent to the deck can be expected to settle significantly (perhaps a few per cent of the fill height). Without special measures, vehicle ride quality will be poor on the approaches to bridge decks, because of the disruption to the intended vertical alignment of the highway pavement. Two approaches are possible: structural run-on slabs and the use of a granular wedge adjacent to the abutment.

A run-on slab will provide a smooth transition between the relatively flexible approach pavement and the non-flexible bridge superstructure by bridging the settling fill immediately behind the abutment. For abutments backfilled by granular materials, especially for integral connections, the backfill will become gradually compacted under horizontal cyclic loading from the abutment and a void will form under the run-on slab, which may cause damage to the slab under traffic load if it has not been designed for this case (Hoppe and Gomez, 1996; Xu *et al.*, 2007a). Run-on slabs are therefore typically designed to span simply supported over the fill material, extending the full width of the carriageway. Particular consideration must be given to the connection between the run-on slab and the abutment, in terms of both the structural design and waterproofing continuity with the rest of the deck.

Run-on slabs are generally discouraged in the UK, owing to concerns regarding maintenance and past performance, as outlined in BD 57/01 (DoT, 2001a). However, they remain a popular solution elsewhere around the world.

An alternative is to provide a wedge of granular fill, tapering upwards away from the end of the bridge deck. This granular wedge must be highly compacted to minimise any future consolidation and hence settlement of the road surface. With this approach it is inevitable that some settlement of the road will occur over time. However, where the road above is constructed with a bituminous wearing course, it is common that the road will be resurfaced at regular intervals, and any settlement of the fill can be compensated for during these routine maintenance activities.

Access chambers

Adequate provision for access is required for the purposes of cleaning and painting; maintenance; jacking, removal or replacement of bearings; and inspection of closed cell and box

members. Document CD 350 specifies that an access gallery (chamber) shall be provided below all bridge deck expansion and rotational joints (shown in Figure 7.2). The width and headroom clearance of galleries shall preferably be at least 1000 mm × 1800 mm, respectively, and shall never be less than 800 mm × 1500 mm (dimensions taken from the now-superseded BA 57/01). Access galleries should be fully enclosed by the abutment and masking walls, with secure doorways providing at least two points of access/egress at convenient locations easily reachable via footways from safe maintenance areas above or below the structure.

Abutment design calculations

The primary function of an abutment wall is to transmit all vertical and horizontal forces from a bridge deck to the ground, without causing overstress or displacements in the surrounding soil mass. The abutment wall also serves as an interface between the approach embankments and the bridge structure, so it must also function as a retaining wall.

The degree of interaction between a bridge deck and an abutment wall depends largely on the nature of the bearing supports, if provided. There is an increasing trend to use integral abutments, since this avoids the additional costs of bearings, and their maintenance. With this form of construction, bending moments transferred at the abutment–deck interface can be significant; there is potential for these regions to suffer from reinforcement congestion unless due consideration is given to detailing from the outset.

The effect of bearing type, end fixity or free supports can readily be idealised in the design process. However, the influence of ground movements, owing to settlement, mining subsidence or earth tremors, is more difficult to anticipate and such effects require individual consideration for a particular structure.

The primary vertical loading acting on an abutment is due to the dead load and live load reactions from the bridge deck. Additional loading arises from the self-weight of the abutment, earth pressure, wall friction between the backfill and the abutment, and live loading immediately behind the abutment.

Traction and braking forces from live loads on the deck are carried at the fixed bearings and may represent a substantial overturning moment on a tall abutment. Although these forces are applied to localised areas of the deck, they can usually be treated as a uniform load across the width of the abutment.

Replacement of bearings during the design life of the structure is likely to require an arrangement of temporary jacks on the abutment in between the permanent supports. This results in an alternative load path during such works, often while the bridge remains open and therefore subject to full live loading. Adequate space should be provided on the bearing shelf for an arrangement of temporary jacks and supports, and any assumptions or constraints regarding temporary support locations should be communicated on the design drawings. Analysis and design of the abutment and superstructure above must include consideration of this alternative load path.

Applied soil loadings

Earth pressures acting on the abutment are generally assessed using the same methods as for retaining wall design.

Typically, stability checks are undertaken using lateral pressures based on the active pressure coefficient, K_a , since a loss of stability would be associated with large displacement away from the retained soil.

Structural checks are based on the at-rest pressures, that is, using the coefficient K_0 , assuming that the deck movements are isolated from the abutment wall via sliding bearings. This is a conservative stance, since deflection of the wall will result in lateral soil pressures reducing towards the active state. The availability of simple soil–structure analysis enables the intermediate pressures to be predicted for a given situation, although due regard must be given to the uncertainty associated with properties of the soil and the structure. The potential of the deck to prop the top of the wall must also be considered, as this will affect the distribution of lateral pressure down the wall's height.

Standard BS EN 1997-1:2004 (BSI, 2010b) notes that, for normally consolidated soil, if the movement behind a retaining structure is less than 0.05% of the retained height, at-rest pressures should be assumed. A design method for calculating intermediate pressures (i.e. between at-rest and active) is given in Annex C of that standard. In lieu of such detailed analysis, standard practice is to calculate bending moments and shear forces using at-rest pressures.

Where the abutment retains a free-draining granular fill placed in situ, and sufficient drainage is provided to prevent the build-up of positive pore pressures, the effect of water is commonly discounted when assessing bending moments and shear forces in the abutment. Such a decision must be taken with due regard to the specification of the fill material, the back-of-wall drainage system in place, and its likely maintenance over the design life of the structure. Where silts or clays are retained, BS EN 1997-1:2004 specifically mandates that water pressures should be based on a water table at the surface of the retained material, unless a reliable drainage system is installed.

Increased lateral pressures can arise as a result of the following.

- Seasonal expansion and contraction of bridge decks supported by integral abutments, or propped by fixed bearings, will lead to seasonal increases and decreases in the abutment loading, and can lead to progressive increases in lateral earth pressure for granular backfills (Clayton *et al.*, 2006; Xu *et al.*, 2007a, 2007b).
- Surface-level loading behind the abutment can increase earth pressures on the back of the wall, caused by highway, railway or construction plant. Such repeated loadings are often arbitrarily treated as an additional surcharge loading on the soil surface.
- To restrict self-settlement, granular backfill will be compacted in layers, and compaction pressures could be greater than active pressures near the top of an abutment (Clayton *et al.* 1993). Integral abutments do not allow forward movement at the top of the wall during compaction.

- The use of compacted stiff clay behind abutments should be discouraged, since swelling of such materials can lead to very large earth pressures on wing walls (Clayton *et al.*, 1991; Mawditt, 1989; Symons *et al.*, 1989).

These are discussed in more detail next.

Effects of seasonal deck expansion and contraction

Longitudinal movements in the bridge deck due to creep, shrinkage and temperature changes cause forces at bearing level on non-integral abutments. The magnitude of these forces depends on the shear characteristics or frictional resistance of the bearings. The coefficient of friction of most bearings lies in the range $f_i = 0.03$ – 0.06 . The frictional force is derived from the nominal dead load and the superimposed dead loads on the deck.

Integral abutments do not have bearings; therefore, the backfill they support is subjected to seasonal increases and decreases in horizontal strain. The deck is stiff relative to the backfill and the soil provides insufficient restraint to prevent movement.

Earth pressures on integral abutments rise and fall seasonally; two effects have been observed.

- It appears from research that the effects on heavily overconsolidated stiff clay can be predicted on the basis of estimated soil stiffness and the effective seasonal temperature change.
- In granular soils, earth pressures may rise progressively, eventually approaching or exceeding passive values based on K_p . Methods have yet to be developed for dealing with, or avoiding, such high pressures. Indeed it is not yet known exactly what type of soil is likely to show this behaviour.

In the UK, the Highway England standard BA42/96 (DoT, 1996) was traditionally applied to derive the earth pressures behind integral bridge abutments. With the introduction of the Eurocodes, BS EN 1997-1:2004 (BSI, 2010b) did not provide equivalent guidance; this has since been incorporated into PD 6694-1:2011 (BSI, 2011). Simplified general rules are given for derivation of K^* pressures, which are a function of the retained height and movement at the top of the wall, combined with the coefficients of at-rest and passive earth pressure, K_0 and K_p , respectively. The resultant design pressures based on K^* can be significantly larger than the at-rest pressures, and in some cases even larger than those based on K_p .

As has been mentioned in a previous section, the methods prescribed by PD 6694-1:2011 depend on the bridge length, abutment type and soil conditions.

The K^* pressures do not represent a major problem for short bank-seat-type integral abutments, owing to the limited height, but can be more significant for full-height walls, as large bending moments can develop in the midheight of the wall and may cause structural damage or failure if the appropriate earth pressure has not been assumed during the design.

Vehicle loading or surcharge

The effect of external loads on earth pressures can be difficult to estimate using conventional earth pressure theory. In the simplest case, for example, a distributed load (q kN/m²) at the ground surface, such as an HA loading, an additional stress equal to $K_a q$ or $K_0 q$ can be added to the earth pressure assumed on the back of the abutment. If the wall is effectively rigid and does not yield, two assumptions are common, that

$$\Delta \sigma_h = K_0 q \quad (7.1)$$

(assuming the pore pressure $u=0$) or, based on elasticity, that

$$\Delta \sigma_h = \frac{\Delta \sigma_v}{(1 - \nu)} \quad (7.2)$$

where $\nu \approx 0.25$.

Methods of dealing with more complex cases are given in Clayton *et al.* (1993).

Standard BS EN 1997-1 (BSI, 2010b) does not recommend specific surcharges, instead noting that designers should determine an appropriate value based on the presence of anything that could cause such a load at the site in question. It is, however, useful to have some frame of reference; the following live load surcharges were listed in Highways Agency document BD 37/01 (since superseded) and continue to be useful guide values:

- 10 kN/m² for highway distributed lane loading
- Up to 20 kN/m² for highway special vehicle loading
- 50 kN/m² for heavy rail (on areas occupied by tracks)
- 30 kN/m² for light rail (on areas occupied by tracks).

For areas loaded by plant during construction, a 20 kN/m² surcharge is commonly adopted; however this should be reviewed based on actual equipment anticipated, which can result in much higher local pressures.

Additional horizontal stress from surcharge is commonly taken as constant over the full height of the wall, with the magnitude derived using one of the two equations given. This assumes that a large area is loaded at ground level and that reduction with depth owing to lateral dissipation through the soil is negligible. Particularly for the larger equivalent surcharges described, which tend to be the result of local concentrated forces, this can be overly conservative, especially in the case of tall abutment walls. Cause 4.1.7 of CIRIA Publication C580 (Gaba *et al.*, 2003) provides guidance regarding calculation of lateral pressures on retaining walls owing to adjacent strip loads, line loads and point loads.

Compaction pressures

The application of compaction plant, such as heavy vibrating rollers, to abutment backfill in layers leads to temporary

but quite large increases in both vertical and horizontal stress within the fill. Some of these stresses remain locked into the fill, and can give considerable additional lateral loading on a cantilever abutment, particularly over the depth just below the top of the wall.

Figure 7.12 shows an idealised earth pressure profile for a compacted granular fill. Design calculations should use the method described in BS 8002:2015 (BSI, 2015b); related background theory can be found in CIRIA Publication C516 (Chapman, 2000). Approximately, for modern plant σ'_{hm} may be of the order of 20 kPa, z_c will be about 1 m, and h_c may be of the order of 3–4 m. Thus, compaction pressures are significant for small walls, and (as can be seen from the pressure distribution in Figure 4.12) also move the centre of pressure towards the centre of the abutment wall height.

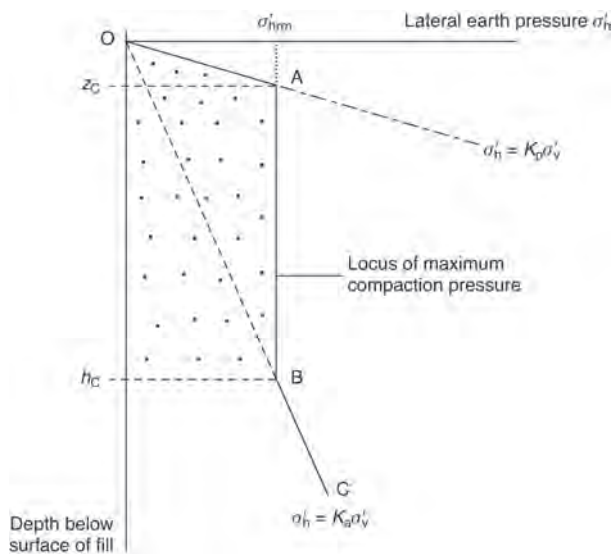


Figure 7.12 Compaction pressure – granular fill (Clayton *et al.*, 1993)

Swelling pressures

Compaction of cohesive fill produces even greater increases in lateral earth pressures than in granular fill, of the order 0.2 to 0.4 times the undrained shear strength. But for such clays the more significant issue is likely to be lateral swelling pressures. For clays placed relatively dry, a relaxation in lateral stress has been observed immediately after compaction (Sowers *et al.*, 1957; Symons *et al.*, 1989). However, as rainwater enters the fill, swelling starts to occur. In situ determinations of the average lateral stresses within a 6 m high abutment backfill of London clay (Mawditt, 1989) showed that horizontal total stresses rose up to 180 kPa near to the centre of the embankment, and up to 70 kPa close to the wing walls. In a pilot scale experiment, Symons *et al.* (1989) observed average lateral pressures on a 3 m high wall of the order of 100 kPa. Given that these figures are of many times greater than the commonly assumed equivalent fluid pressure, it is suggested that cohesive backfill should not be used behind abutments.

Earth pressures on spill-through embankments

The so-called ‘spill-through’ or ‘skeleton’ abutment (Figure 7.2(b)) is in common use for multispan bridges. In this form of structure, the abutment columns are embedded in the embankment, which slopes down away from the cross-beam beneath the bridge deck.

For this type of structure:

- the influence of the soil in front of the abutment should be considered
- soil arching and the effects of friction between the soil and the sides of the columns may be important.

In the Building Research Establishment survey of UK design practice carried out by Hambly (1979), it was found that out of 20 statements of current design methods given by experienced soils engineers for the design of spill-through abutments, there were 12 different methods in use. The four most common design criteria, in order of increasing magnitude of resulting earth pressure, according to Hambly, were as follows.

- For stable embankments with side and end slopes of 1 or 2 or less, it is argued that the spill-through columns will neither reduce the stability of the embankment nor significantly influence movements of the embankment. It is assumed that there will be no net lateral earth pressures on the columns.
- Chetty and Adams (1938) recommend that active pressures should be assumed to act on the rear face of the columns, but that the area of the column face should be arbitrarily doubled. Shear on the column sides is ignored. This approach is based on the assumption that soil behind the abutment flows forward between the columns, leading to increased column loads due to arching and column side shear. No pressure is taken on the front face of each column, on the basis that it is difficult to compact the soil in this zone. Stability is improved by the action of the self-weight of the fill acting downwards on the base slab.
- Huntington (1967) recommends that full active pressure should be assumed to act only on the rear face area of the columns if they are widely spaced, but should be assumed to act over the gross area if the space between the columns is less than twice the column width. In effect, this approach means that the soil is either assumed not to arch, or to arch perfectly between the columns. The fill in front of the columns is assumed to sustain up to active pressures, taking account of the batter slope, acting on the area of the column face alone.
- Full active pressure is taken over the gross width of the rear of the abutment, with no allowance for the soil resistance on the front.

Lindsell (1985) demonstrated, however, that most of the conventional assumptions are not justified. In reality, lateral pressures on spill-through abutments are balanced until the fill reaches the level of the top of the columns. If the bridge deck bearings are not free to slide during the deck pour, or the bridge deck is carried on some other part of the cross-beam during construction, large lateral pressures may result from deck

expansions immediately after the deck is poured. In any case, out-of-balance forces will result from the compaction of fill behind the cross-beam. Experiments by Moore (1985), coupled with observations of the deflections of the Wisley spill-through abutment (Lindsell, 1985), which indicate lateral movements at the top of the abutment that are never greater than about 5 mm, suggests the following.

- Neither the active state nor the passive state will be approached.
- Friction on the side of the columns is more likely to be mobilised than any significant proportion of passive resistance in front of the columns. This friction is likely to depend on the lateral pressure on the sides of the columns produced by compaction.

Long-term monitoring of total pressure cells mounted in the Wisley abutment with locked bearings showed that, with seasonal cycling, longitudinal earth pressures near the top of the 'retained' side of the wall approached passive values (see the section on seasonal deck expansion and contraction).

Top-down construction

For bridges constructed from the top down, loading on the abutment wall during excavation is akin to that experienced during deep basement construction. The soil behind the abutment remains in situ, as opposed to other methods where it is backfilled behind the newly constructed structure.

This construction method imposes loads on the abutment wall that are locked in and is heavily dependent on the interaction between the retained soil and the structure of the abutment, as both deform inwards during excavation.

The design method for top-down abutments is therefore similar to that for embedded retaining walls. It can be undertaken by hand using the limit-equilibrium approach or using appropriate soil–structure interaction software to model the wall throughout the construction sequence. Detailed guidance is provided in Chapter 64 of the *ICE Manual of Geotechnical Engineering* (Burland, 2012a, 2012b), BS 8002:2015 (BSI, 2015b), and CIRIA Publication C580 (Gaba, 2003).

Installing back-of-wall drainage is not feasible for top-down constructed abutments, since the existing ground remains in situ. As such, the design must consider horizontal pressures exerted by groundwater, in both the temporary and permanent cases.

Bearing pressures below spread foundations

Unless piled, the size of an abutment base is largely controlled by the allowable bearing pressure of the ground at relatively shallow depth. Traditionally, the variation in ground pressure across the base has been taken to be as illustrated in Figure 7.5, that is to say it is assumed to be linear, and the width of

the base is selected to ensure that there is no tension at the rear of the heel under working loads, i.e. using the medieval middle-third rule.

The assessment of a suitable bearing pressure for a bridge abutment is a complex problem.

- The base of a retaining wall, by reason of its intended use, can be expected to have both vertical and horizontal loads, as well as large moments applied to it. There is considerable disagreement about the prediction of bearing capacity for quite simple foundation loads, so that for retaining wall foundations the precise available bearing capacity is unknown.
- In clays, the bearing capacity of foundations is routinely treated as a problem that is critical in the short term, analysed using the undrained shear strength. The soil load on a wall (owing to active pressure) may increase with time. However, in addition, the footings may be constructed at the bottom of the cut, where swelling and softening will occur with time after construction.
- The simplified triangular pressure distribution cannot be sustained by soil, which yields locally and redistributes load.

Therefore, for a foundation supporting the loads from a retaining wall, the applied loads are complex, the time at which the critical (i.e. lowest) factor of safety occurs is unknown and calculated contact pressures are estimates. Since the wall loads may be applied rapidly, for example by compaction, the short-term factor of safety against bearing capacity failure under the maximum load should be calculated. For walls retaining in situ soils, the long-term applied pressure and bearing capacity (based on effective stress calculations) should be used. This will give a conservative design.

Short-term bearing capacity should only be used in saturated clays, because for granular materials and very dry cohesive soils 'drain' as rapidly as the load can be applied. Further guidance is available in many textbooks, for example, Simons and Menzies (2000), and in BS 8004 (BSI, 2015a).

Pile support

Because of the limited differential settlements that can be tolerated by most structures, it is likely that many bridge substructures will be supported on piles. There are two key issues in pile design.

- The selection of an appropriate type of pile for the ground and groundwater conditions, and taking into account local practice and plant availability.
- Design of the pile, to calculate allowable loads and estimate settlements.

Determination of ground conditions as close as possible to the abutment and pier locations is critical to this decision-making process. The value of well-specified, high-quality ground investigation in advance of the detailed design stage is hard to overstate.

Many problems occur because of the use of an inappropriate pile type, for example

- Lack of pile integrity (cast in situ or continuous flight auger (CFA) piles can be affected by high-groundwater conditions)
- Shaft softening may occur where piles penetrate water-bearing sand or clay (laminated) soils
- Pile drivability (precast concrete or steel piles may not penetrate to the required depths)
- Ground-borne vibration (driven piles may affect nearby structures, and residents).

The selection of an appropriate type of pile is probably the most important design decision, and is best made on the basis of experience of similar ground conditions, and with the support of piling subcontractors.

Many good texts describe pile design, and can guide the engineer (Fleming *et al.*, 1998; Tomlinson, 1994). However, it is important for the engineer to remember that one of the most powerful tools is precedent practice. Most piling contractors maintain records of their previous work and of load testing to verify pile capacity, and the larger contractors will have experienced specialist engineering design staff in-house. Early involvement with a piling contractor can be of great benefit.

Ground movements

Estimation of ground movements under the complex loading regime of a bridge abutment is extremely challenging. The advent of advanced geotechnical computer modelling means that solutions are, in theory, available. However, uncertainties about loadings, ground conditions, the relative stiffness of the structure and soil, and the load-settlement behaviour of piles and other foundation elements means that, in practice, a simple approach is normally used, such as the Boussinesq solution. Settlements are usually predicted on the basis of a uniformly distributed load of a spread foundation, a single pile or a pile group settlement. Since soil stiffness is non-linear in reality and depends on stress state and strain level (e.g. increasing with effective stress and decreasing with strain), the selection of proper soil stiffness will be essential to settlement prediction.

Most settlement of foundations on sands and gravels will take place immediately when the loads are applied. However, for clay, long-term settlement, e.g. consolidation and creep, will continue at a decreasing rate over a long period. Further guidance can be found in Tomlinson (2001), BS 8004 (BSI, 2015a) and BS EN 1997-1 (BSI, 2010b).

Stability

The stability of an abutment should be checked for four basic modes of failure:

- sliding
- bearing capacity failure
- overturning
- overall instability.

Historically, design to the British Standards considered stability effects using applied working loads and the derived factor of safety (applied loads divided by calculated resistance), compared with a suitable lower limit. The Eurocodes, now in common use in Europe and elsewhere around the world, instead require stability checks to be undertaken at the ultimate limit state. Loads are factored up or material properties are factored down, and the results compared to ensure that the design effects are less than the design resistance. Partial factors to be applied are defined in BS EN 1997-1:2004 (BSI, 2010b). Detailed commentary on the design approaches in the Eurocode can be found in Chapter 64 of the *ICE Manual of Geotechnical Engineering* (Burland *et al.*, 2012a, 2012b).

As such, a single factor of safety is not so clearly defined for comparison with the traditional design methods. The following discussion of the various modes refers to overall 'lumped' factors of safety, considering working loads, as this continues to be a simpler indicator, particularly for early design stages.

Sliding

When passive resistance in front of the toe can be relied on, the minimum factor of safety taken in design is normally 2.0. If the passive pressure contribution is neglected, a minimum factor of safety against sliding is usually 1.5. A shear key is sometimes provided in the base slab to mobilise greater soil resistance when otherwise the resistance to sliding is inadequate.

Typical design values for the coefficient of friction between the base slab and the soil are

clay 0.2 ($\phi' = 12^\circ$); sand 0.4 ($\phi' = 24^\circ$); gravel 0.4 ($\phi' = 24^\circ$) (7.3)

These conservative values allow for the effective angle of friction between the base and the soil to be considerably less than its peak value.

Bearing capacity failure

The vertical pressure on the base is calculated by resolving the applied vertical load and overturning moment about the centre of the footing, resulting in a linear distribution of stress. The peak value is then compared with the calculated capacity.

Historically, such capacity checks utilised a high factor of safety against working loads, typically around 3.0. However, this was primarily driven by reducing pressures to a level that would result in acceptable settlement. The Eurocode design method will tend to result in a smaller 'overall' factor of safety, which, while satisfying the requirements of the ultimate limit state checks, may mean settlements under SLS load become the governing check. Hence, a separate SLS check is necessary, to avoid unacceptably high movements under working loads.

Overturning

Overturning is checked by taking moments about the toe when the most adverse load combination is acting on the structure. A minimum factor of safety of 2.0 is normally adopted, providing the resultant reaction lies within the middle third. If there is 'tension' in the bearing pressure at the heel, then a higher factor of safety may be used as a further precaution against failure.

For the Eurocode design approach, where overturning bearing capacity failure is considered at the ultimate limit state, this check is unlikely to be critical. The bearing pressures local to the toe will typically exceed the design resistance before the overturning check fails.

Overall instability

A slip circle analysis is essential for a bank seat form of construction and may be necessary for other types of abutment when the soil strata well below the structure is weaker than the soil layers at foundation level. Where soil strengths are based on tests, a minimum factor of safety would be 1.5. Particular care is needed during construction if an intermediate pier foundation is being excavated at the toe of a cutting slope, when there is a bank seat positioned at the top.

Structural issues

Base design

The toe of a base slab is designed to resist the peak ground pressures acting on the base, although some relief can be obtained from the self-weight of the toe and any superimposed fill. The heel must also be designed to resist upward ground pressure, but in this case the resultant shears and moments may be reversed by extreme loading conditions caused by fill, live load surcharge and self-weight. The base slab may be supported on piles; in this situation, the bearing pressures would be replaced by calculated loads in each pile.

Wall design

The stem of an abutment wall is designed to resist the bending moments and shears produced by the horizontal forces and, in the case of integral bridges, from the bending imposed by the deck. Direct stresses owing to vertical loads are normally very low and may be neglected for wall design. Significant in-plane stresses can develop at the root of torsion blocks on horizontally cantilevered wing walls. In the case of simple vertical cantilever walls, the critical section for moments and shear forces occurs at the root of the wall, while for integral bridges the top of the wall and connection with the deck can also be critical.

Concentrated horizontal loads can occur at bearing level and may also be present at the rear of the curtain wall, owing to traction and braking effects. Similarly, where parapets are founded on top of the wing walls, concentrated impact loads must be considered. A standard method is to distribute these loads vertically at 45° when calculating bending moments in the wall.

Stiffness of foundations

For the design of integral bridges, arches and other structures that interact with the soil in their overall structural behaviour, the stiffness of the foundations can be a critical factor in the design of the structural elements.

For an integral bridge, the stiffness of the soils below the abutment assists in resisting the applied loads while also restraining the structure against natural movements, such as creep and shrinkage of the concrete, or temperature fluctuations. Greater soil stiffness will increase load effects in one part of the bridge but reduce them in other regions. Therefore, a range of stiffnesses must be considered, to ensure that a safe design envelope is derived for each element of the substructure and superstructure.

In these cases, it is common practice to apply the half/double rule. That is, a best estimate of the soil foundation stiffness is derived, based on the available information and understanding of the soil–structure behaviour. The bridge design is checked with a foundation stiffness of half this value and again with double this value to ensure that the structure can cope with any variations in the soil properties that may be found on-site.

Types of pier

Most intermediate supports for bridge decks comprise a foundation supporting either discrete columns, as seen in Figure 7.13, or a wall, as seen in Figure 7.14. The latter arrangement is usually described as a 'leaf pier', with the support member generally having a width that exceeds four times its thickness and hence falling under the traditional definition of a wall rather than a column. Discrete columns may be grouped together to form transverse portal frames with a capping beam (Figure 7.15), or cantilevered crossheads extending from a single column may form a 'T-pier' (Figure 7.16). Where a number of columns are employed, these may share a single footing or be supported independently.

Leaf piers

This type of intermediate support is common in modern bridge construction, since it is economic to construct. It is usually designed as a solid reinforced concrete wall, with the overall length of the pier equal to the transverse width of the bridge deck. Hence, typical leaf piers for an overbridge can be 4–10 m in length when supporting a single carriageway. The least lateral dimension may be 0.5–1.0 m in width and it is common practice to taper the pier dimensions from the base up to the



Figure 7.13 Example of a column pier arrangement



Figure 7.14 Example of a leaf pier arrangement

soffit of the deck. The greatest transverse dimension is required at the top, in order to provide sufficient bearing support for the superstructure.

Vertical and horizontal loads transmitted from the superstructure disperse rapidly from the top of a pier. Hence, the overall design of a leaf pier is normally conducted on a metre-strip basis, assuming a uniform distribution of axial and bending effects. The magnitude of the axial compressive stresses in a concrete pier is normally between 0.5 and 1 N/mm² under dead loading and is unlikely to be more than 2 N/mm² under the most severe live load conditions. The degree of bending will depend on the articulation of the deck and the length of the superstructure.

Discrete columns

Individual concrete columns are often used to support footbridges and bridge decks with high skew or greater height than

the minimum headroom clearance. Columns may be vertical, inclined or even curved, to produce greater aesthetic appeal.

A column section is normally required to resist biaxial bending and significant axial loading. Concrete columns are therefore often circular or square, but hexagonal and octagonal sections are also common. A typical square section may have a side length ranging from 0.4 to 0.8 m, depending on the loading and height of the column. Axial stresses may amount to 3–5 N/mm² under full service loading, so that section dimensions are usually selected to avoid the need to consider axial stability.

A typical bridge column with a height of 6–8 m is normally designed as a ‘short column’. This situation can be achieved by an appropriate choice of cross-section and articulation conditions to control the effective length and eliminate slenderness effects.

Columns of this type that are located adjacent to a road or railway passing under the bridge can be extremely vulnerable



Figure 7.15 Portal frame columns

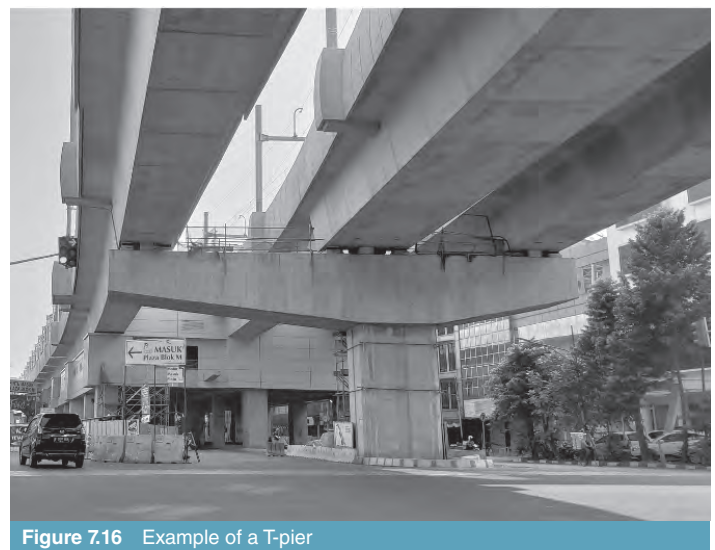


Figure 7.16 Example of a T-pier

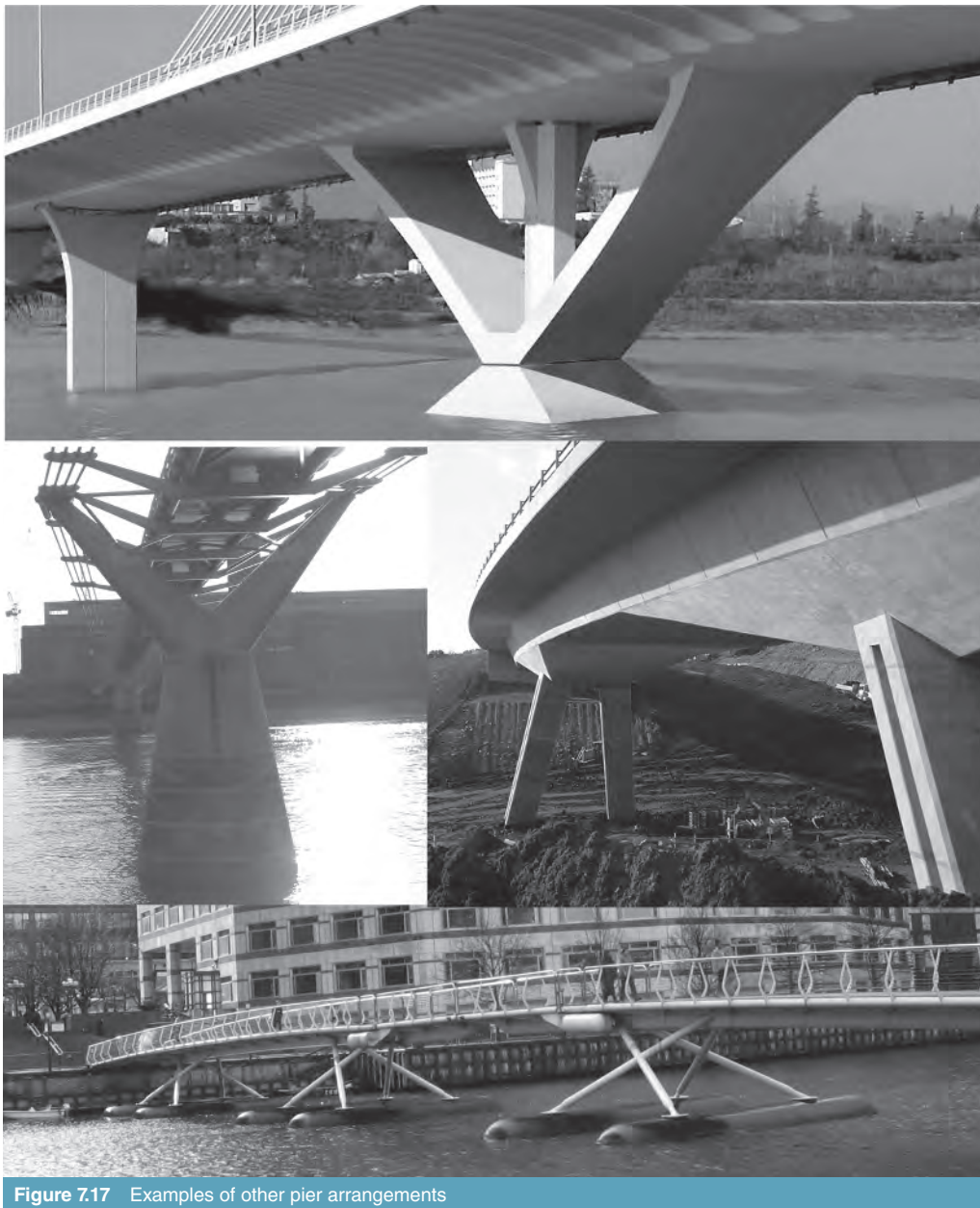


Figure 7.17 Examples of other pier arrangements

to collision impact from passing vehicles. In these cases, they usually require some form of protection or strengthening to resist the potential impact loading.

Arrangements during bearing inspection and replacement must be considered, particularly for column-type piers, where space at the top is limited. Where it is anticipated that bearing replacement will involve temporary jacks supported on the same columns, the column heads must be sized accordingly.

Portal frames

Concrete columns are often grouped into pairs or sets of three, by capping them with a transverse cill beam (Figure 7.15). The

cill beam provides continuous intermediate support for the superstructure, in a similar manner to a leaf pier. The transverse frame action developed by the capping beam, the columns and (when a single footing is used) the foundation slab produces much greater transverse flexural rigidity than a group of individual columns, but has no significant effect on the longitudinal flexibility. These characteristics are important where wind loading and vehicle impact effects must be considered, but the superstructure design requires flexible intermediate support to cater for longitudinal movements.

Again, protection for individual columns is usually needed where they are adjacent to a highway carriageway or railway and could be subjected to vehicle collision loading.

T-piers

Where available space at ground level is limited, a single pier can be used with a crosshead cantilevering either side to suit the width of the superstructure above, forming a 'T' (Figure 7.16). This is particularly common for construction of highways or light rail above existing road networks, where a number of piers would interfere with existing lane arrangements and sight lines.

The nature of such pier arrangements results in large forces being supported eccentrically to the pier; as such, significant crosshead and member sizes are required. Heavy reinforcement, and in some cases prestressing, is required to transfer the applied loads down to substantial foundations. In many cases, the most severe loading arrangements for such piers is during construction, where construction sequencing constraints may require the deck to be constructed asymmetrically.

Deflections of the crosshead cantilevers must be carefully considered, owing to differential loading, thermal effects and creep. This is particularly critical for rail systems, which are generally intolerant to vertical movement.

Other pier arrangements

Other pier arrangements can be used to enhance the aesthetics of a bridge by providing visual features from key viewing points, or they can improve the structural behaviour of the bridge by providing more efficient supports. Figure 7.17 shows just a few of the many variations of pier shapes and arrangements that have been used.

Pier foundations

The vast majority of piers are founded on either spread footings or piles, which are subject to the same considerations as for abutment foundations, discussed previously.

Numerous variations on foundation types exist; space within this chapter does not permit a comprehensive list. Caisson construction, which effectively provides a hybrid between spread footings and piles, is a common solution for major bridges. Tomlinson (2001) provides thorough guidance on caisson construction and design, as well as other bridge foundation types.

Pier design considerations

Many of the design considerations discussed previously for abutments apply to piers also; those are not repeated here for the sake of brevity and reference instead should be made to the earlier sections. For design of intermediate supports, considerations of slenderness and resistance to applied lateral loads tend to dominate more, and these are discussed in more detail next.

In the following sections, the term 'column' is applied to all pier types; the principles described below are equally applicable to leaf piers where the 'column' is often technically a wall.

Applied loadings

While the principal loading applied to most piers is vertical, horizontal load effects often govern, particularly as pier heights increase. Longitudinal loading from the superstructure is dependent on articulation, discussed in more detail in the following section. This can be substantial, highlighting the need for early decisions regarding articulation that consider the whole bridge structure.

Longitudinal and transverse forces resulting from wind, impact, braking, shrinkage and thermal effects all require detailed consideration. For bridges supporting continuously welded rail (CWR), traction and braking loads may be distributed by the rail over large distances and track-bridge analysis may be required to determine the load share carried by the bridge piers. On wide bridges and portal frame designs, the effects of shrinkage and temperature will play an important part in the design of bearings and columns. Biaxial bending will occur in such situations and the design must cater for the transverse behaviour. Skew bridges are frequently provided with circular or hexagonal columns to allow the biaxial bending effects to be simply treated.

Impact loading on piers

Equivalent static design forces for new structures are presented in EN 1991-1-7 (BSI, 2006a) for the case of collision with substructures of bridges over highways and railways.

For existing bridges over highways, CS 453 suggests that supports should be made sufficient strong or be adequately protected, to resist collision impact forces without allowing collapse of the supported structure. The nominal collision loads acting horizontally on bridge supports, together with their height of application are given in Table 3.2 of CS 453.

Articulation

The connection between an intermediate support and the deck largely determines the type of loading to be carried by the piers. Similarly, the structural connection between the columns and footing controls the degree of axial loading and the bending effects at the critical section immediately above the base.

For longer multispan bridges especially, the balance must be struck between providing longitudinal fixity at a larger number of piers to share longitudinal applied loads, and fixing at fewer piers to minimise substructure forces caused by expansion and contraction of the deck.

For bridges supporting rail, the interaction between the track and the bridge must also be considered. Movements of the deck induce stresses in the rails, which in turn act as structural members to distribute forces down the length of the bridge over long distances.

Thus, decisions about articulation of the bridge affect the design of the rail and vice versa, potentially requiring addi-

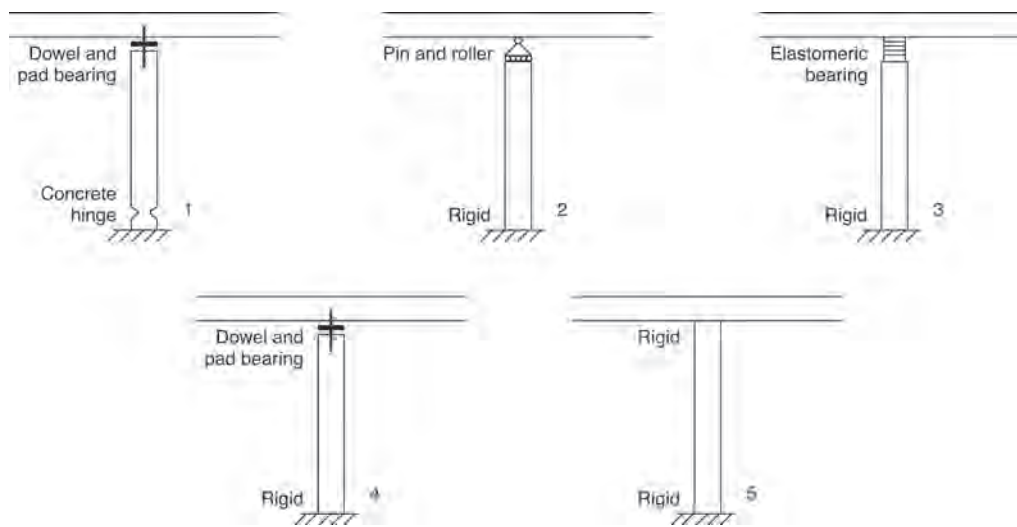


Figure 7.18 Articulation of pier columns

tional rail-breather joints or bridge bearings, all of which incur whole-life costs, owing to maintenance and replacement. Comprehensive guidance on rail–structure interaction can be found in UIC 774-3 (UIC, 2001), UIC 776-2 (UIC, 2009) and BS EN 1991-2 (BSI, 2010d).

The common forms of articulation for longitudinal movements in a bridge deck are illustrated in Figure 7.18. Columns of Types 1, 2 and 3 are required where large movements must be accommodated. The loading in any columns of this type will be predominantly axial, with relatively small bending moments. The bases of columns with Type 2 or 3 end conditions will carry bending moments, but a Type 1 column will transmit axial load only.

Columns with Type 4 or 5 end conditions will provide restraint to longitudinal movements of a bridge deck. Therefore, longitudinal braking forces and temperature effects will cause significant bending moments at the root.

Biaxial bending effects on columns may be particularly severe in a portal frame design, T-piers or wide bridge decks with rigid connections with the tops of the columns. It is important that the selection of bearings and end conditions should take all loading combinations and ranges of movement into account at the design and construction stages.

As with abutments, integral connection between piers and the superstructure are preferable, to minimise whole-life costs and risks associated with inspection, maintenance and replacement of bearings at pier heads.

Slenderness

The influence of buckling effects may become important if the unrestrained length of a column is large with respect to the column cross-section. Where the ratio of effective length (l_0) to

thickness (h) is limited to 12–15, the effects of slenderness are relatively small. This ratio will normally be satisfied for standard two-level structures, but may be exceeded at three-level interchanges and bridges crossing a deep-sided valley. When tall columns are necessary, the sections may be designed to be flexible so that movements at deck level can be accommodated through Type 4 or 5 articulation conditions. This should not usually present a stability problem if the effective length is less than the actual length (l_0).

The assessment of the effective length of a column is fundamental to the design of a section (Cranston, 1972; Jackson, 1988). Figure 7.19 illustrates the potential buckling modes and conservative estimates of the effective heights for the five articulation arrangements shown in Figure 7.18. In reality, most situations result in an intermediate case, as (for example) actual foundation stiffness is never truly rigid. The designer is left with the choice of selecting a suitably conservative simplified idealisation or calculating the actual effective length for the case in hand. With modern finite-element analysis software, the latter is reasonably straightforward, although published data for cases such as those in Figure 7.19 continue to provide a helpful check.

The slenderness ratio is the figure used for comparing the effective column height with the cross-sectional geometry. Historically, and within the British Standards, it was calculated by dividing the effective height by the thickness of the column. (In the case of tapered elements it is conservative to take the average thickness.) Eurocode 2 (BSI, 2015c), however, defines the slenderness ratio as the effective height divided by the radius of gyration of the uncracked column section. Both definitions remain in common usage; it is important to be clear which is being considered in any design checks.

Longitudinal movements in a deck can have a significant effect on columns designed with articulation types 3, 4 and 5.

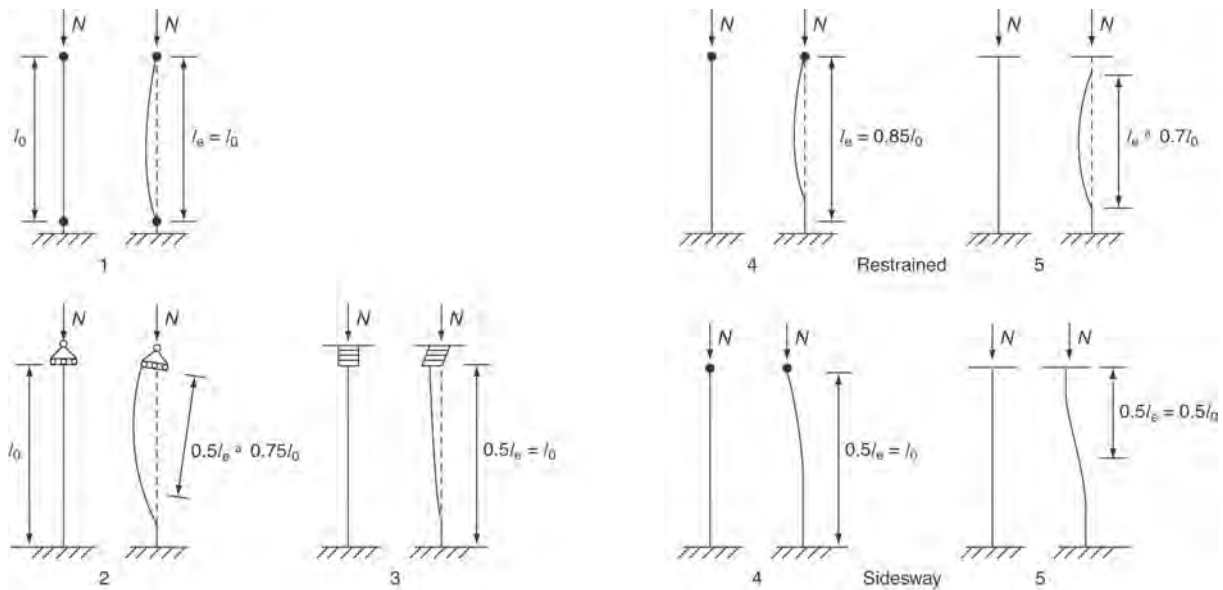


Figure 7.19 Buckling modes for columns and piers

A bridge deck may provide full restraint in the transverse direction, so the effective length is less than the actual length. In the longitudinal direction, thermal movements may create sideways; the effective length of a Type 3 or 4 column could then be twice the actual length. In addition, the effective length will increase, owing to flexibility of the base slab and soil foundation, which are unlikely to meet the fully fixed assumption illustrated in Figures 7.18 and 7.19.

The overall effective length of a pin-ended column with a flexible foundation may then amount to $2.5l_0$, when sideways occurs during seasonal movements of the deck.

Design approach and analysis

Many pier columns in the UK were designed for bridges on the basis of working stresses and the principles contained in CP114 (BSI, 1969) during the period 1955–1980. Permissible stresses for steel and concrete were given in BE1/73 (DoT, 1973), with reference to CP114 for permissible loads on short and long columns. Where the slenderness ratio (based on section thickness) was less than 15, the column was treated as ‘short’.

As this ratio increases above 15, the deflection effects become increasingly significant in practical reinforced concrete construction. The permissible axial loads were therefore reduced by a set of coefficients to allow for slenderness effects, in CP114.

The introduction of limit state design for reinforced concrete columns began with the first edition of BS 5400: Part 4 (BSI, 1978), and continues most recently in the Eurocodes. This fundamental change in design philosophy affected the analysis of slender piers and columns at ultimate conditions. However, little guidance was given in BS 5400 for the serviceability situation, and this remains the case in Eurocode 2. The uniaxial

compressive stress in concrete is limited to $0.6f_{ck}$ (comparable with $0.5f_{cu}$ in BS 5400) and this is likely to cause the serviceability condition to be critical. However, this depends on the value of the modular ratio (m) adopted in the calculations and little specific guidance is given in the codes. For simplified section checks, a value of $m=15$ is recommended, unless the dead load is less than 30% of the live load.

An initial analysis would normally be conducted on an elastic basis, calculating stiffnesses from the gross concrete cross-section of the column, and with the mean value of modulus of elasticity (see BS EN 1992-1-1, Clause 5.4 (BSI, 2015c)). It is suggested that, providing the ratio of effective height to member thickness is less than 25, no allowance for instability effects is usually necessary at the service load.

Consideration should be given to the increase in moments in the pier as the column top deflects under applied load, causing further eccentricity of the vertical forces in the pier (the so-called ‘P-delta’ effect).

Columns subjected to high bending moments are liable to suffer from tensile stresses at the extreme fibres. Crack widths for columns should therefore be checked in accordance with the appropriate design standard.

The modern design of concrete columns is based on ultimate strength criteria with checks for crack widths and stresses under service conditions. The principles were originally developed by Cranston (1972) during the drafting period for CP110 (BSI, 1972), which was intended for framed building structures. Consequently, the effective column heights did not deal specifically with the bridge columns of types 1, 2 and 3 illustrated in Figure 7.18. Additional information was introduced in BS 5400-4:1984 (BSI, 1984) for bridge column analysis at the ultimate limit state. Specific consideration was given to defining effective lengths for various bearing types and support conditions. The Eurocode provides equivalent definitions but with

different proposed values of effective length for a number of support conditions. As with the definition of slenderness ratio, therefore, care should be taken to maintain consistency with the applicable design standard, as mixing guidance from the different codes can be unconservative.

Where effective lengths are required for particular column arrangements not presented, this is simple to derive using modern finite-element analysis software. The stiffness of any restraining members should allow for cracking, unless it can be shown that such members remain uncracked at the ultimate limit state.

Eurocode 2 (BSI, 2015c) proposes three alternative methods for consideration of second-order effects caused by slenderness at the ultimate limit state:

- a 'general method' based on non-linear analysis, expected to be undertaken using finite-element analysis software
- a method based on nominal stiffness, which returns a magnification factor to be applied to calculated first-order moments
- a method based on nominal curvature, which returns an estimation of the moments due to second-order effects, which are then added to the first-order moments, including the effects of construction tolerances ('initial imperfections') and deck deflections.

The latter, 'additional moment', concept is similar to that employed in BS 5400 (BSI, 1984), which was developed from a greater knowledge of slender column behaviour in framed buildings. The basic principle is to estimate the deflections of the columns at collapse and calculate the additional moments arising from the deflections.

These methods accounting for slenderness should be considered in conjunction with second-order moments caused by P -delta effects.

Ultimate limit state checks are then undertaken, considering the combination of the first-order moments (including the effect of initial imperfections) and the calculated nominal second-order moment. The magnitude and distribution of the initial moments depends on the type of bearings and the degree of end restraint to the columns (Jackson, 1988).

The stiffness of the foundations can have an effect on the overall structural behaviour of the piers if they are built integral with the deck, similar to that described previously for abutments.

Construction

The construction sequence and concreting procedures are often left to the contractor's discretion, although the designer will normally specify the location of construction joints and design the reinforcement cages to suit.

In cases where the design is influenced by the construction methodology, however, communication of assumptions and limits is vital. Key assumptions made by the designer relating to construction sequence and temporary works should be clearly highlighted via the design drawings and designer's risk

assessment, alongside project-specific residual risks to be managed by the contractor on-site.

Abutment construction

A series of horizontal and vertical construction joints is normally required to build an abutment wall in two or three stages. A horizontal joint at 100–150 mm above the base slab, to form a kicker for the next lift of concrete, is essential to achieve accurate construction of the main wall of a cantilever abutment or the individual columns in a spill-through abutment. A further horizontal joint is usually introduced at 100–150 mm above the bearing shelf to enable the curtain wall at the rear of the abutment to be constructed as a third stage. However, a spill-through abutment design may also require an additional horizontal joint at the top of each column support to form the transverse cross-beam, which constitutes the bearing shelf.

Wide abutments and integral wing-wall designs may require the introduction of several vertical construction joints. Longitudinal vertical joints may also be necessary through the base slab or bearing shelf sections of an abutment. Wherever possible, the joints should be hidden in features such as recesses, or better still avoided on visible surfaces altogether to achieve a better appearance to the wall.

The upper 0.5 m of a wing wall often contains a string course section, which carries the bridge deck parapet fencing to the ends of the structure. The string course is frequently constructed separately from the main wing-wall stem, requiring a further horizontal joint at the top of the wing-wall section.

The location of such construction joints and the timing of the construction sequence might have significant structural effects and influence the durability of an abutment.

Substantial stresses might be induced by the thermal expansion and contraction of new concrete cast against previous sections. Additional stresses are likely to be induced by differential shrinkage between fresh concrete and older sections, and reinforcement must be designed and detailed to control cracking accordingly. Comprehensive guidance can be found in CIRIA Publication C766 (Bamforth, 2018), and in Eurocode 2 (particularly BS EN 1992-3 (BSI, 2006b)).

The timing of the granular backfilling behind an abutment should also be considered in the overall design of the bridge, particularly if 'rigid frame' or 'propped abutment' designs are used.

The construction of the bridge deck may represent the most severe load case in the service life of an abutment wall (Lindsell and Buchner, 1987). Potential interaction between an in situ concrete deck, the falsework and an abutment can be critical, owing to thermal expansion and early shrinkage of the deck.

Pier construction

The construction of individual in situ concrete columns is usually undertaken in two stages. A foundation slab with a construction joint is formed at 100–200 mm above the root of the column. For column heights of 5–10 m, the remaining height is

then poured in one lift. Although the critical section for bending effects is theoretically at the root of a column, in reality the construction joints just above the base slab will constitute a plane of weakness. Flexural cracks are likely to develop at this construction joint and ultimate collapse would probably commence at the joint section.

Concrete walls for 'leaf piers' are constructed in a similar manner, but an added complication may arise on wider structures. Where pier widths exceed 4m, vertical construction joints may be introduced during casting of the vertical wall sections. Differential shrinkage effects may become significant, since the temperature and age differences between the foundation concrete and the wall sections at the time of casting can be substantial. Consequently, vertical cracks are commonly found in the wall sections, immediately above the horizontal construction joint. The crack widths can be serious, unless adequate quantities of horizontal reinforcement are provided in the first few metres above the base. CIRIA publication C766 (Bamforth, 2018) provides extensive guidance on the control of such cracking by the provision of reinforcement.

The construction of portal frame supports may require a third stage in the casting process, to form the capping beam. Additional secondary effects in the transverse direction can be particularly severe, owing to the temperature rise and fall in the capping beam immediately after the initial 'set' of the concrete. Subsequent shrinkage of the capping beam and temperature differences between a backfilled foundation slab can also create significant transverse bending of the edge columns in a transverse frame.

Similarly, for T-piers, the capping beam is constructed in a third stage. Depending on the length of the cantilevers on either side of the column, this can require substantial falsework and propping. Where T-pier crossheads are post-tensioned, this introduces an additional stage and time in the process; prior to stressing, the capping beam must achieve a level of strength in excess of what would be required simply to strike falsework for a reinforced concrete element. Typically, this required strength is 25–35 N/mm² (cylinder).

For deck structures formed on steel or precast concrete beams with an in situ slab, the relative timings of constructing the integral connection and the deck slab affect the moments and shears in both the piers and the superstructure. Any such assumptions from the design process should be communicated clearly to the contractor in the design drawings. Careful detailing is required to avoid reinforcement congestion and clashes within the pier–superstructure stitch.

The construction effects produced by the addition of the superstructure may represent substantial temporary loading conditions on all types of intermediate support. Where early cracking occurs at construction joints and exposed surfaces, there is a long-term risk of water ingress and the development of a local plane of weakness under ultimate load conditions. The design of concrete columns and piers should therefore take into account the probable sequence of construction and appropriate reinforcement quantities and detailing provided, to cater for anticipated secondary effects.

Temporary works

Excavation costs associated with the construction of a deep foundation base for an abutment and any independent wing walls can represent a substantial proportion of the substructure price. Temporary stabilisation of the adjacent soil may be necessary in the form of sheet piling or de-watering operations. Such matters should be considered at the preliminary design stage, to select the most appropriate form of deck and substructure.

The principal cost of constructing the main abutment stem, wing walls and intermediate pier columns relates to the formwork and supporting falsework. The effective concreting pressures on the formwork will depend on the pour height, rate of pouring, section width and compaction procedures. Hence, the physical construction and the overall costs of the formwork and falsework will depend on these parameters. Any simplifications in the structure geometry and details should reduce the reinforcement detailing and formwork costs.

T-piers and portal frames require substantial falsework if the cross-beams are constructed in situ, or alternative temporary works (craneage, temporary prestress, etc.) if they are precast in pieces. These types of pier often require substantial foundations, particularly for T-piers, where overturning moments may be very large, with associated excavation cost similar to those for abutment footings.

Further guidance regarding temporary works required for concrete bridges can be found in the UK's Concrete Bridge Development Group (CBDG) Technical Guide No. 16 (CBDG, 2020).

Precast concrete construction

Precasting of parts of piers, or in some cases entire elements, is increasingly common and can bring substantial benefit in some circumstances, including



Figure 7.20 Precast shell for crosshead being installed



Figure 7.21 Precast pier being lifted into place



Figure 7.22 Aesthetic detailing on precast piers

- where access to pier locations is restricted or carries safety risk, for example in fast-flowing rivers or between carriageways of busy roads
- when particular finishes are required, which benefit from factory-quality conditions
- for larger projects, where the number of similar elements required, or the improved speed of installation, justify the up-front cost associated with mould production.

This may include the following.

- Precast shells of pile caps, piers and crossheads, filled with in situ concrete on-site. This eliminates falsework and can avoid additional temporary works in hazardous areas (e.g. cofferdams in rivers required for in situ construction), as well as providing a higher-quality finish. Figure 7.20 shows an example of precast shells for crossheads, providing a permanent formwork for an integral connection to the superstructure above.

- Construction of pier columns using precast blocks, which are stressed together, minimising on-site works and allowing rapid construction.
- Precasting of whole pier columns, fixed to the footing below with an in situ stitch (Figure 7.21).

Such off-site construction can reduce overall construction times, as piers and crossheads can be cast during, or even in advance of, the foundation works. It is particularly attractive for bridge sites where precasting facilities already exist in relatively close proximity, as this minimises the cost associated with set-up and transportation.

Precasting provides the opportunity for a quality of finish that is much more difficult to achieve on-site. This can include architectural finishes and quite intricate details, which can be worked into the moulds. Figure 7.22 provides one such example.

Wherever precasting is employed, careful detailing and consideration of the associated temporary works is required to ensure continuity with the in situ parts of the works.

Durability aspects

The durability of the substructure is a function of design decisions taken at every stage, from conception onwards, and of the method and quality of construction.

For countries with significant rainfall, water management is a consistent cause of poor performance in bridge works, associated with leaking joints and faulty or badly managed drainage systems. Where road salts are routinely applied, this exacerbates the issue and accelerates corrosion and deterioration.

Previous sections in this chapter have identified design decisions that can address this to some degree, but the quality of construction also plays a fundamental part in improving the durability of concrete substructures. Construction joints are a common area of weakness and must be well considered and executed, in order to avoid defects.

Horizontal construction joints in abutment walls will inevitably lead to vertical cracking in the concrete section immediately above the joint line, unless positive measures are taken to prevent this. A pattern of vertical cracks will emanate from the construction joint just above the base slab and may extend as far as 4–5 m up the abutment stem. In a typical highway location, the long-term effect of salt spray from passing traffic may lead to early corrosion of the front face reinforcement.

The introduction of a horizontal joint immediately below the string course on a wing wall can have serious consequences. Differential shrinkage stresses will probably initiate a regular pattern of vertical cracks, running completely through the string course section. Traffic passing over the structure will splash road salts directly onto the string course and there will be a high risk of corrosion in the parapet reinforcement. Such effects are easy to predict and steps should be taken at the design stage to minimise the consequences. The concrete mix should be designed to resist the penetration of chlorides. The effects of differential shrinkage may be reduced by careful selection of materials and timing of construction events. Crack widths and spacings can be controlled by judicious spacing and location of the internal reinforcement in the area immediately above a horizontal joint line.

Vertical construction joints can also lead to durability problems in pier and abutment walls. Similar consideration should be given to the control of crack widths and penetration of road salts into joints.

One of the primary areas of concern in all types of abutment wall, and at piers below expansion joints, is the top of the bearing shelf. Water running from the carriageway surfaces above will normally flow onto the bearing shelf; every effort should be made at the design stage to drain this water away using a positive drainage system. The majority of existing abutment walls, and a number of piers, suffer from water staining and chloride-induced corrosion, where inadequate provision has been made for effective drainage of the surface water.

Buried surfaces, including the rear faces of abutment walls, are normally protected with two coats of bitumen paint, prior to backfilling with granular material. Consideration should also be given to surface protection of the ballast wall and bearing

shelf concrete, which may be subject to continuous water leakage from the carriageway surfaces.

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Chapter 8

Reinforced concrete bridges

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This chapter outlines the use of reinforced concrete in bridges. Reinforced concrete is a versatile material, used in most modern bridges in some way. It can be shaped for architectural effect; it can be precast to standardise, and it is used in structures from small culverts to major bridges and viaducts. Reinforced concrete is one of the most economic forms of bridge construction for small and medium spans. This chapter outlines the common forms of reinforced concrete bridges and the key issues governing their design. The key design criteria for durability, serviceability and ultimate limit states are outlined.

Introduction

Reinforced concrete bridges are a very common and economical form of construction, with a wide variety of structural types. This chapter initially gives a short history of reinforced concrete bridges and then reviews the forms of structure in which reinforced concrete construction is used; each of the more common forms is then considered in more detail. Compliance with codes and regulations is necessary in the design of a bridge but is not sufficient for the design of an efficient, elegant and economic structure. An understanding of the behaviour, what physically happens and how failure occurs is vital to any designer. Without this understanding, the mathematical equations are a meaningless set of abstract equations. One aim of this chapter is to give an understanding of the key behaviours of reinforced concrete bridges.

History and future trends

Concrete has been used in bridges for over 2000 years; concrete arch bridges with spans of up to 30 m were built by the Romans (O'Connor, 1993). Plain concrete arches reappeared in Europe in the 1850s, with reinforced concrete arches in the 1970s. By 1900, various types of arch, truss and beam bridge had been developed and built in Europe and the USA. The UK was late in developing reinforced concrete bridges (Chrimes, 1996), with the major use occurring from 1900 to 1914, mainly using patented systems (particularly by Hennebique) from Europe. Examples are the River Wey Bridge, an integral beam-and-slab structure from 1913 (Figure 8.1), and the original Mizen Head Arch Bridge (Ruane *et al.*, 2013), built in 1910, which was replaced in 2009 by a replica (Figure 8.17). The materials in early bridges were somewhat different from the materials used today; concrete strengths were lower and more variable, with much less quality control on aggregates, the amount of chlorides, cement content and water content. The strength of the concrete was relatively slow to develop, with strengths being developed in 100–150 days rather than 28 days. Reinforcement

was usually plain mild steel bars or flat steel strips. Cover to reinforcement was variable, often less than 25 mm, with detailing rules variable by modern standards.



Figure 8.1 Beam-and-slab bridge over the River Wey, dating from 1913

From 1914 to 1950, there was some development of concrete quality and detailing and some iconic structures were built, those by Maillart (Billington, 1974), such as the Salginatobel Bridge (Figure 8.2), being the most well-known. Other developments were the cantilever construction of a reinforced concrete bridge over the Rio do Peixe in Brazil in 1930. Prestressing had been patented in 1888 and was developed by Freyssinet from the 1930s into its modern form. With most of the bridge types and technical innovations in place, the development of reinforced concrete bridges from 1950 was primarily in volume (numbers of bridges constructed) and developments in materials with the adoption of deformed bars with higher yield strength, although some important reinforced concrete structures, such as Waterloo Bridge (Astin, 2017) (Figure 8.9), were constructed. The importance of concrete cover and lower water contents in higher strength concrete for durability was also confirmed and covers to reinforcement near 50 mm are now normal for bridges. High-strength concrete has been developed, with strengths over 100 MPa made possible by using additives such as silica fume. Self-compacting concrete with superior flowability and no segregation was developed in

the late 1980s; it requires less skilled labour than conventional concrete. Non-metallic reinforcement, such as various types of polymer bar (see Table 8.1) have been developed; these have a higher strength but fail in a brittle way at lower strains and also with a lower stiffness than steel reinforcement, meaning that the serviceability issues of cracking and deflection are more prominent than the ultimate limit states of flexure or shear failures.



Figure 8.2 Salginatobel Bridge by Maillart

The sustainability of bridges has become a key issue and the reduction of concrete and reinforcement volumes to reduce the carbon footprint is important (Collings, 2006, 2020). The reduction in the amount of conventional cements used and the use of cement replacements, such as pulverised fuel ash (PFA) and ground granulated blast-furnace slag (GGBS) is more common, with lime-based concretes also being considered once more. More efficient structural forms are also vital; for concrete bridges, simple arch forms have been used from the start (Figures 8.2) and have a place in future optimised bridges (Figure 8.7).

Structural forms

Reinforced concrete bridges cover a considerable range from foundations, substructures and superstructures, with a range of forms; arches, slabs, beams, towers, tunnels, and so on. Reinforced concrete is also used in the decks of prestressed concrete and steel–concrete composite bridges (see Chapters 9 and 11). Most reinforced concrete structures are small span, with solid slabs used on culverts, cut and cover tunnels and integral bridges to about 20 m span, or beam-and-slab type structures up to about 30 m span. For longer spans, up to 50 m, reinforced concrete boxes may be used. Reinforced concrete is also used in the world's largest and tallest structures; the caisson foundations of the Akashi Kaikyo (Tanaka *et al.*, 1993) and Çanakkale Strait suspension bridges are of reinforced concrete and the slender piers of the Millau Viaduct are reinforced concrete.

Piles

Reinforced concrete piles from 300 mm to over 3.5 m diameter have been used to support structures; they transfer the load from the substructure to the ground. These piles are normally

circular and have to resist high axial forces with bending and shear.

Foundations

Reinforced concrete foundations, pile caps and caissons are used to support most bridges. For most bridges, the foundations are simple slab structures with low span-to-thickness ratios, such that strut and tie methods of analysis (Schlaich and Schafer, 1991) are appropriate. For large structures, caissons are used; these introduce large voids in the structure, with walls subject to significant in-plane and out-of-plane loads. For such structures, the integration of axial forces together with bending and shear using modified compression field theory can be appropriate (Vecchio and Collins, 1986).

Substructures

The abutments, bank seats, piers and columns that support the majority of bridges are of reinforced concrete. Abutments or bank seats form the ends of bridges; ideally, they will be integral with the superstructure. However, for bridges over 90 m long, bearings and expansion joints may be needed. Care should be taken with the design of joints and the drainage of water from abutments, as the majority of durability issues with reinforced concrete bridges relate to leaking joints or drainage. Where integral bridges are not used, inspection galleries under joints and around bearings must be provided (Highways England, 2001).

Piers form intermediate supports to multispan bridges; again, ideally, they will be integral. Shapes of piers vary, and may be wall-like structures, pier bents with columns and a cross-beam or simple column-only structures. Piers are often adjacent to the highway or railway and need robustness from impact forces to ensure that the bridge does not collapse if the pier is hit by a vehicle.

Towers

For large viaducts, like the Millau Viaduct (Figure 8.3), or large cable-stay or suspension bridges, the pylons or towers

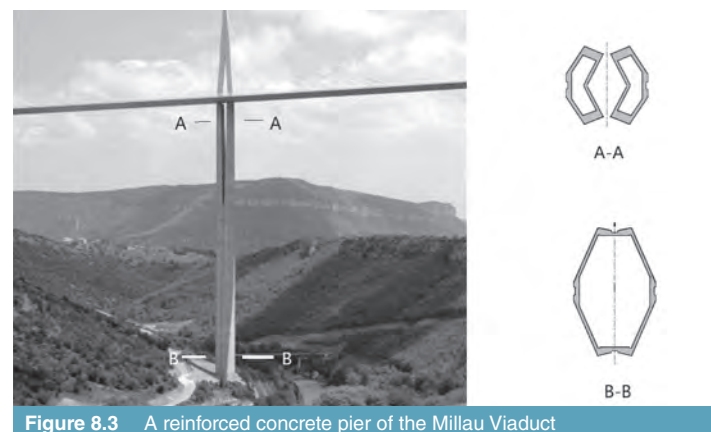


Figure 8.3 A reinforced concrete pier of the Millau Viaduct

will often be of reinforced concrete. The towers carry significant vertical forces; sections of the tower may be solid or voided and are usually constructed using slip- or jump-form techniques.

Pipes, culverts and tunnels

Pipes, culverts and tunnels are structures with a relatively constant structural shape that are surrounded with soil. The soil loads tend to dominate the loading effects, with live loads and temperature having less influence. The minimum soil depth to a buried structure is approximately 800 mm – less than this the effects of temperature – and other loads will be as conventional bridges. The walls, roofs and bases of these structures are usually solid reinforced concrete slabs (Figure 8.4). These walls and slabs will be designed to resist coexistent flexure, shear and axial effects. These structures are normally constructed in situ but, in some situations, can be jacked into position.



Figure 8.4 Entrance to a cut and cover reinforced concrete tunnel

Green bridges

Wildlife bridges or ecoducts have been used in North America and Europe to help the movement of wildlife through landscapes fragmented by roads or railways. Green bridges are also being used in Asia. Research has shown that road and rail corridors have the potential to be enhanced by providing connecting corridors across them, providing important habitats for some species, such as pollinators. Birds and bats are much more likely to cross major road and rail infrastructure at wide green bridges.

There are many shades of green bridges to consider, all the way to grey (conventional bridges). The definitions used here primarily follow the environmental considerations, rather than any structural configuration. A natural green bridge will be at least 70m wide, possibly more, depending on the landscape; most of the best of this type of structure are tunnels. The large width is aimed at the connectivity of ecosystems and providing habitat continuity across the bridge. A wild green bridge has a smaller width of 40–70m and is targeted at allowing specific

species to cross. A wild green bridge will not have significant provision for human access. The environmental credentials of the wild green bridge can be marred by high carbon footprints if soil is simply put on a conventional bridge and so lower-carbon solutions with earth-covered reinforced precast arch structures are often used; these structures are ideal for the permanent static loading of soil.

A mixed green bridge has a dual use, having an environmental wildlife function but also carrying a highway or footway, and is the most common form in Europe. The disturbance to ecosystems and wildlife from the human access normally means that the human access across the bridge is the prime driver for the bridge, while the provision of space for wildlife on the mixed green structure provides an additional benefit rather than being the core aim; these bridges are typically conventional beam-and-slab type structures. The addition of the green part is often seen as costly, increasing the bridge's area. However, where the highway crosses at a high skew, it is often economic to provide a wider bridge with a shorter span and no skew, avoiding costly skew joints and bearings and the additional maintenance requirements that come with them, utilising the additional free areas as the green wildlife area.

Light grey bridges are conventional structures with narrow green strips from 2 to 3m wide or grey-green bridges with wider green strips. These structures do not provide the full environmental benefit of green bridges, but the green strip may provide corridors for smaller wildlife species, birds, bats and pollinator plants. In more urban areas, these light grey bridges may become garden bridges. One of the main problems of using conventional bridges and adding green is the significant disruption of the green function during maintenance; having a habitat established on the bridge for 30 years then ripping it apart to maintain joints, waterproofing, and so on. Hence, all green and light grey bridges must be integral, without joints and bearings.

Natural green and wild green bridges have significant ecological benefits but come with potentially large environmental costs in terms of carbon footprint. There is a significant variation in the carbon footprint of both green and grey bridges (Figure 8.5).

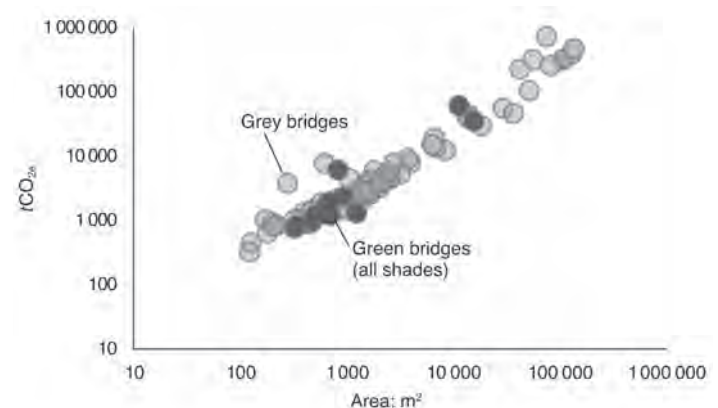


Figure 8.5 Carbon footprints of green and grey bridges

Integral bridges

Integral bridges, where the superstructure is connected to the substructure, are preferable to those using joints and bearings, as bearings and joints allow movement and are the major cause of water ingress and deterioration of the bridge. The integral bridge is more robust, particularly with regard to accidental impact, and requires less maintenance of expansion joints and bearings. The design of the integral structure is similar to a conventional bridge, but the effects of soil pressures and movements need to be considered more carefully (England *et al.*, 2000). In the UK, it is mandatory for structures less than 60 m long with skews less than 30° to be integral. Longer integral structures are often built (Cooke and Collings, 2018).

Continuous bridges

The use of a continuous beam or slab is usually preferable to a series of simply supported spans, as the number of bearings and expansion joints is reduced. Both joints and bearings are potential long-term maintenance liabilities. However, in this form of structure, the continuity moments over the supports are of opposite sign to those at midspan and the maximum moment and maximum shear may occur together. Making the deck continuous or building it into abutments (see the section on integral bridges) leads to a significant reduction in the mid-span sagging moments in the slab. With this form of structure, there will often be a redistribution of peak moments owing to cracking (Pisanty and Regan, 1998) prior to yielding and the more significant redistribution of moments that occur with the formation of plastic hinges. The advantage of this continuity in material terms is significant.

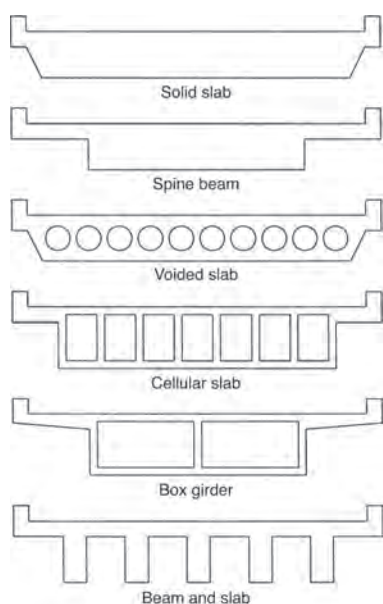


Figure 8.6 Types of reinforced concrete deck section

Bridge deck types

For reinforced concrete bridges, the deck slab could be of a number of types from solid slabs, voided slabs, spine beams, beam-and-slab or boxes (Figure 8.6).

Solid slab bridges

The solid slab is the simplest form of reinforced concrete bridge deck. Ease of construction resulting from simplicity makes this the most economic type for short-span structures. Solid slab bridges can span up to 20 m with haunches at supports. However, self-weight effects can be significant on slabs thicker than 600 mm, and 900 mm is typically the economic limit for this type of bridge. Solid slabs also have good distribution properties, making them efficient in carrying concentrated movable loads for highway bridges. Lantsoght *et al.* (2015) outline simple rules, assuming 1:1 spreading of loads to the supports to estimate shear (Figure 8.7).

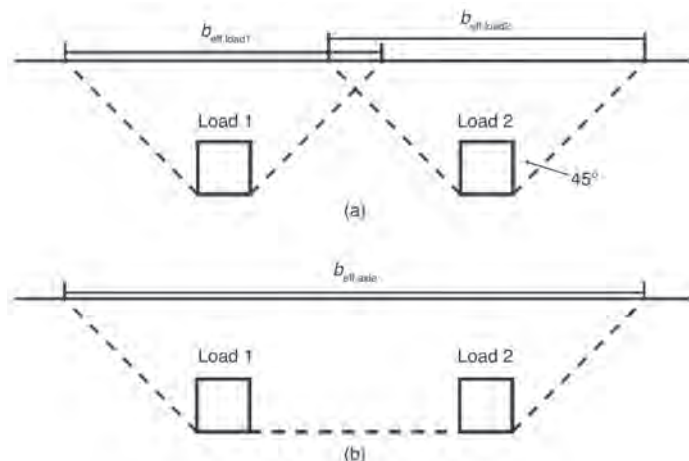


Figure 8.7 Spread of wheel and axle loads on a bridge

Reinforced concrete slab bridges are normally cast in situ. An exception is very short-span shallower structures (typically up to some 6 m span and 3.6 m height, see Figure 8.8), which can be most economically precast complete as box culverts or L-shaped retaining walls. The slab capacity can be increased locally by the addition of reinforcement. This makes slab bridges particularly suitable for geometrically complicated viaducts, such as arise in some interchanges in urban situations. Curved decks with varying skew angles and discrete piers in almost random locations can be readily accommodated.

Voided slabs

One of the commonest ways of lightening a solid slab is to use void formers. Voided slab bridges become economic above spans of 12–15 m. Voids are usually circular, with sufficient space between the voids such that the concrete can carry shear forces and that shear links or ties can be placed if required.

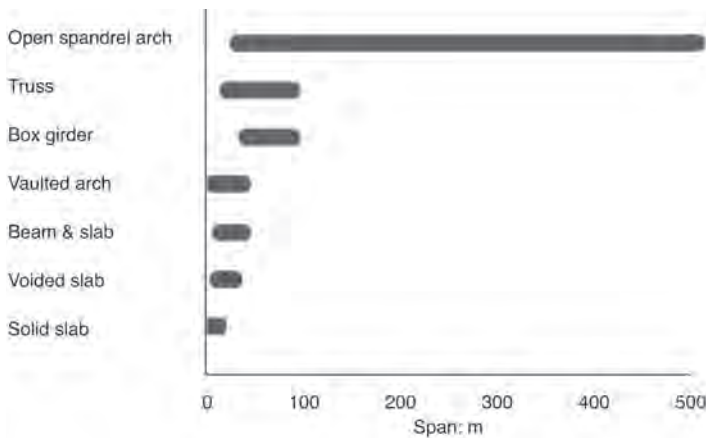


Figure 8.8 Typical span range for various bridge types

Square voids have been used but tend to give problems with air voids and are not recommended. Polystyrene is not impermeable; the voids should therefore be provided with drainage holes at their lower ends. It is important to ensure that the voids and reinforcement are held firmly in position in the formwork during construction to avoid voids floating.

Shear stress (in the webs between voids) is likely to become excessive near supports; stopping the voids and having a solid section where shears are high is a common solution. Transversely, such a structure behaves quite differently under uniform and non-uniform bending. In the former, the top and bottom flanges act compositely, whereas in the latter they flex about their separate neutral axes. This means that the flexural inertia can be an order of magnitude greater for uniform than non-uniform bending. The behaviour can, however, be modelled using a shear deformable grillage (Hambly, 1991).

Spine beams

Above a span of about 10–12m, the dead weight of a solid slab bridge starts to become excessive. For narrower bridges, significant weight savings can be achieved by using relatively long transverse cantilevers, giving a bridge of spine beam form. Spine beams are a form of solid slab where the full depth slab covers only part of the bridge width; a thinner slab cantilevers from the spine. This form of structure is more efficient. The width of the spine is governed by the cantilever slab (typically 3 m maximum), the shear capacity of the spine at the supports or the width needed to place the flexural reinforcement without congestion (not closer than 125 mm centres and no more than two layers of reinforcement). This can extend the economic span range of this type of structure to around 16m or more; above this span, a lighter form of construction is desirable.

Beam-and-slab

Beam-and-slab bridges allow the optimisation of beams spanning longitudinally and thinner slabs spanning transversely. The optimum layout minimises the number of beams, with a

spacing of at least 3 m. The number of beams is again determined by shear and detailing requirements, as for the spine beam type. Because the deck slab forms a large top flange, the beams of beam-and-slab bridges are more efficiently shaped to resist sagging rather than hogging moments. It may therefore be advantageous to haunch them locally over piers, even for relatively short-span continuous bridges. At spans above 30 m, there may be insufficient flexural compression in the beam over continuous supports and box girders may be a better option. One of the disadvantages of a beam-and-slab structure, compared with a solid slab, is that distribution properties are relatively poor.

Box girders

Box girder bridges are often haunched at supports (Figure 8.9) and can span to almost 100 m. A box girder could be considered as a large voided slab. Access into box voids is required for inspection and maintenance, so a minimum box depth of about 1.8 m is required. Formwork can only be removed if the section is deep enough for access, which effectively means around 1.2 m minimum depth. Permanent access to the voids is often provided. In older structures, this was often through access holes in the top slab. This also means that traffic management is required to gain access and that there are problems of water leaking into the voids. It is, therefore, preferable to provide access from below.



Figure 8.9 Waterloo Bridge: a haunched reinforced concrete box girder

Arches

The longest spans (over 500 m) and most dramatic purely reinforced concrete bridges are open spandrel arches (see Figures 8.2, 8.8 and 8.17). The arch form suits reinforced concrete well, as the compressive force in the arch ribs increases its flexural strength (see Figure 8.14). As a result, the form is quite efficient in terms of materials. It is most suitable for crossing valleys in hilly country where ground conditions are good (to resist the arch horizontal thrust).

The simplest way to build an arch structure is on falsework. However, the falsework required is very extensive and hence



Figure 8.10 Construction of an arch rib in cantilever segments using temporary cable stays

relatively expensive. Because of this, arch bridges are often more expensive than structurally less efficient forms, such as prestressed cantilever bridges that can be built with fewer temporary works. It is possible to devise other ways of building arches. They have been built out in segments from either end, supported by tying them back with temporary stays (Figure 8.10).

The efficiency of arch structures arises because the shape is optimised for resisting the near uniform forces arising from dead load, which is the dominant load. The profile of the arch is arranged to minimise the bending moments in it. Theoretically, the optimum shape approximates to a catenary, if the weight of the rib dominates, or a parabola, if the weight of the deck dominates, but the exact shape is unlikely to be critical. Because the geometry is optimised for a uniform load, loading the entire span is unlikely to be the critical live load case, unlike simple single-span beam bridges. It will normally be necessary to plot influence lines to determine the critical case. For uniform



Figure 8.11 Multiple vaulted arch using precast reinforced concrete units

loads, this is often loading half a span.

Arch bridges have been built in which the live load bending moments are taken entirely by the girders at deck level, enabling the arch ribs to be very slim in appearance. Deep arches with very thin decks have also been built. However, the more usual approach is to build the arch ribs first and then build the deck structure afterwards, the deck and arch structures having similar flexural stiffness and thickness.

Solid arch vaults or barrels with soil backfill or with spandrel walls are economic and fast to build if precast (Figure 8.11); they are becoming popular and are particularly suited for use as green bridges, as they have relatively low material content and carbon footprint. Arch vaults span from 4 to 40 m.

Trusses

In the past, reinforced concrete truss structures with spans up to 90 m have also been built but they are not often used in modern practice because the building costs are high, owing to the complexity of the formwork and reinforcement.

Materials and analysis

Materials

The behaviour of a reinforced concrete structure is heavily influenced by the properties of its component materials. A brief summary of the key properties is given in Table 8.1. In particular, the reader should note that concrete has a good compressive strength and a small tensile strength, while steel reinforcement has a high tensile strength. It is the exploitation of these different properties that makes reinforced concrete construction economic.

Material	Strength: N/mm ²	Elastic modulus, E: kN/mm ²	Other properties
Concrete	$f_c = 30\text{--}60$ $f_{ct} = 3\text{--}5$	25–35	Creep and shrinkage effectively reduce the elastic modulus over time.
Steel	$f_y = 250\text{--}500$ Similar in tension and compression	210	Tensile range may be affected by fatigue and compression range by buckling. Corroded by air, water and chlorides if exposed.
GFRP	$f_u = 2000$	80	Glass, basalt, aramid and carbon fibre reinforced polymers are resistant to chloride corrosion. Breaking strain of polymers, 1%–2.5%.
BFRP	$f_u = 3300$	85	
AFRP	$f_u = 2700$	120	
CFRP	$f_u = 3700$	220	

Table 8.1 Comparison of material properties

Analysis

For reinforced concrete structures, conventional beam, plate, grillage or finite-element analysis can be conducted, as with other bridge types (see Chapter 4). However, for an accurate assessment of the internal forces within the bridge, the effects of concrete cracking must be taken into account. It is permissible to use uncracked section properties for reinforced concrete bridges, but this will give a conservative estimate of moments, shears, and so on, and is likely to overestimate crack widths and underestimate deflections and rotations. A fully non-linear finite-element analysis will tend to give a more accurate analysis provided tests on similar structures are available to validate the analysis and the various material parameters needed.

A three-phase approach to the analysis of reinforced concrete bridges (Collings and Sagaseta, 2019) will provide a better approximation to the behaviours at serviceability and ultimate limit states. Where the characteristic load on the bridge does not crack the concrete over significant areas, or the maximum is less than approximately twice the cracking moment, or the maximum vehicle load is a once-only load, uncracked section properties may be appropriate. This is usual for beams and slabs with low span-to-depth ratios. Where span-to-depth ratios are very low (less than 6), a strut and tie analysis is likely to be more appropriate. Where the characteristic load causes cracking and maximum moments are significantly greater than the cracking resistance, or the characteristic loads will occur a number of times over the life of the bridge, consideration of cracking with tension stiffening is appropriate for estimation of section properties. The stiffness of the section may be estimated using

$$EI_{\text{eff}} = (1 - \zeta)EI_c + \zeta EI_y \quad (8.1)$$

where EI_c is the initial uncracked stiffness, EI_y is the fully cracked stiffness at higher strains, β is a coefficient depending on the loading, which is approximately 1.0 for a single loading or 0.5 for a number of loadings on the structure (such as traffic loads), and

$$\zeta = 1 - \beta(M_{cr}/M_E)^2 \quad (8.2)$$

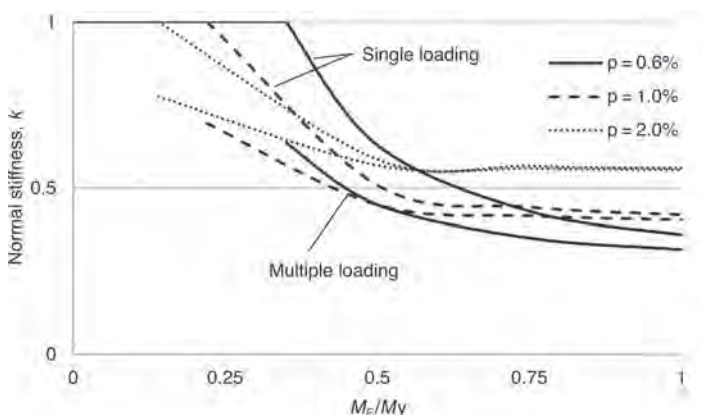


Figure 8.12 Stiffness of cracked reinforced concrete slab

Figure 8.12 outlines the results of Equations 8.1 and 8.2 for different reinforcement contents and loadings. The flexural, torsional and axial stiffnesses of the concrete section will be affected by cracking. The cracking of reinforced concrete structures is likely to lead to some redistribution of moments and forces, even at the serviceability limit state (Pisanty and Regan, 1998). Cracking of concrete will also lead to some dilation, which may lead to restraint forces or additional movements at supports.

Concept design

In the conceptual design of a reinforced concrete bridge, the designer must choose the type of structure to suit the site, brief, cost budget, carbon footprint, environmental considerations and any aesthetic requirements. The shape of a reinforced concrete bridge is usually decided by experience, aided by typical span-to-depth ratios (Table 8.2). There was a fashion for very shallow bridges in the 1960s and 1970s, as they were considered to look more elegant. However, unless increasing the construction depth has major cost implications elsewhere, it is likely to be more economic to use more than the minimum depth. The appearance disadvantage can be resolved by good detailing of the edges. Bridges with short transverse cantilevers at the edges tend to look shallower than vertical sided bridges, even if actually deeper. One of the first considerations is the structural type and span. Figure 8.8 gives an indication of the span ranges for various reinforced concrete structures. Structural forms, such as arches, are usually the most efficient and aesthetic types but require good foundation conditions or the benefits will be offset by additional foundation costs. Integral solid slab or spine beams decks are the next choice, with haunching of continuous spans.

Structure	Span-to-depth or span-to-rise ratio
Simply supported beam or slab	14–18 Dependent on reinforcement content, with lower L/h ratios for more heavily reinforced structures.
Continuous beam or slab	18–25
Continuous beam or slab with haunches	20–33 at midspan 16–20 at supports
Cantilevers	6–10
Arches	5–14
Trusses	9–16

Table 8.2 Typical span-to-depth ratios

It is important to get the correct geometry for a bridge; skews greater than 30° should be avoided, as they will make integral bridges difficult and so drive up costs and lead to long-term durability problems at joints and bearings.

Span-to-depth ratios of slabs, beams and arches are noted

in Table 8.2. There is some leeway with these ratios; however, when span-to-depth ratios become larger than recommended, deflections, ductility and reinforcement content may become critical issues to resolve.

The efficiency of a design may be compared with those of other bridges using an effective thickness concept. The effective thickness of a concrete bridge deck is the volume of material in the superstructure divided by the bridge deck area. The effective thicknesses of actual bridges and theoretical limits are shown in Figure 8.13. Bridges near the lower limit of effective thickness are often more economic and have a lower carbon footprint, as material content is a significant influence on these parameters.

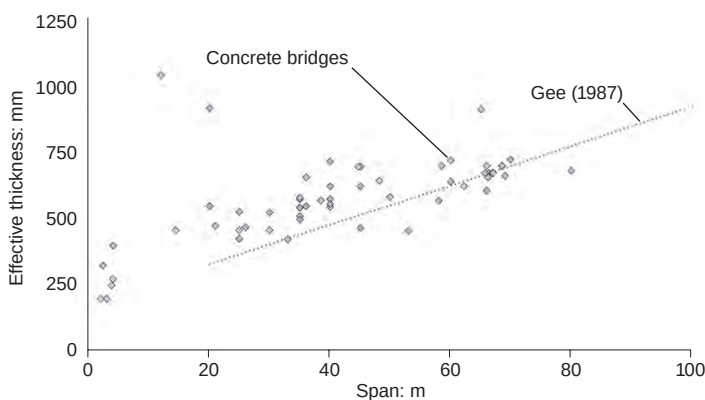


Figure 8.13 Effective thicknesses of reinforced concrete bridges

Detailed design

The detailed design of a reinforced concrete bridge will require calculations of the ultimate limit state effects to ensure that the structure has sufficient strength and is robust; checks to ensure that progressive collapse does not occur may need to be made for bridges with a number of tension elements or where precast elements are used. A detailed design will require the checking of crack widths, deflections and, in some cases, frequencies and accelerations at the serviceability limit state. The long-term durability of the structure will need to be assessed. Fatigue assessments may be required for some railway bridges. Detailing of the concrete and reinforcement within the concrete will be required.

Ultimate limit state

At the ULS, calculations of the compression or tensile capacity of parts of the structure, such as columns, walls, struts and slabs, will be made. For concrete sections with a span-to-height ratio less than 20, where slenderness and buckling effects are small, the compression capacity can be estimated using

$$N_{uc} = 0.56f_{ck}A_c + 0.7f_yA_s \quad (8.3)$$

The tension capacity can be calculated using

$$N_{ut} = 0.87f_yA_s \quad (8.4)$$

Note that any tensile capacity of the concrete is ignored in this equation.

Structures with tensile elements are more likely to be prone to progressive collapse if elements fail. Consideration of the failure of one or more tensile elements must be considered in design. For both tension and compression elements, additional moments from the eccentricity of load and from the effects of tolerance on straightness must also be considered.

Calculations of the flexural capacity will be made. For a robust and ductile structure, the flexural capacity should be governed by the tensile yielding of the reinforcement

$$M_{ut} = 0.87f_yA_s(d - x/2) \quad (8.5)$$

and not the compression capacity

$$M_{uc} = 0.8bxf_{ck}(d - x/2) \quad (8.6)$$

which may result in a brittle failure mechanism.

For most bridges, there are likely to be both flexural and in-plane forces. The interaction of flexure and axial forces is best considered by assuming the strains across a section to derive an axial–flexure interaction diagram. Figure 8.14 is an interaction diagram for a rectangular reinforced concrete section with symmetrical reinforcement. There is an increase in flexural capacity for most reinforced concrete sections under modest axial compressions, the maximum moment being carried at 25% to 30% of the maximum axial capacity. There is always a reduction in flexural capacity with axial tensions.

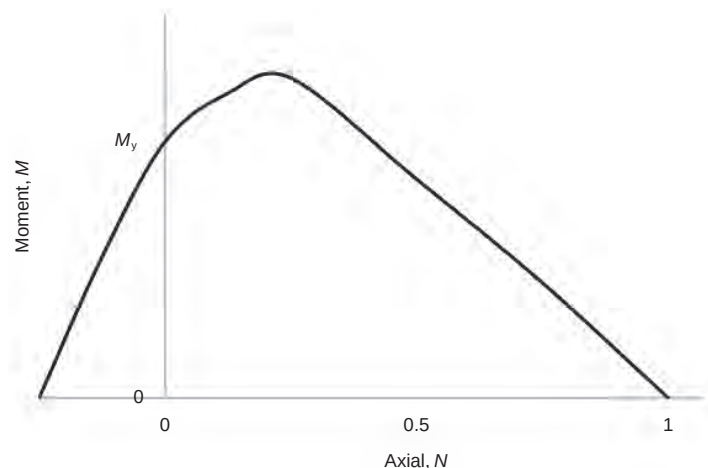


Figure 8.14 Axial–flexure (N–M) interaction diagram

Calculations of the shear capacity will be made at the ultimate limit state; critical sections are normally near supports. For solid slabs, it is normal to avoid using shear reinforcement, and the shear capacity of the slab is a function of the concrete strength and the amount of flexural reinforcement (ρ), as given by

$$V_c = 0.15kbd(100\rho f_{ck})^{0.33} \quad (8.7)$$

$$k = 1 + (200/d)^{0.5} \quad \text{but not more than } 2 \quad (8.8)$$

It should be noted that, for shear, there are size effects (k); deeper sections will have relatively lower shear capacity than smaller sections.

Where there are both shear and in-plane axial forces, there will be an interaction of the two. Figure 8.15 is an interaction diagram for a rectangular reinforced concrete section with symmetrical reinforcement. There is an increase in shear capacity for small compressions (up to $0.2N$) and a reduction in shear capacity for coexistent tensions.

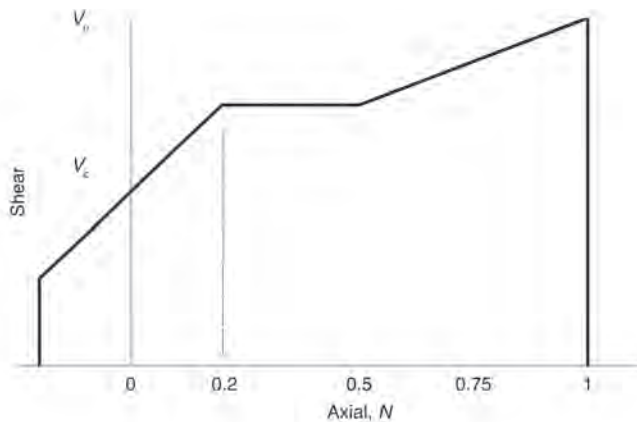


Figure 8.15 Axial-shear (N - V) interaction diagram

At high axial loads, if the section is uncracked in bending, the maximum shear stress may be determined using a principle stress approach, as with prestressed concrete; however, the maximum shear stress should not be greater than

$$V_u = 0.15bd f_{ck} \quad (8.9)$$

For voided slabs or beams in beam-and-slab decks, shear links will be required. The shear capacity of the section with links at spacing s is

$$V_l = 0.75A_s f_y / s \quad (8.10)$$

It should be noted that the width of the web or the width between voids, not the full slab width, must be used.

Equation 8.10 assumes a 45° truss analogy; an alternative variable truss angle method can be used. A flatter truss angle enables a greater shear force to be carried by the same links but may require more longitudinal reinforcement. The evidence indicates that the varying angle truss approach gives a more realistic representation of behaviour and a more consistent overall safety margin. Concrete truss elements at angles of 30° – 40° are appropriate. It should be noted that links will only be effective if the concrete has cracked in shear; it is important to limit the size of shear cracks (Muttoni and Fernandez-Ruiz, 2008).

A check on punching shear at pier or bearing supports or from concentrated wheel loads is required. The distance of the assumed failure plane varies from $0.5d$ to $2d$ from the support, depending on the code being used. The distance usually has no

physical significance; it is the distance at which the empirical limiting shear matches that of tests. The failure cone (Figure 8.16) extends from $0.5d$ to $3d$ from the load. For Eurocode 2 (BSI, 2005), the punching load capacity (P_{cl}) at a shear plane $2d$ from the load or support is given by

$$P_{cl} = 0.15k b_p d (100 \rho f_{ck})^{0.33} + 0.2 \sigma_c b_p d \quad (8.11)$$

where b_p is the shear perimeter length; σ_c is the in-plane stress across the shear plane and may increase or reduce the punching capacity in a similar way to the interaction of shear and axial loads shown in Figure 8.15. Experimental evidence indicates that most deck slabs that have some local restraint and compressive membrane action (Collings and Sagaseta, 2015) will provide this in-plane stress.

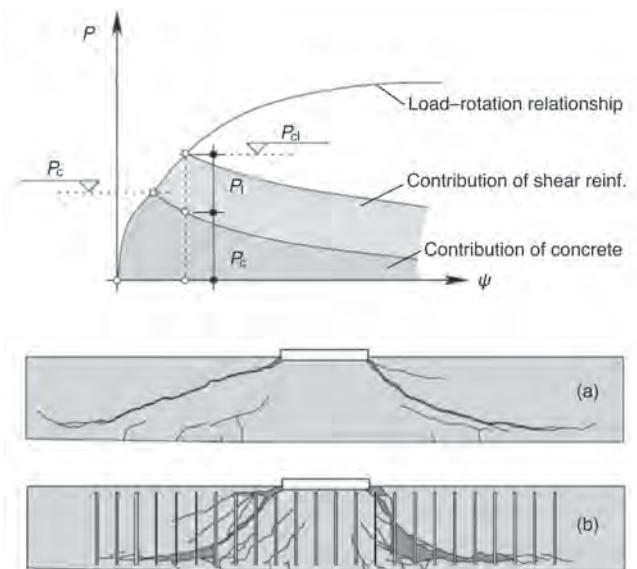


Figure 8.16 Punching shear failure, load-rotation curve and failure (a) without and (b) with links

Serviceability limit state

At the serviceability limit state, the stresses in the section should be limited. The stress in the reinforcement should be less than $0.8f_y$ to ensure that the structure remains linear in behaviour. The stress in the concrete should be less than $0.45f_{ck}$ to avoid longitudinal cracking, significant creep effects and non-linear material behaviour.

To limit crack widths such that they do not affect durability or cause unacceptable appearance, the stress in the reinforcement should be limited; the stress depends on the bar size, design cover and the reinforcement spacing, as given in Table 8.3. For bridges, it is normal to use a 0.3 mm crack width limit. A 0.2 mm limit may be specified in some circumstances around joints and water drains or downpipes, which may leak. It is possible to calculate crack widths; however, it should be noted that such calculations are approximate.

Steel stress: MPa	0.2 mm		0.3 mm	
	Maximum bar size	Maximum bar spacing	Maximum bar size	Maximum bar spacing
160	25	200	32	200
200	16	150	25	200
240	12	100	16	200
280	–	–	12	150
320	–	–	10	100

Table 8.3 Stresses to limit crack widths

A particular problem with cracking can arise with reinforced concrete that is constrained when first cast. The concrete heats up as the cement hydrates and tends to crack as it cools. In practice, early thermal effects are rarely critical in bridge decks, except for second cast items, such as parapet edge beams; they may also be critical in long walls. Reduced cement content, cement replacement and the use of minimum reinforcement content are usually sufficient to control these cracks.

For highway bridges, there are no specific deflection limits; bridges with span-to-depth ratios given in Table 8.2 should have deflections that will not adversely affect function or appearance. However, an estimate of the long-term deflection of the bridge should be made as, for all but the smallest slabs, it will be necessary to provide a precamber to the structure to offset these deflections. There are live load deflection limits of span/1000 for some railway bridges.

For footbridges and some railway bridges, the resonant frequency of the structure may be important to meet the comfort criteria for users. Typically, footbridges with frequencies greater than 4 Hz will not require detailed dynamic analysis. Typically, railway bridges with frequencies (f_z) up to those given by

$$f_z = 45/(\text{span})^{0.75} \quad (8.12)$$

will be satisfactory. High-speed railway bridges with train speeds above 200 km/h may require dynamic analysis; bridges of 10–50 m span with frequencies above f_z and below $2f_z$ are likely to be satisfactory and only require an assessment of resonance with respect to the train bogie spacing and speed.

Durability

Durability is one of the most important design criteria, as it will determine the lifespan of the bridge. The Wey Bridge of Figure 8.1 was built in 1913; it was widened and another bridge built directly alongside it, and it had major repair and reconstruction work in 1990. It is currently in good condition and likely to have a lifespan exceeding 120 years. The Salginatobel Bridge of Figure 8.2 was completed in 1930; it was extensively repaired, and waterproofing added in 1976. However, deterioration continued and in the 1990s the waterproofing and drainage were replaced, the parapets rebuilt and parts of

the exposed concrete shotcreted to improve cover to the reinforcement. Owing to the importance of the structure, it is likely that it will continue to be maintained and achieve a lifespan of 100 years or more. The Mizen Head Bridge was built in 1910; by the 1990s, deterioration of structural elements was clear (Ruane and Healy, 2004). Despite repair and rehabilitation, by 2005 concern over the structural integrity of the vertical tensile hangers meant that the bridge was closed to the public, giving a 95 year lifespan. The bridge was demolished in 2009 and a replica bridge (Figure 8.17) constructed with modern concrete, steel and cover requirements (Ruane *et al.*, 2013).



Figure 8.17 Mizen Head Bridge, a new bridge replicating the original structure

In practice, the main issues causing durability problems are inadequate cover, lack of waterproofing, water ingress at joints, poor drainage details and the use of de-icing agents. Codes give cover and crack width limits and minimum concrete quality. Drainage is particularly important; if it is poor and leads to puddles, the evaporation of water leaves salt (chlorides) behind. The difficulty of ensuring good drainage at joints is one reason for the use of integral and continuous bridges. Waterproofing to the top surface of the deck and around concrete at joints should always be provided.

The exposure conditions of the structure will affect its durability. Structures in or near the sea, where water contains chlorides, will require additional corrosion protection, as will foundations in aggressive ground water. For exposed structures, additional cover, lower-porosity concrete, air entrainment to the concrete, surface coating to the concrete or non-corrodible reinforcement may be required.

Detailing

Having designed a reinforced concrete bridge and produced drawings showing the concrete shape and size (Figures 8.18 and 8.19), the reinforcement within the concrete must be detailed to fit inside it. The reinforcement size, spacing, lapping and anchorage and scheduling are undertaken. The details in this section are based on Eurocode 2 requirements (BSI, 2005).

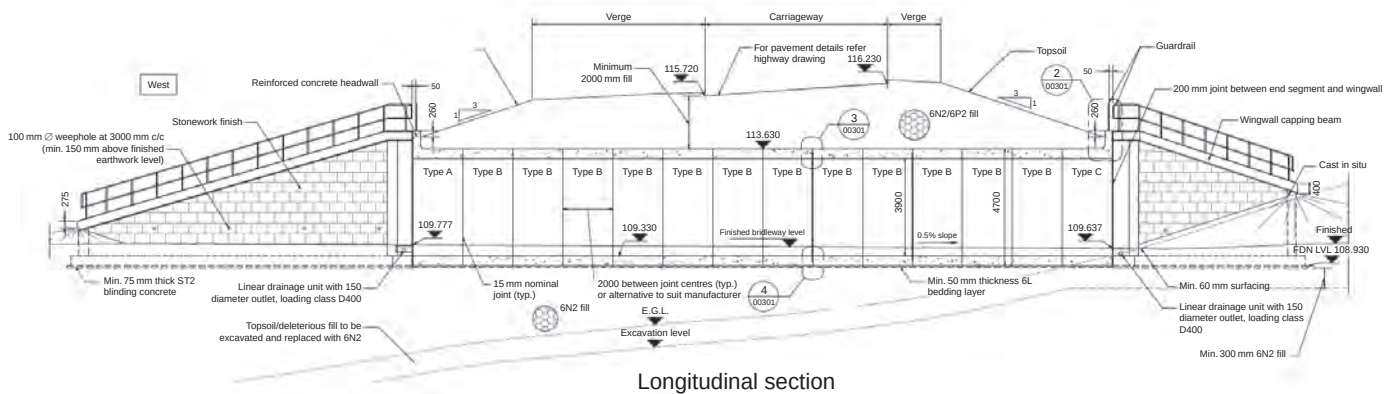


Figure 8.18 Long section of a reinforced concrete buried underpass, typical of a detail drawing

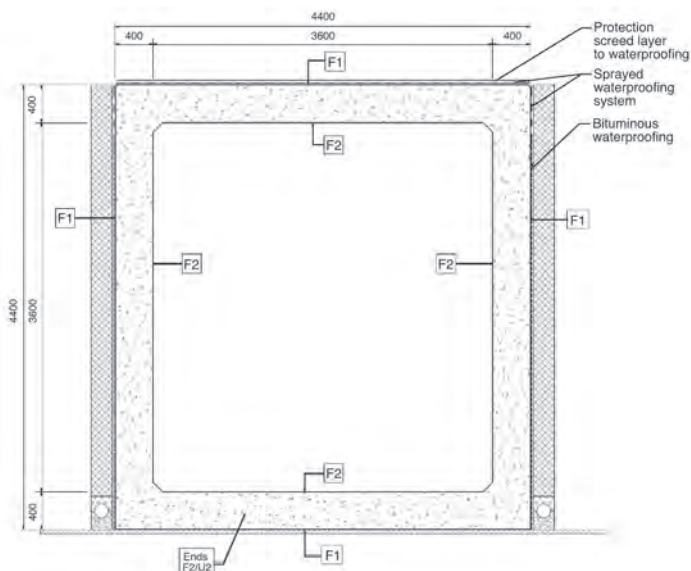


Figure 8.19 Cross-section of a reinforced concrete buried underpass, typical of a detail drawing, showing finishing required by the specification

Minimum and maximum reinforcement

For reinforced concrete, a minimum area of reinforcement in any one face in one direction is defined as

$$A_{s\min_1} = 0.0025bh \quad (8.13)$$

Below this minimum area, the section is classed as plain concrete and assumed to be unreinforced.

For beams and slabs, the minimum flexural reinforcement in one face and one direction should have a capacity greater than the flexural capacity of the plain concrete section, such that if it cracks the reinforcement does not yield:

$$M_{cr}=0.16bh^2f_{ct}<M_{ult}=0.75dA_s f_y \quad (8.14)$$

The minimum area is normally taken as

$$A_{s\min.} = 0.003bd \quad (8.15)$$

The maximum reinforcement in a section will be governed by preventing brittle failure of the concrete section, such that $M_{ut} > M_{uc}$ (Equations 8.5 and 8.6), giving

$$A_{s \max_1} = 0.4bd(f_{ck}/f_y) \quad (8.16)$$

with the absolute maximum as

$$A_{s\max} = 0.004bh \quad (8.17)$$

For beams and slabs where shear reinforcement is required, the minimum area of the shear links is given by

$$A_{sv\min} = 0.08bs(f_{ck})^{0.5}/f_y \quad (8.18)$$

As the minimum value of s is $0.75d$ for conventional concrete and reinforcement strengths

$$A_{\text{symin}} = 0.0012bd \quad (8.19)$$

may be used.

For columns and arches with primarily axial loads, the minimum total reinforcement in all faces in one direction is

$$A_{\text{scmin}_1} = 0.12 \sigma_{\text{cf}_v} \quad (8.20)$$

For concrete stresses $\sigma_c < 8.5$ MPa

$$A_{\text{sc min},0} = 0.002bh \quad (8.21)$$

is the minimum:

The maximum area of reinforcement in a column or arch is

$$A_{\text{scmax}} = 0.04bh \quad (8.22)$$

Anchorage and laps

Where reinforced concrete decks and integral abutment walls intersect or at deck ends beyond bearings, the reinforcement should be anchored to avoid pull-out. The anchorage of longitudinal reinforcement is typically about 20 reinforcement bar diameters (20ϕ , where ϕ is the reinforcement bar diameter). The anchorage can be calculated; it is a function of the bond strength between the reinforcement and concrete (f_{bd}), the local

stress field and confinement (α_{235}) in the concrete, typically taken as 0.7, and the stress in the reinforcement (σ_s), as

$$l_{\text{bd}} = 0.25 \alpha_{235} \Phi \sigma_s / f_{\text{bd}} \quad (8.23)$$

$$f_{\text{bd}} = 2.25 f_{\text{ct}} \eta_1 \eta_2 \quad (8.24)$$

where η_1 is a bond coefficient, normally 1.0 for good bond conditions and 0.7 in poor bond conditions near the tops of slabs thicker than 250 mm, and η_2 is a bond coefficient that depends on the bar size relative to the cover. The anchorage of shear links should normally be by bends or hooks around the main longitudinal bars and extending 5ϕ to 10ϕ beyond it.

For most reinforced concrete structures, there will be a need to lap bars and transfer stresses from one bar to another. The laps are typically 40ϕ to 50ϕ . The lap length can be calculated in a similar way to the anchorage, as

$$l_{\text{bd}} = 0.25 a_{236} \Phi \sigma_s / f_{\text{bd}} \quad (8.25)$$

where α_{236} depends on confinement and the amount of reinforcement lapped in one area; this coefficient is 1.0 if less than 25% of reinforcement is lapped, 1.4 for 50% and 1.5 if 100% is lapped in one area.

Reinforcement detailing

Having designed the main flexural and shear reinforcement, determined the likely spacing, determined minimum reinforcement requirements in other places, calculated the concrete cover, determined where laps occur and estimated the lap lengths, the reinforcement can be detailed to fit within the concrete outline. Traditionally, this is achieved using a two-dimensional drawing showing elevations and sections of the reinforcement layout (see Figures 8.20 and 8.21), with the bars numbered and called up in a schedule, which can be used to determine the necessary reinforcement bends for construction. Today the process is often carried out within a three-dimensional drawing BIM (building information model) with the dimensions and weights automatically calculated.

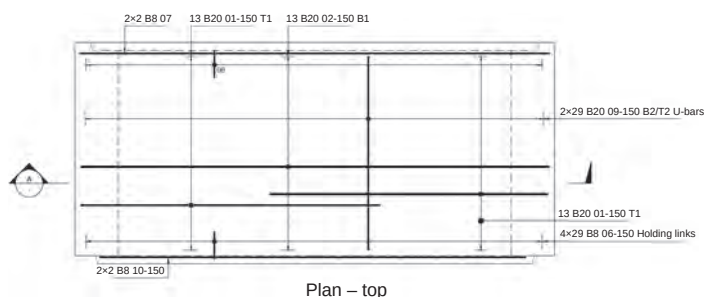


Figure 8.20 Typical reinforcement detail of slab

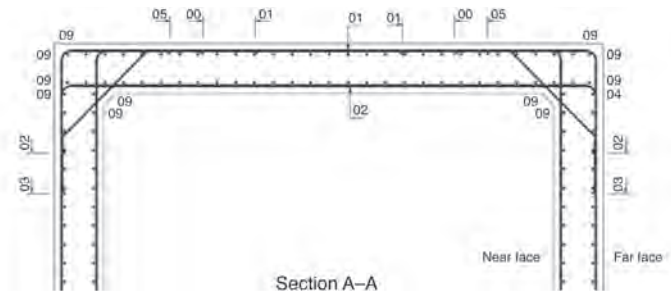


Figure 8.21 Typical reinforcement section of slab, showing bar locations

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Chapter 9

Prestressed concrete bridges

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This chapter provides coverage on all aspects of the design and construction of prestressed concrete bridge decks, from the fundamentals of prestressing and the equipment used, to insight into the problems that can occur and how they have been dealt with in different projects around the world. It explains key issues for the designer to consider and provides guidance to the site engineer on some of the techniques and temporary works used in their construction. Simple in situ and precast beam bridges are described, along with the more advanced techniques of precast segmental and incrementally launched decks. Both pre- and post-tensioning are included, as well as precast and in situ construction. Recent developments in the use of non-metallic materials to improve the durability of prestressing systems and higher-strength concrete are included to show how prestressed concrete bridges are likely to develop in the future.

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Introduction

Prestressed concrete decks constitute a large proportion of the current bridge stock around the world. Since prestressing was introduced to concrete bridges in the early 1930s, it has challenged the bridge engineer's imagination, as new techniques developed and spans became longer and the appearance of bridges more important. As the need for sustainability and resilience have become more important in society, prestressed concrete bridges have shown that they have a vital role to play in today's infrastructure construction, whether it be in an urban setting (Figure 9.1) or a river crossing (Figure 9.2), for a highway or a railway, for short spans and long spans.

Concrete is strong in compression but weak in tension; however, concrete can be prestressed to ensure that it remains within its tensile and compressive capacity. Prestressing is applied as

an external force to the concrete, using wires, strands or bars, and can increase the capacity of concrete alone many times. In recent years, higher-strength concrete, fibre-reinforced concrete, self-compacting concrete and the continued development of prestressing systems and materials have all helped in pushing the boundaries in the design and construction of prestressed concrete bridges.

The development of prestressed concrete has given the bridge engineer increased flexibility in the selection of bridge form and in the construction techniques available, resulting in prestressed concrete frequently being the material of choice for bridges with spans ranging from 25 m, with precast beams, up to 530 m, when cable-supported. In situ or precast concrete; simply supported, continuous or cable-stayed structures; beams, slabs or boxes – all feature in prestressed concrete bridges.



Figure 9.1 Urban flyover



Figure 9.2 Pont Briwet

This chapter presents the design and construction of the different types of prestressed concrete bridges, along with the materials and equipment used and assumes that the reader has a basic understanding of prestressed concrete, which can be applied to the specific application of bridgeworks.

Prestressed bridge types

Deck form

Many factors affect the choice of deck type, span arrangement and general layout, while prestressed concrete decks cover a wide range of construction forms, with span lengths ranging from 25 m or less for single spans to over 500 m in cable-stayed bridges. For spans below 25 m, reinforced concrete is likely to be preferred, while for spans above 500 m steel or composite cable-stayed and suspended decks are used; however, between these span lengths, prestressed concrete often gives an economic, aesthetic and simple solution.

The choice of deck form depends largely on the individual site constraints, project requirements and designer's preference. The advantages and disadvantages of the different options need to be carefully considered to arrive at the optimum solution. Figure 9.3 indicates the typical span ranges adopted for different deck types.

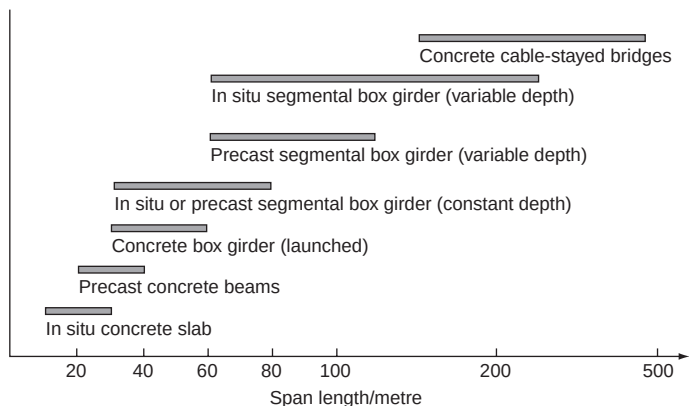


Figure 9.3 Typical span range for different deck types

For bridges with an overall length of 60 m or less, it is common to build the deck continuous and integral with the piers and abutments to reduce the number of expansion joints and bearings and eliminate the maintenance problems that can occur with these elements. For longer structures, the deck should still be made continuous for as long as practical, depending on the structural form, with concrete box girders having been constructed up to 1.7 km between expansion joints. The need for bearings between the deck and substructure depends on the deck type and stiffness of the substructure. Precast segmental construction often utilises bearings to simplify erection, while in situ deck construction can either have bearings or easily be built into the substructure, provided that deck movements do not generate excessive forces in the piers and foundations.

Span arrangements are frequently governed by the nature of the obstruction being bridged but, if possible, the spans should be arranged to suit the type of bridge being constructed. With precast beams, it is preferred to keep a standard beam length throughout to standardise casting and erection equipment, while with in situ, precast segmental or launched continuous decks, the end spans should be arranged to be approximately 70% of the length of the internal spans to balance out the design bending moments. Where the deck is built by balanced cantilever, the end spans should be reduced to approximately 60% of the length of the internal spans to minimise any out-of-balance effects of the end balanced cantilever, although care should be taken to ensure that no uplift occurs at the abutment bearings, otherwise a tie-down arrangement is needed.

Solid-slab bridges

In situ, solid-slab decks are often used for shorter concrete spans where easy access is available, with the advantages including simple construction and formwork layout, while the disadvantages are that prestressing a solid concrete element is inefficient and dead load becomes excessive as spans get longer. For these reasons, short-span solid-slab decks are usually formed from simple reinforced concrete; prestressing them is not common for bridgeworks.

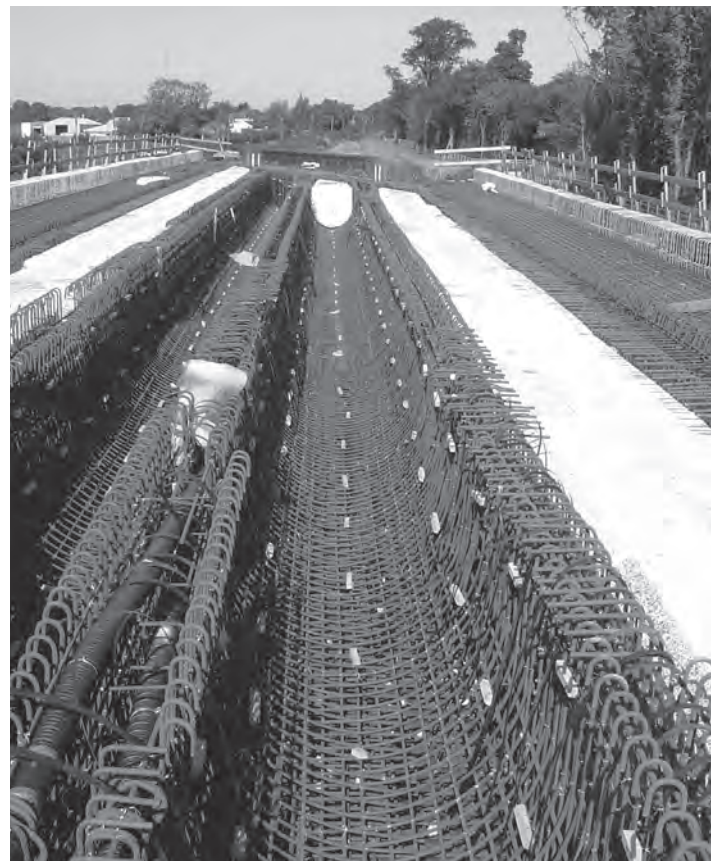


Figure 9.4 Voided slab deck under construction

Voided slab bridges

In situ voided slab decks (Figure 9.4) were commonly used during the early period of prestressed concrete bridge development within the span range 25–35 m and depth/span ratios of up to 1:20. The voids are often shaped from polystyrene void forms, either round (Figure 9.5) or rectangular with bevelled corners, leaving a 150 mm thick concrete section above and below. Plastic pipes have also been used to form the voids. The concrete is usually cast supported by falsework from the ground. The advantages include simple construction and formwork arrangement, while the disadvantages include a heavy deck section to prestress and the need to hold down the voids during concreting; this has more recently resulted in beam-and-slab decks or multi-cell boxes being used in preference.

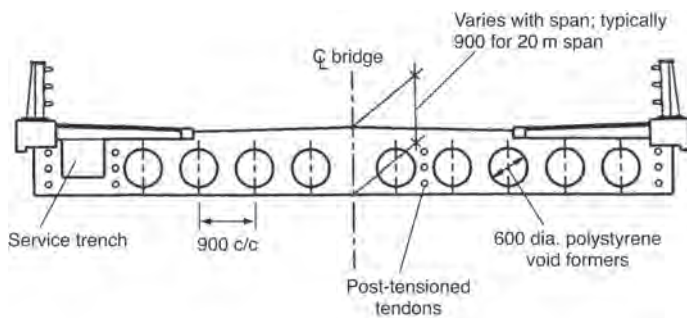


Figure 9.5 Typical voided deck slab section

Beam-and-slab bridges

Beam-and-slab decks (Figure 9.6) utilise either precast or in situ concrete beams, with the deck slab cast in situ and a range of different beam types available (Figure 9.7). Spans normally range from 25 m up to 40 m, although precast beams have been used on spans up to 50 m. Depth/span ratios are usually of the order of 1:16, with either post-tensioned or pre-tensioned strand positioned to take the sagging moments, as indicated in Figure 9.8.



Figure 9.6 Beam-and-slab deck under construction

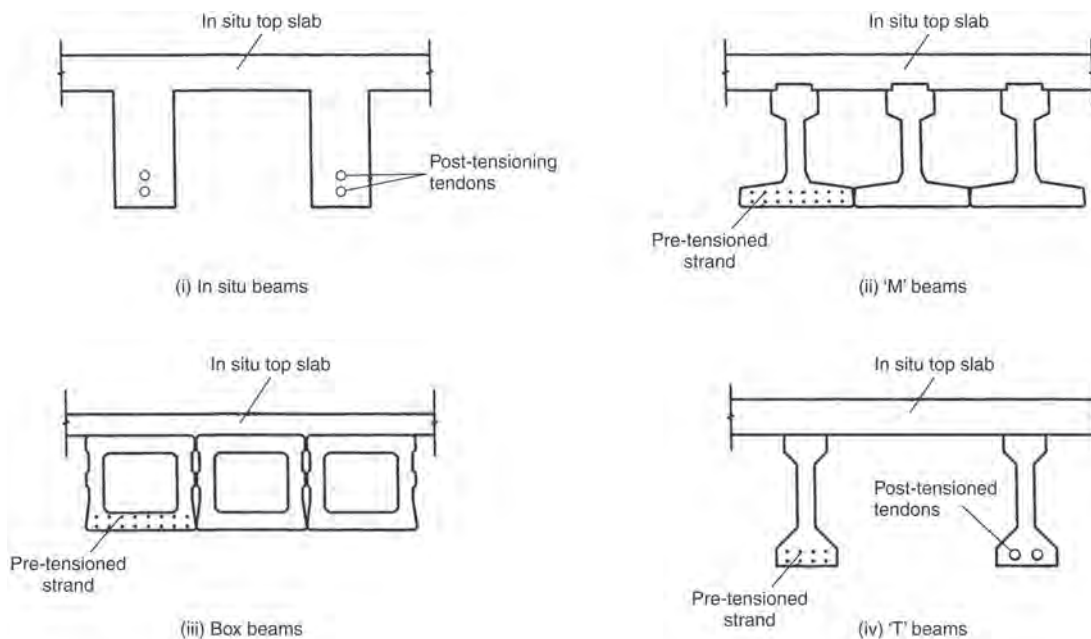


Figure 9.7 Typical prestressed beam-and-slab arrangements

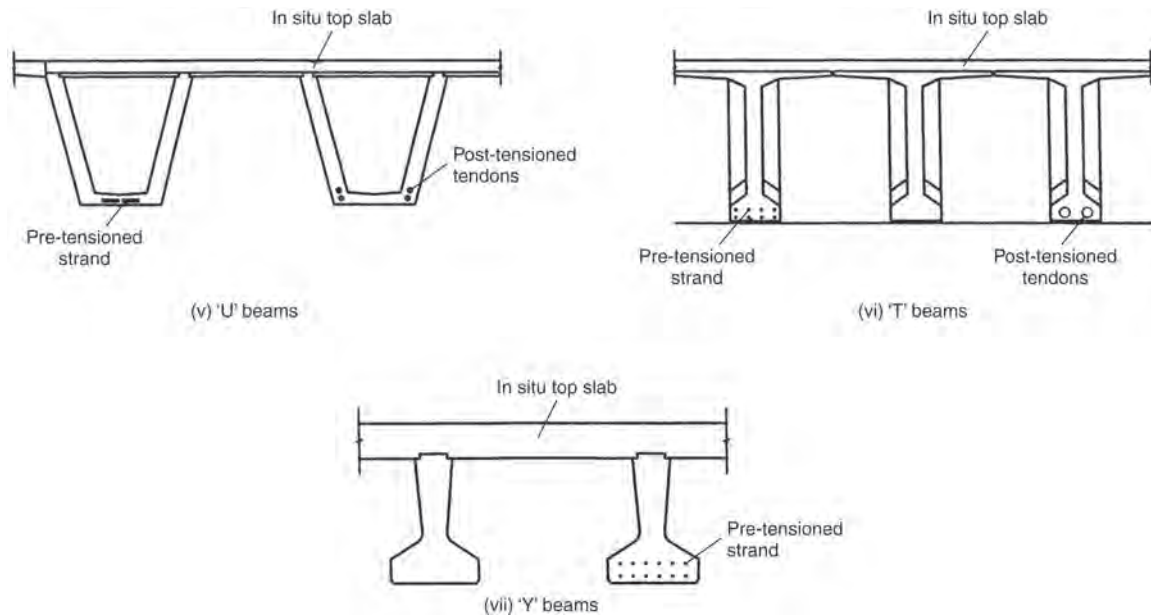


Figure 9.7 Continued

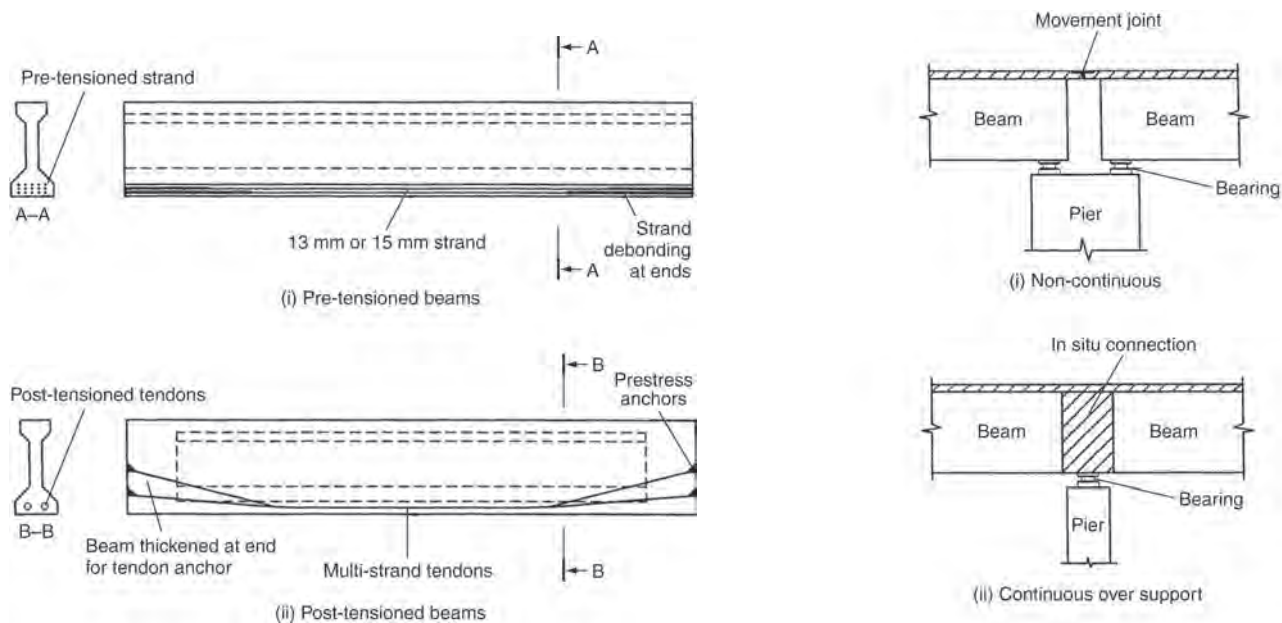


Figure 9.8 Typical prestress layout in beams

Figure 9.9 shows some typical arrangements for the beams at piers, where the beams may be connected or the top slab made continuous over the piers to minimise the need for expansion joints in the road surface. The beams in any one span are normally connected together at the support location by a transverse in situ concrete beam, or diaphragm, which provides rigidity to the arrangement and can be used to transfer load from the beams to the supports.

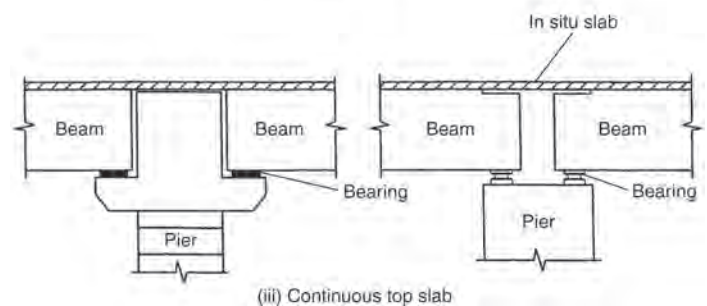


Figure 9.9 Beam continuity

Where the beam-and-slab deck is cast in situ, traditional formwork and falsework arrangements may be used to support the concrete while it hardens. Post-tensioning tendons are then threaded through the ducts cast into the concrete and stressed, lifting the deck off the formwork and allowing the falsework and formwork to be removed.

The advantages of precast beams include economies in repetitive precasting and rapid construction, while the disadvantages include the need for good access for transportation and positioning of the beams and heavy lifting equipment for the longer spans.

Precast beams are usually made in special casting yards, which have a series of casting beds to form the beams. For pre-tensioned beams, jacking or anchor frames at either end of the casting bed provide anchorages for the strands to be stressed against. The reinforcement and shutters are assembled around the stressed strand, and the concrete poured. Although the side shutters can be stripped away after a day or two, the concrete is normally required to reach at least 35 MPa before the strands are released from the jacking frames and the force transferred to the beam concrete. For post-tensioning, the beams are made with ducts and anchorages cast in to allow the tendon to be threaded through and stressed after the concrete has sufficient strength. Figures 9.10 and 9.11 show typical pre-tensioning and post-tensioning casting bed arrangements, respectively.



Figure 9.10 Casting bed for pre-tensioned beams



Figure 9.11 Casting bed for post-tensioned beam

For simply supported beams, the applied moment reduces towards the end of beams to zero at the supports. If a constant prestress force with uniform eccentricity is applied along the beam length, this can generate high compression in the bottom and tension in the top over a significant length of the beam ends. To prevent this being a problem for the design, in post-tensioned beams the tendons may be draped with the anchors raised up at the ends (Figure 9.11), while for pre-tensioned beams some of the strands may be debonded by placing in a plastic tube over a length at each end; alternatively, some strands may be deflected up by means of an anchor frame at $\frac{1}{4}$ or $\frac{1}{3}$ points along the beam.

The beams are usually positioned by crane (Figure 9.12); however, for long, multispan bridges or where ground access is poor, an erection gantry (Figure 9.13) may be used to lift the beams and carry them into place. After positioning the beams, the formwork is placed to allow the top slab to be cast. Permanent formwork (Figure 9.6) is used between the beams, which often extend to the edge of the deck. If side cantilevers are required along the edges of the deck, these are formed using temporary works, either supported from the beams themselves or from the ground, if access permits.



Figure 9.12 Crane erection of precast beams



Figure 9.13 Gantry for erection of precast beams

In situ multi-cell box girder bridges

Multi-cell box girder decks are similar to voided slabs, with the voids occupying a larger proportion of the deck section, creating slabs and webs that act together to span between the piers. A typical arrangement is shown in Figure 9.14, with the longitudinal prestressing tendons running through the webs and transverse diaphragms included along the span and at each pier position. Concreting and formwork construction normally dictate a minimum depth of 1200 mm; with depth/span ratios of up to 1:25, the span lengths are normally greater than 30 m and may extend to 50 m. Above 50 m, other forms of construction are usually preferred. The advantages include efficient use of concrete and prestress and simple construction where easy access is available. The disadvantages include the need for extensive labour activities on-site, and the falsework may become complex if access is difficult.

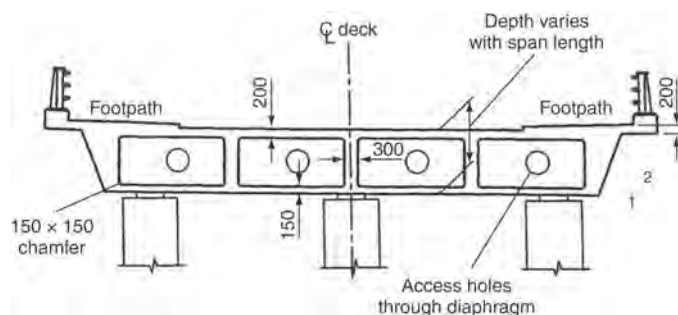


Figure 9.14 Typical multi-cell box girder arrangement

The formwork for concreting may be supported either from falsework off the ground or from a gantry spanning between the piers. It usually simplifies the formwork to cast the deck section in several stages, with the bottom slab, outer webs and diaphragms being cast first, followed by the interior webs and top slab soon after. Traditionally, the time taken to complete the casting of the section is required to be less than 7 days so that any differential shrinkage between the webs, slabs and diaphragms is kept to a minimum. If longer than this, the differential effects need to be considered in the design. Longer decks are usually cast in sections, which can vary from several

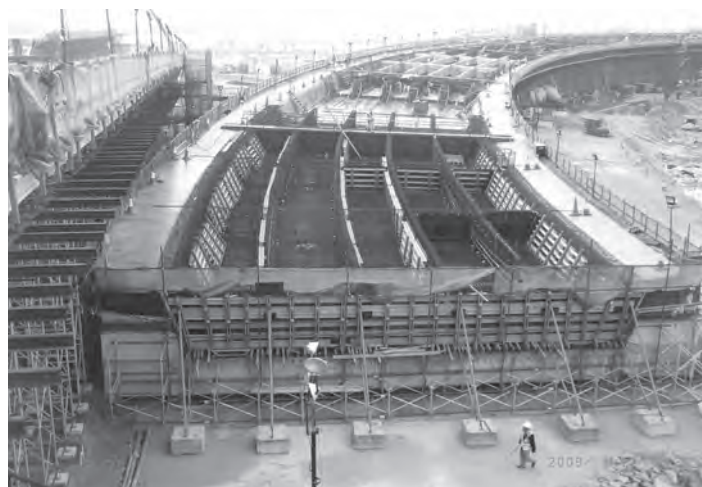


Figure 9.15 Multi-cell box girder deck under construction

spans down to several metres, so that construction and concreting activities can be better controlled, although this has to be balanced against the greater number of construction joints needed across the deck with shorter sections. Figure 9.15 shows a multi-cell box girder being constructed in stages on full-height scaffolding.

In situ single-cell box girder bridges

In situ single-cell box girder decks, such as the typical section shown in Figure 9.16 are used for a wide range of spans, from 40 to 300 m when haunched. The prestress tendons are located in the top and bottom slabs as well as the webs, where a draped tendon profile is used. The advantages are the efficient use of concrete and prestress and the flexibility in span arrangements, while the disadvantages include the labour-intensive activities on-site and the long construction times needed for the larger structures.

For spans up to 60 m, a constant depth section with depth/span ratio of 1:20 would usually be used, with the concrete cast in sections span by span, supported by falsework from the ground (Figure 9.17) or with a truss spanning the piers (Figure 9.18)

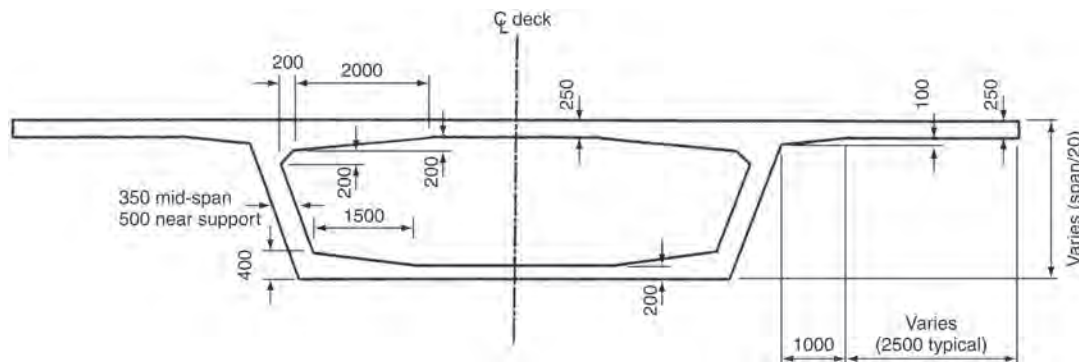


Figure 9.16 Typical single-cell box girder section



Figure 9.17 In situ box girder, construction on falsework



Figure 9.19 In situ balanced cantilever construction



Figure 9.18 In situ box girder cast on self-launching truss



Figure 9.20 Balanced cantilever end span falsework



Figure 9.21 Balanced cantilever pier head segment casting

Above 60 m spans, it is common to use a haunched section cast within a form traveller as a balanced cantilever about a pier (Figure 9.19). Haunched decks have a variable depth along the span, being deeper at the pier and shallower at midspan, with depth-to-span ratios typically 1:16 and 1:45, respectively. Segment lengths are usually between 3 and 5 m long, with a typical casting cycle resulting in a segment being cast every 7–8 days. At midspan, the two opposing cantilevers are joined together with a closure segment and the longitudinal prestress installed to make the deck continuous. The side spans are usually longer than half the main span, resulting in an additional length of deck being needed to reach the end piers after the balanced cantilevering is completed. This can be done by casting the last part of the end span on falsework (Figure 9.20) or by using props to support the cantilevering deck as it is further extended out until the pier is reached. The first part of the deck above the pier is also cast on falsework (Figure 9.21) to provide a working platform to assemble and support the form travellers before the balanced cantilevering commences. The in situ pier head segment length is usually in the region of 12 to 14 m and depends on the length required for the form travellers.

Precast segmental box girder bridges

Precast segmental single-cell box girder decks are found to be very economic for long bridge lengths, owing to the savings associated with maximising repetition in factory conditions. Segments are cast off-site, transported into position and fixed to the previously erected segments to create the final structure. Span lengths typically range from 40 to 150 m, above which the segment weights become excessive. For spans up to 70 m, a constant depth section is often adopted, similar to in situ construction (Figure 9.16), with a depth/span ratio of approximately 1:20. Above 60 m spans, the deck is more likely to be haunched, with depth-to-span ratios of 1:16 at the pier, reducing to 1:35 at midspan. Segments are positioned by crane (Figure 9.22) where access is suitable, by erection gantry (Figure 9.23) for long lengths of viaducts or by lifting frame (Figure 9.24) where a crane or gantries are not suitable. The advantages of precast segmental construction include rapid erection with minimal on-site work, while the disadvantages include the costs of setting up the casting yard and the special erection equipment needed.



Figure 9.22 Precast segmental erection by crane



Figure 9.23 Precast segmental erection by gantry



Figure 9.24 Precast segmental erection by lifting frame

The segments are made in purpose-built casting yards in specially designed formwork, most commonly using the short bed method with shutters as shown in Figure 9.25. Each segment is cast against its neighbour; when hardened, it is moved into the counter-cast position to be cast against, after which it is moved to a storage area. In the alternative long bed method, the segments are cast against each other in a long line on permanent soffit shutters with the rest of the formwork moving down the line for each segment casting. After a series of segments have been cast, they are all removed from the bed to the storage area.



Figure 9.25 Precast segment casting cell

By casting segments directly against each other either in the short bed or long bed method, a perfect fit is achieved when the segments are erected; by carefully controlling the relative position of the segments when they are being cast, complex horizontal and vertical alignments may be achieved, especially with the short bed casting method. Decks have been constructed in this way with horizontal radii of less than 45 m, crossfalls of 10% and gradients of 6%.

Before the segment being placed is positioned against the already erected portion of the deck, the end faces are lightly sand-blasted or cleaned to remove any deleterious material

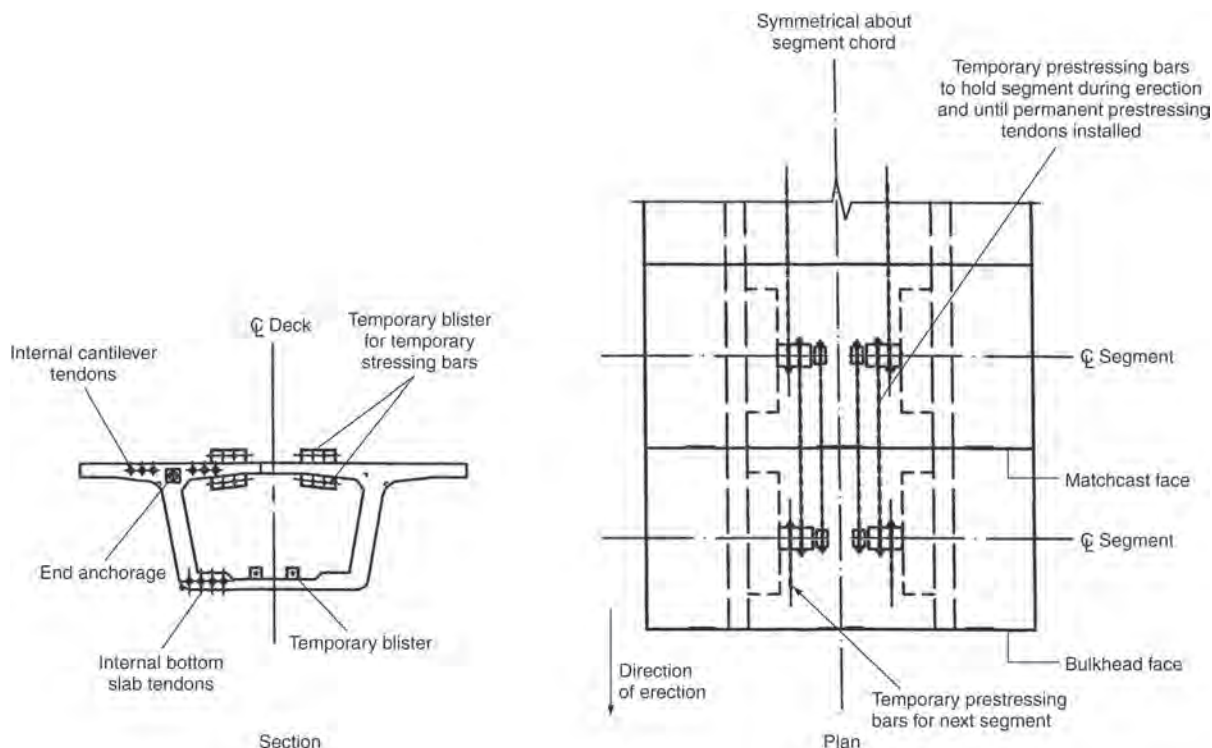


Figure 9.26 Typical temporary prestress for precast segmental erection

and to provide a good key for the epoxy glue, which is spread evenly over the matching faces. Temporary prestress is then applied to hold the segment in place, as indicated in Figure 9.26. The temporary prestress applies an average compressive

stress of between 0.2 and 0.3 MPa, with a minimum local stress of 0.15 MPa and a flexural stress difference less than 0.5 MPa over the segment joint being epoxied, to ensure that the joint thickness is constant throughout, normally between 1 and

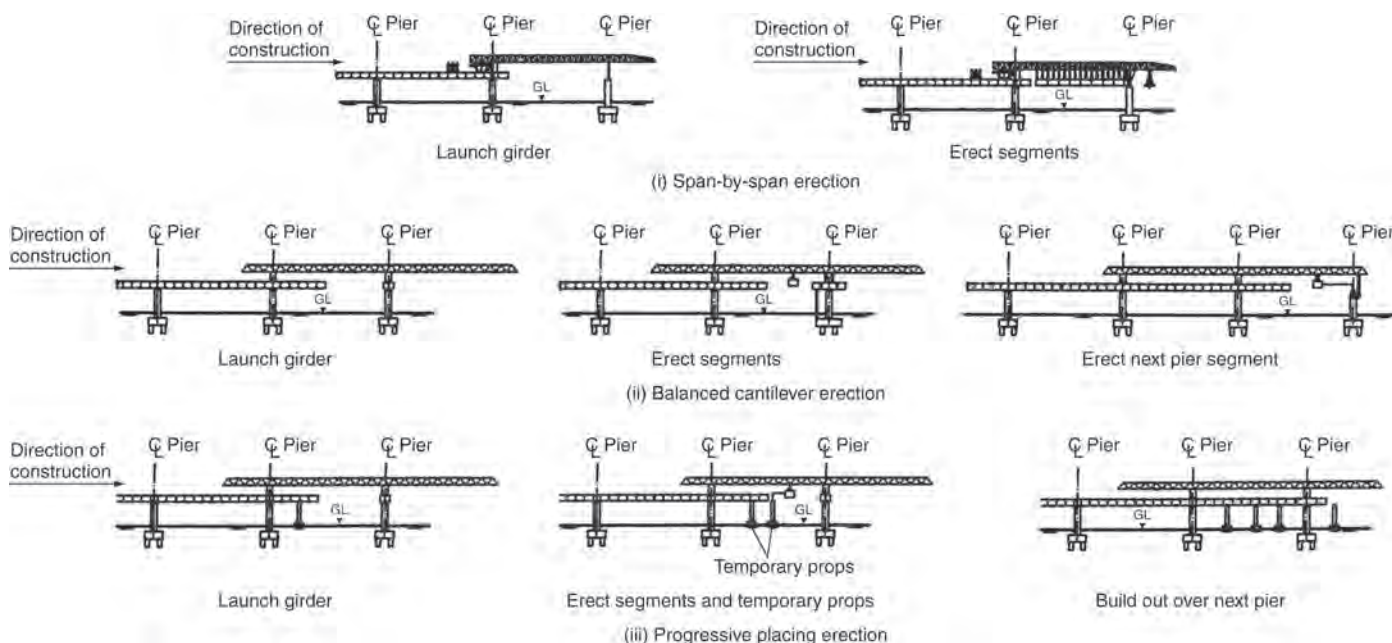


Figure 9.27 Precast segmental deck erection methods



Figure 9.28 Span-by-span segment erection with overhead gantry



Figure 9.29 Span-by-span segment erection with underslung gantry

3 mm. The epoxy helps to lubricate the joint during erection, allowing the segment to slide into position; once hardened, the epoxy seals the joint and provides structural continuity between the segments.

Several bridges have been built with dry joints, where no epoxy is used between the segments and the design relies on shear friction to transfer the loads in both the permanent and temporary conditions. To date, dry joints have been used only in tropical climates where freeze–thaw cycles do not occur; their suitability in more aggressive climates has yet to be proved.

Figure 9.27 indicates three alternative systems for segment erection. The most common method involves using the balanced cantilever technique, either with a gantry (Figure 9.23), a crane (Figure 9.22) or a special lifting frame fixed to the deck (Figure 9.24). The first segment over the pier is lifted into position and placed on temporary supports to allow it to be correctly aligned. Subsequent segments are transported into position and lifted on either side of the pier segment, being fixed to it by epoxy and temporary prestress. Further segments are positioned, with erection progressing out from both sides of the pier in a balanced manner and with permanent prestress cantilever tendons installed as required. When the cantilever reaches midspan, it is connected to the opposing cantilever from the adjacent pier by an in situ concrete stitch and the remaining prestressing tendons are installed along the length of the span and across the stitch to complete the connection.

For spans of less than 50 m, an alternative erection method is to place the segments span by span. In this technique, a gantry, either overhead (Figure 9.28) or underslung (Figure 9.29), is used to support a complete span of segments, which are then pulled together by applying the permanent prestress, allowing the gantry to release the span on to the bearings and to launch itself forward, ready to erect the next span. Above 50 m, the span weight usually becomes too large for this method to be economical.

A third method is progressive placing, where erection is started at one end of the bridge and continues out progressively until the other end is reached. This method usually needs

temporary props along the span to support the deck until the next pier is reached, as shown in Figure 9.30. This method is the least used for precast segmental decks, owing to its slower erection rate compared with the other methods available.



Figure 9.30 Progressive placing of segments

Incrementally launched box girder bridges

Where the bridge alignment is straight or on a constant radius curve, either vertical or horizontal, incrementally launched single-cell box girder decks may be used to overcome access problems or to avoid obstructions at ground level. Commonly used for spans up to 60 m, the technique has occasionally been used for longer spans up to 100 m, with the help of temporary piers placed to reduce the effective span length during launching. Deck depth must be constant for launching purposes, with the ratio to the launched span generally 1:16 or less.

Casting segments behind the abutment, the deck is pushed or pulled out over the piers, as shown in Figure 9.31. A specially prepared casting area is located behind the abutment with sections to assemble the reinforcement, to concrete and

to launch, with a typical layout shown in Figure 9.32. Segment lengths are normally standardised for a bridge and are usually 20–30 m long.



Figure 9.31 Incrementally launched box girder



Figure 9.32 Casting area for launched box girder

The first segment is cast and moved forward on temporary bearings. The second segment is then cast against the first and both are moved forward by a further increment, with subsequent segments cast and the deck moved until it reaches the opposite abutments and its final position.

The area immediately behind the casting bay is utilised for steel fixing and placing the prestressing ducts, which may progress at the same time as other operations. When the deck is launched, the steel cage is attached to the concrete and pulled into position, ready for concreting. The formwork system needs to be specially designed to allow it to be lowered, leaving the deck on temporary supports ready for launching. Temporary bearings are used on each pier and in the casting area to launch the deck and consist of a steel plate, surfaced with stainless steel on laminated rubber pads, with a Teflon pad fed in between the bearing and concrete deck to provide a low-friction sliding surface.

The launching devices may be fixed to the abutments, which provide the resistance against the thrust needed. A typical launching device, as shown in Figure 9.33, would jack up the deck a small amount to grip the structure and then push or pull the deck forward a small distance before dropping down to release

the structure and to move back ready to start another stroke. As the deck moves out, the pushing force has to increase to overcome the frictional force on the temporary bearing, which may be between 2–6% of the vertical load. Greater pushing forces are needed where a deck is being launched up a slope; if going down a slope, a braking device may be needed. The launching force is normally resisted by the abutment, which must be designed to resist sliding or overturning. Additional resistance for the launch may be mobilised by providing the casting area with a ground slab as a working platform and by connecting this slab with the abutment. To keep the deck correctly aligned during the launching operations, guides are fixed to the piers. When the launching is complete, the deck is jacked up and the temporary bearings replaced by the permanent bearings.

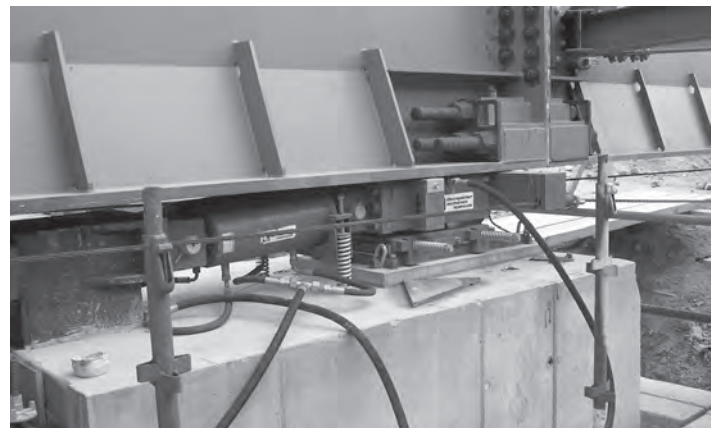


Figure 9.33 Launching device (under launch nose)

As the deck is launched over a pier, large cantilever moments occur until the next pier is reached. To reduce these moments, a temporary lightweight steel launching nose (Figure 9.31) is fixed to the front of the box. The effectiveness of the launching nose is governed by its length and stiffness, and the optimum arrangement needs to be chosen to balance the cost of the nose against the cost of catering for additional moments in the deck. Typically, the launching nose length is about 60% of the span length, and has a stiffness (EI) of about 10–15% of the concrete deck. An alternative to a launching nose is to utilise a temporary tower and stay cables over the front portion of the deck in order to reduce bending moments. The tension in the stays is adjusted as the deck passes over a pier to control the moments and forces imposed on the structure, which complicates the launching process. This process can give rise to safety concerns for the structure and is not commonly used.

Concrete cable-stayed bridges

Providing stay cables to support the deck results in a slimmer deck section and longer spans being achieved. The deck elements may be either a concrete box or a beam-and-slab form; typical deck arrangements are shown in Figure 9.34. The deck may be made up of precast elements, transferred to site and

lifted into position, or in situ segments may be cast inside a travelling form. Either method normally involves building the deck out from the pylons in a balanced manner, installing the stay cables as the construction proceeds. The advantages of a concrete deck are that the horizontal component of the stay force causes compression in the deck, which is readily taken by the concrete, while the higher mass and stiffness of a concrete

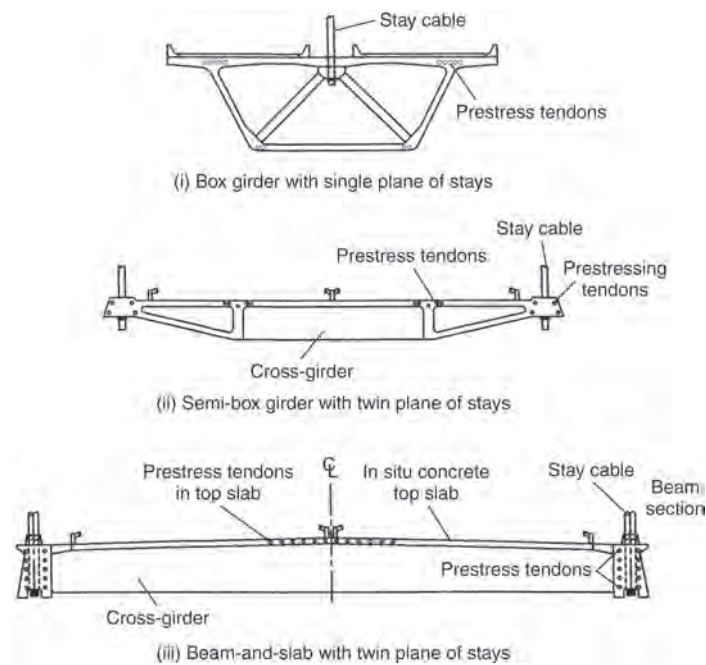


Figure 9.34 Types of cable-stayed deck



Figure 9.35 Vasco da Gama Bridge under construction

deck compared with a steel or composite deck reduces its susceptibility to vibrations or aerodynamic movements.

The box girder shown in Figure 9.34(i) has inherent torsional strength, making it suitable for use with a single plane of stays, while the concrete cross-section is well suited to precasting, enabling fast and simple erection. This form of deck was used for the Brotonne Bridge in France, which had a main span of 320 m and box depth of 3.8 m, and also for the Sunshine Skyway Bridge in Florida, with a main span of 366 m and box depth of 4.47 m.

Where twin planes of stays are used, either a 'semi-box' (Figure 9.34(ii)) or a beam-and-slab (Figure 9.34(iii)) arrangement is commonly used. A semi-box arrangement was used for the Pasco-Kennewick Bridge, USA, with a main span of 299 m and deck depth of 2.13 m, while typical examples of the beam-and-slab type are the Vasco da Gama Bridge, Portugal (Figure 9.35), with a

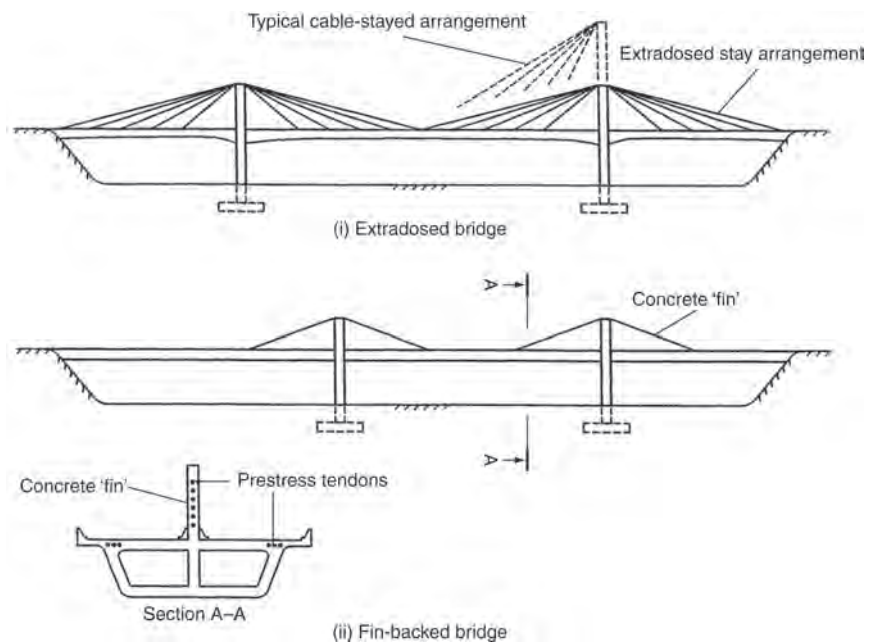


Figure 9.36 Extradosed and fin-back bridge arrangements.

main span of 420 m and deck depth of 2.6 m, and the River Dee Crossing, UK, with a main span of 194 m.

Extradosed bridges

A variation on the cable-stayed arrangement, the 'extradosed' bridge combines a stiff concrete deck with 'shallow' cable stays anchored to a reduced-height pylon and has proven economic in the 100–200 m span range. The relative stiffness of the deck gives a structural behaviour more like an externally prestressed deck than a cable-stayed structure and the design is carried out on that basis. A typical arrangement is shown in Figure 9.36(i).

Fin-backed bridges

These are similar to the extradosed bridge, with the stays encased in a concrete wall or 'fin', as shown in Figure 9.36(ii). The fin is an extension of the deck section and stiffens the deck adjacent to the pier while also providing protection to the stay arrangement.

Truss bridges

Although not commonly used because of the complexity in casting and joining the diagonals and chords of the truss, several precast segmental viaducts and cable-stayed decks have utilised trusses for the 'webs' in order to reduce weight, with a typical arrangement being that used on Bubiyan Bridge, Kuwait, shown in Figure 9.37.



Figure 9.37 Bubiyan Bridge under construction (courtesy of Bouygues)

Stressed-ribbon bridges

Several footbridges have been constructed utilising prestressed concrete as a 'stressed-ribbon' deck, where the tendons are laid out between the abutments and a concrete deck constructed around them before additional prestress is imposed to compress the deck. The Kilmacanogue footbridge in Ireland used this technique to good effect.

Materials in prestressed concrete bridges

Concrete

The required strength of the concrete is determined by the compressive and shear stresses generated in the concrete by the prestress and applied forces. Minimum strengths of $f_{ck} = 35 \text{ MPa}$ (N/mm^2) and $f_{ck, \text{cube}} = 45 \text{ MPa}$ (N/mm^2) are typical for prestressed concrete; however, it is becoming more common to use higher strengths, with $f_{ck, \text{cube}}$ up to 70 MPa not unusual, while even higher values have been achieved for some projects.

The rate of gain of strength of the concrete is important, as this governs the time at which the prestress can be applied. At the time of transfer of the prestress force to the concrete, it is normally required that the concrete strength be at least a minimum of $f_{ck, \text{cube}} = 30 \text{ MPa}$, although this can vary, depending on the tendon and anchor arrangement and the magnitude of load applied to the tendon. To minimise creep and shrinkage losses in the prestressing, the cement content and water/cement ratio (w/c) of the concrete should be kept to a minimum, compatible with the high concrete strengths required.

Special fibre-reinforced concretes, such as Ductal®, where the cement paste and fine aggregate are mixed with steel fibres to achieve compressive strengths up to 200 MPa, have been used on several bridges, although they have not attracted wider use, owing to the higher costs involved. These special high-strength concretes allow thinner, lighter structures and include other advantages for the prestressing, such as higher elastic modulus, minimal shrinkage and lower creep. Ductal® was used for the Peace Footbridge, Korea.

Self-compacting concretes with higher flow characteristics are becoming more common, although still not widely used. These concretes can provide higher strengths and better flow characteristics, allowing easier placing of the concrete around the prestressing ducts and congested reinforcement in areas where there are a number of prestress anchor locations or at diaphragms and deviators.

Prestressing tendons

High-tensile steel is used as wire, strand or bars, with nominal tensile strengths commonly varying between 1570 and 1860 MPa for wire or strand, and between 1000 and 1080 MPa for bars. Typical material properties for tendons are indicated in Table 9.1. Higher-strength wires and strands with tensile strengths of up to 2230 MPa have been developed in recent years but their use has been limited by their availability and cost.

After the load is applied to the prestressing steel, stress relaxation occurs, which results in a reduction of the force in the tendon. The magnitude of relaxation varies, depending on the steel characteristics and the initial stress levels applied, with typical values of 2.5–3.5% when a stress of $0.70f_{pu}$ is applied,

	Nominal diameter: mm	Nominal area: mm ²	Nominal mass: kg/m	Yield strength: MPa	Tensile strength: MPa	Minimum breaking load: kN	Young modulus: kN/mm ²	Relaxation after 1000 h at 20°C and 70% ultimate tensile strength: %
7-wire strand								
13mm super	12.9	100	0.785	1580	1860	186.0	195	2.5
15mm super	15.7	150	1.18	1500	1770	265.0	195	2.5
Stress bars								
20mm	–	314	2.39	835	1030	325	170	3.5
32mm	–	804	6.66	835	1030	828	170	3.5
50mm	–	1963	16.02	835	1030	2022	170	3.5
Cold-drawn wire								
7mm	7	38.5	302	1300	1570	60.4	205	2.5
5mm	5	19.6	154	1390	1670	32.7	205	2.5

Reference should be made to the manufacturers' literature for the properties of individual wires, strands or bars to be used.

Table 9.1 Typical tendon properties

which reduces to below 1% when the initial stress is $0.50f_{pu}$ or less.

Fibre-reinforced polymer tendons have been used on several projects in Europe and the USA to eliminate the corrosion problems associated with steel tendons. The materials available include carbon, glass or aramid fibres with epoxy, vinyl ester or polyester resins, although carbon fibre-reinforced polymers (CFRPs) within an epoxy resin have become the most common.

The properties of the CFRP depend on the fibre and resin design and can give tensile strengths up to 3000 MPa and values of E of 140–300 GPa. The use of CFRP for prestressing tendons appears advantageous but has yet to be widely adopted, owing to their current high cost compared with the traditional steel alternatives.

Cement grout

Cement grout is used to fill the void around post-tensioned tendons and their ducts; a water/cement ratio between 0.35 and 0.40 is typically used, with admixtures often added to improve flow and reduce shrinkage and the water/cement ratio. In some countries, such as the UK, there is a tendency towards prebagged grout, where the cement and admixture are premixed and delivered to the site in accurately measured quantities, with just the water needing to be added.

Grease or wax grout

Grease or wax grout is used for some external tendons to enable destressing or replacement. The grease or petroleum wax is injected into the duct at 80°C or 90°C before cooling to give a soft, flexible filler.

Prestressing systems

Wires

Individual wires are sometimes used in pre-tensioned beams but have become less common in favour of strands, which have better bonding characteristics. Wire diameters are typically between 5 and 7 mm, with a minimum tensile strength of 1570 MPa, and can carry forces up to 45 kN.

Strands and tendons

A seven-wire strand with a tensile strength of 1860 MPa, either 13 or 15 mm in diameter, is a very common form of prestressing and can be used either singularly for pre-tensioning or in bundles to form multi-strand tendons for post-tensioning (Figure 9.38). The most common post-tensioned tendon sizes utilise 7, 12, 19 or 27 strands to suit the standard anchor blocks available, but tendons can incorporate up to 55 strands for larger



Figure 9.38 Prestressing strand, anchor and duct (courtesy of Freyssinet International)

forces. Stressing to 75% ultimate tensile strength (UTS) gives a typical jacking force of 140 or 199 kN for a 13 or 15 mm diameter strand, respectively, while the larger multi-strand tendons can carry forces up to 10 000 kN. During stressing, jacks are placed over the tendon, gripping each strand and pulling it until the required force is generated. Wedges are then placed around the strand and seated into the anchor block so that, on release of the jack, the wedges grip the strand and transfer the force onto the anchor and into the concrete.

Bars

Individual bars can vary in diameter from 15 to 75 mm and are used in post-tensioned construction with jacking forces ranging from 135 to 3000 kN. The bars are placed in ducts that have been cast into the concrete between two anchor blocks on the concrete surface. The bars are pulled from one end by a stressing jack and held in place by a nut assembly, which transfers the load from the bar to the anchor block and then into the concrete (Figure 9.39).



Figure 9.39 Prestressing bar and anchor (courtesy of McCall's Special Products Ltd)

Anchorage

At each end of the tendon, the forces are transferred into the concrete by an anchorage system. For pre-tensioned tendons, the anchorage is by bond of the bare strand cast into the concrete, while for post-tensioned tendons the anchorage can be by anchorage blocks, or by bond for some types of cast-in dead-end anchors. Figures 9.38 and 9.39 show typical live-end anchorages for strands and bars respectively, while Figure 9.40 shows a typical cast-in dead-end anchorage for a post-tensioned strand.

When external post-tensioned tendons are used, they should be removable and replaceable, and the detail of the anchorage should be arranged to allow this. Where cement grout is used, this can be achieved by providing a lining on the central hole of the anchor to ensure that the grout around the tendon does not bond to the anchor, allowing the tendon to be cut and pulled out if necessary.

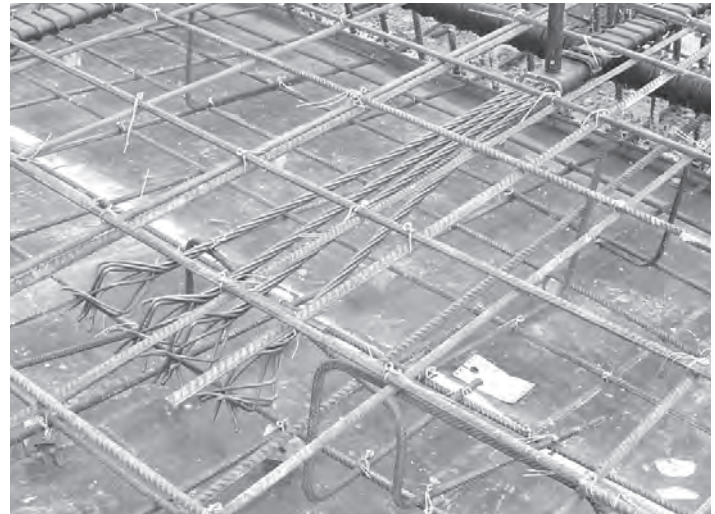


Figure 9.40 Strand with cast-in dead-end anchorage

Stressing jacks

Typical jacks for stressing single- and multi-strand tendons are shown in Figures 9.41 and 9.42, respectively, while Figure 9.43 shows a typical jack for bar tendons. For single strands, smaller tendons or smaller bars, the jacks, weighing up to 250 kg, can usually be easily handled and manoeuvred into position with readily available lifting equipment; for the larger tendons, special lifting frames or cranes are required to move the jacks, which can weigh up to 2000 kg. The design of the bridge deck and anchorage arrangement should always take into account the access needed to set up and operate the stressing jacks and associated equipment needed during the construction.

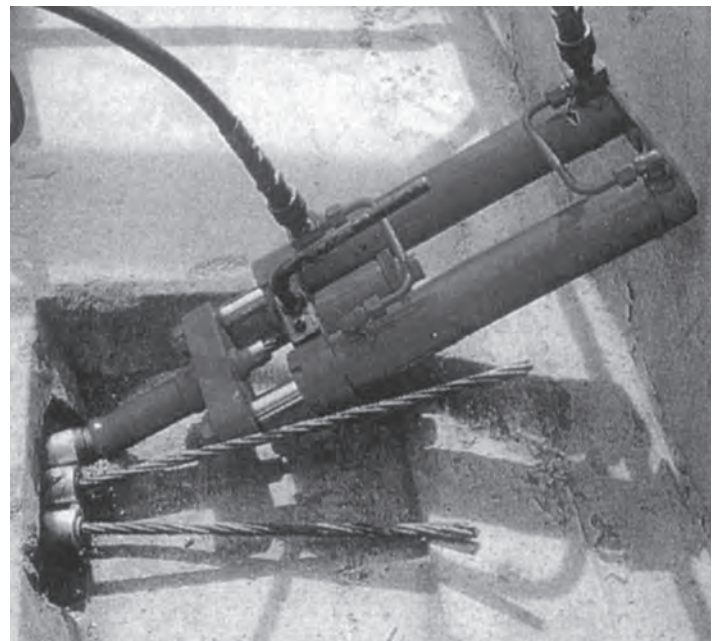


Figure 9.41 Single-strand stressing jack (courtesy of VSL International)



Figure 9.42 Multi-strand stressing jack

Tendon couplers

It is possible to couple tendons together to assist in the stage-by-stage construction methods frequently adopted. Figure 9.44 shows the coupler arrangement for multi-strand tendons, where a special anchor block is used to enable the tendon to be extended into the next stage of concrete after it has been stressed against the previous stage. This arrangement can simplify the tendon layout and save the cost of a separate anchor. However, the concrete section must often be widened to accommodate the coupler arrangement. Some design codes, such as Eurocodes and AASHTO, prohibit the coupling of all tendons at the same location in a deck, to reduce the impact of all the tendons being anchored and then extended at the same point. Bars can also be joined together by a simple threaded coupler, as shown in Figure 9.45.

Corrosion protection and ducting

Wires, bars and strand are generally used uncoated; however, to protect the tendons during storage or to reduce friction losses

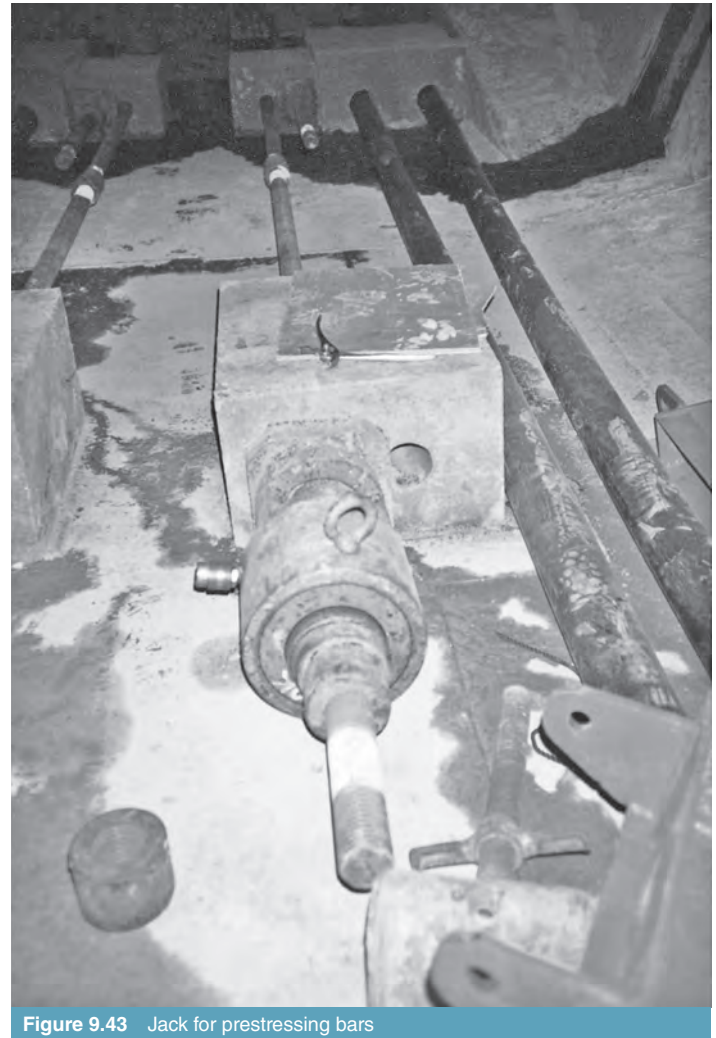


Figure 9.43 Jack for prestressing bars

during stressing, they may be coated with soluble oil, which is washed off prior to grouting the duct. Galvanised bars and strands are also available, as are epoxy-coated strands in some countries, although these have yet to be commonly adopted in the prestressing of bridges.

Pre-tensioned strands and wires are encased by the concrete



Figure 9.44 Tendon coupler (courtesy of VSL International)



Figure 9.45 Bar coupler

and are protected by the benign environment created by the cement. Protection is enhanced by providing a minimum concrete cover to the strands and limiting any tensile stresses in the concrete in the vicinity of the strand to reduce the risk of the concrete cracking, which would reduce the protection provided.

Post-tensioned tendons are normally placed inside ducting to allow them to be stressed inside the hardened concrete, which provides some protection to the tendons. The ducts are then filled with grout to ensure that moisture does not enter and a benign environment is created to give a robust protection system. For internal tendons, the ducts have traditionally been manufactured using galvanised mild steel strips; more recently plastic ducting systems have been adopted to provide a water-tight barrier around the tendon as protection against corrosion, in addition to the cement grout used to fill the ducts after the tendons have been stressed.

For external tendons, high-density polyethylene (HDPE) ducts are used with either cement grout or grease around the strands. The HDPE provides long-term durability, while the advantage of using grease is that it allows the tendons to be more easily destressed and restressed or replaced, which is an important benefit of external tendons. Where cement grout is used, restressing of the tendon is not possible, and removal of the external tendon involves cutting it into short lengths and pulling it out of the deviators and anchor blocks, which requires careful detailing of the duct arrangements. The HDPE ducts need to be strong enough to prevent deformation when pressurised during grouting and to resist the strand punching through at deviated positions, requiring a wall thickness generally greater than 6 mm. Figure 9.46 indicates the different protection systems used.

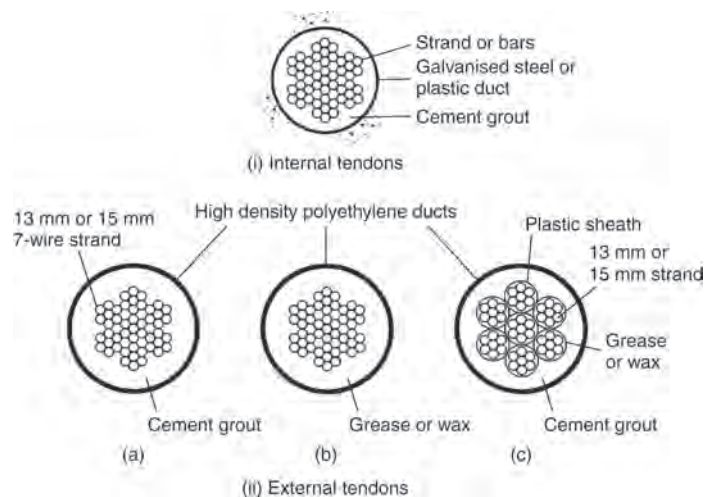


Figure 9.46 Tendon ducts and protection systems.

Grouting of the ducts is normally carried out on-site, although grease filling of external tendons can take place in the factory for single strands in individual sheaths or for smaller multi-strand tendons. For cement grout, the mixing

and pumping into the duct is carried out in a continuous operation, with typical equipment shown in Figure 9.47. The grout is pumped in at the low end and steadily pushed through until the duct is full. Grout vents need to be placed at regular intervals along the duct and at all high and low points to ensure that all the air is expelled and the ducts are filled. Discovery during the 1990s of poorly grouted ducts in a number of bridges in the UK led to a review of procedures and to publication of the Concrete Society Technical Report No. 47 (Concrete Society, 2002), to give guidance on all aspects of grouting of tendons.



Figure 9.47 Grouting equipment

Principle of prestressing

Prestressing of concrete structures is achieved by internal force transfer between the prestressing tendon and the concrete. The tendons are placed within the concrete member, as internal tendons, or alongside the concrete, as external tendons. For post-tensioning, the tendon is pulled using a jack and the force is transferred directly to the hardened concrete, while for pre-tensioning the tendon is jacked against a temporary anchor frame before concreting and the force released from the anchor frame to the concrete when sufficient strength has developed. In these ways, an external compressive force is applied to the concrete, which can be used to counter the tensile stresses generated under the bending moments, torsion and shear forces present.

The concrete can be either fully prestressed, which ensures that the longitudinal stresses are always in compression, or partially prestressed, which allows some tension to occur under certain loading conditions. Modern codes recognise that the design of prestressed and reinforced concrete is similar, with the prestressing treated as an applied compressive force, provided that suitable provisions for serviceability requirements are met.

Prestress design

General approach

The choice of prestress type and arrangement is influenced by the structural form of the deck and the construction methods employed. Pre-tensioned strands or wires are used in precast beams, with typical arrangements shown in Figure 9.8(i), where the economies from construction repetition can offset the cost of providing a casting bed with anchor frames strong enough to resist the total prestress force.

Prestressing bars generally have a higher cost per kilonewton of force than strands but are usually considered to be easier to handle and install and are often used where they simplify construction or in temporary works, such as during the erection of precast segmental decks (Figure 9.48) or during the launching of box girder decks.



Figure 9.48 Prestressing bars used for deck erection

Post-tensioned tendons have the advantages of being installed on-site, with flexibility in the tendon layouts that can be achieved and in the forces that can be applied. Tendons can be profiled to achieve maximum benefit along the span and over the support, such as the arrangements indicated in Figures 9.8(ii) and Figure 9.49. The jacking systems used normally require a tendon length of more than 5 m, while for internal tendons longer than 120 m the friction losses, even when double end stressed, become excessive. External tendons have much smaller friction losses and have been installed with lengths over 300 m.

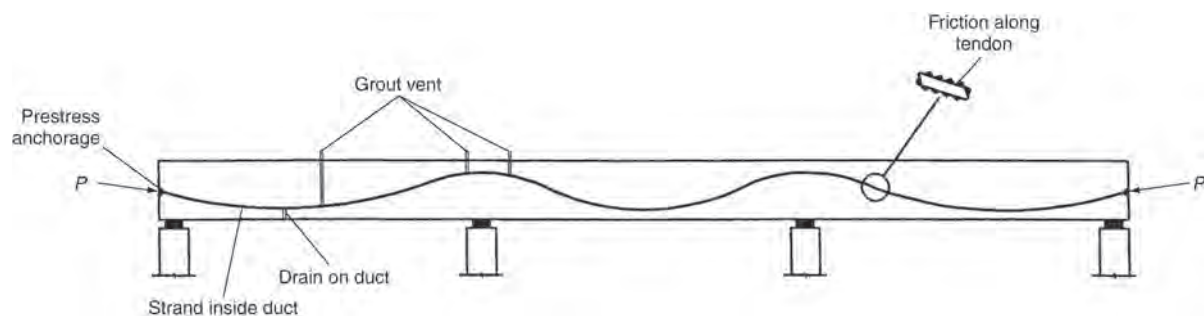


Figure 9.49 Typical post-tensioned tendon layouts

External tendons (Figure 9.50) have been used extensively for both new construction and bridge strengthening and offer a solution where the tendons can be easily inspected and replaced if necessary, something that is difficult where internal tendons are used. For new construction, external tendons cost more per tonne than the equivalent internal tendons, owing to the higher HDPE cost; however, this can be offset against savings elsewhere, as with the tendons outside the concrete section, the concrete sizes, particularly the webs, can be reduced in thickness, and reinforcement fixing and concrete placing is made easier. In precast segmental construction, using external tendons with span-by-span erection has resulted in very rapid erection of long viaducts to give overall cost savings in the project.



Figure 9.50 External tendons

The prestress tendon layouts for most types of bridge structure are now well established, and any new design would normally start by considering a tendon arrangement similar to that used on previous structures of the same type, for example straight strands in the bottom of precast beams, cantilever tendons and continuity tendons for balanced cantilever construction or simple draped tendons for in situ, continuous decks. Consideration of the construction methodology and equipment to be used often influences the choice of anchorage location and whether to use couplers to connect the tendons

during staged casting of the decks. The size of post-tensioned tendons is influenced by the space available to place ducts and anchorages, as well as the access to position jacks. When using post-tensioned multi-strand tendons inside box girder decks, it is often prudent to limit the tendon size to a maximum of 19 strands to more easily position the anchorages and to limit the size of jacks needed for the stressing operations inside the box. Where tendons are stressed on the end of a structure and the stressing jacks are positioned using cranes or lifting frames outside the deck, larger tendons can be used.

For most structures, the prestress quantity is governed by the serviceability limit state (SLS) stress check. When developing the concept designs, the number of tendons required at each critical section, such as midspan and over supports, is estimated by calculating the stresses on the section due to the applied loads and construction effects and deriving the prestress required to keep these stresses within the allowable limits, based on the prestress primary moments and an initial estimate of the prestress secondary moments. A suitable tendon layout is then developed to give the desired prestress at the critical sections; this allows the prestress secondary moment to be derived more accurately and compared with the original estimate. It may take several iterations of adjusting the prestress arrangements before the required prestress stresses are obtained and the actual secondary moment matches the assumed secondary moment to give the final design. This process may be facilitated using modern analysis software, where individual tendons can be added, removed or adjusted quickly within the model and the analysis rerun to check the stresses along the deck. After establishing the prestress requirements at SLS, the ultimate limit state (ULS) moments should be checked at the critical sections and, if necessary, additional tendons added to ensure that adequate resistance can be mobilised, although this would require a recheck of the serviceability stresses to ensure that they are still within the acceptable limits.

For structures with external, unbonded tendons, the quantity of prestress required may be governed by the ULS moment capacity, especially where the increase in stress in the tendon is small, as described later in this chapter, and when no non-prestressed reinforcement is available, as is the case for precast segmental construction. In these situations, the number of tendons required at critical sections can be determined from the ultimate moment check, and a serviceability check can ensure that the stresses are within their acceptable limits.

The magnitude of the prestress secondary moments set up along the deck varies greatly for any type of structural form and will depend on the prestress tendon layout chosen. For most bridges, it is normally found that the secondary moment is sagging, reducing the design hogging moments over the piers and increasing the design sagging moments in midspan. For concrete box girders with spans in the range 35–40 m and typical tendon layouts, the prestress secondary moments may be of the order of 5000–10000 kNm at internal piers, while for heavily prestressed structures and for longer spans the secondary moments may be significantly larger. Secondary moments are eliminated if a concordant tendon profile is adopted, but

in bridgeworks this is not usually achievable, owing to the requirements of the structural form and constructability.

The quantity of prestress needed for any particular bridge form is proportional to the span length of the bridge. Spans of 30 m can use as little as 15 kg/m² of deck, while spans of 100 m can use up to 50 kg/m² of deck. Quantities do vary, depending on the shape of the deck and type of construction, and also on the approach taken by the designer.

In some countries, it is recommended that, when designing post-tensioned structures, allowance is made for the future installation of an additional 10% or more of prestress, which can easily be incorporated with external tendons. The need for this has come about because of excessive loss of prestress or deflections in some existing structures, or the need to upgrade or strengthen the structure to take heavier loads, and it would seem a sensible provision to make on all new post-tensioned bridges.

Primary and secondary effects

When the prestressing tendons apply load to the structure, the resultant forces and moments generated can be considered as a combination of primary and secondary or parasitic effects. For a theoretical beam with a concordant tendon arrangement, no secondary moments are set up, but this rarely occurs in practice. The stage-by-stage construction sequence and envelope of loads applied to the structure inevitably result in a non-concordant prestress layout, and secondary moments may be significant. The secondary moments may often be beneficial by reducing the total hogging moments at the piers, but at the expense of increasing the sagging moments in the span.

The primary effect is that of the moment, shear and axial force generated by the direct application of the force in the tendon on the section being considered. Secondary effects occur when the structure is statically indeterminate and restraints on the structure prevent the prestressed member from freely deflecting when the prestress is applied.

These secondary effects can be derived using a number of different methods, the most common before the advent of modern analysis software being the equivalent load method, where the loads from the tendons are applied directly to the structural model and the combined primary and secondary effects (Figure 9.51) are derived directly from the analysis output. The forces and moments from the tendon, P_h , P_v and M_p , are applied at each anchor position, while along the structure equivalent loads are applied to the model wherever the tendons have an angle change. Care needs to be taken in setting up the model and applying the prestress load to ensure that the structural behaviour is accurately represented. The member layout should follow the neutral axis of the structure, and friction losses in the tendons incorporated by applying appropriate forces and moments on the members along the model.

For simply supported decks, unrestrained at the ends, the structure is free to deflect when prestressed and no secondary effects are set up. For continuous decks, when the prestress is

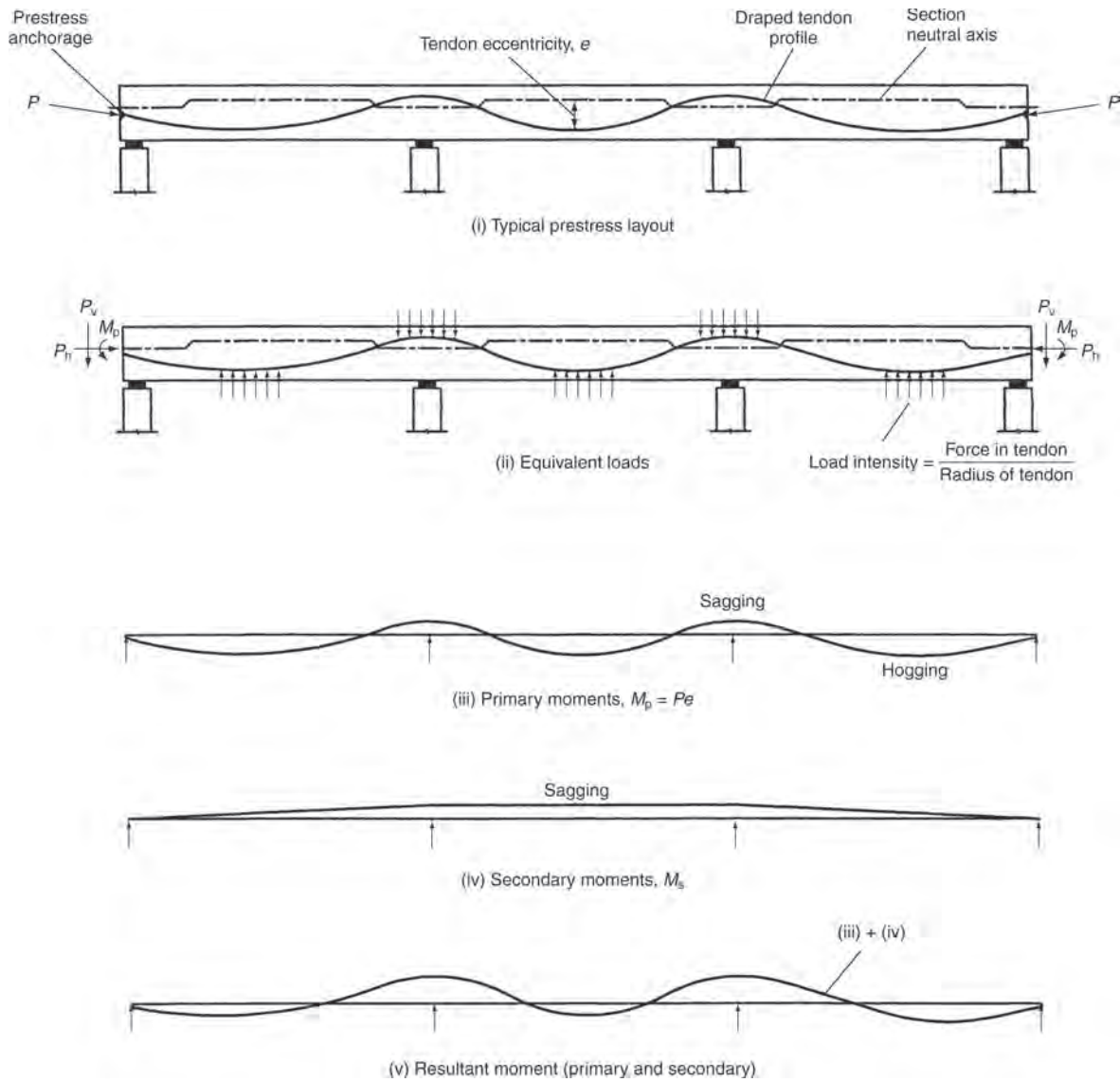


Figure 9.51 Prestress in continuous decks

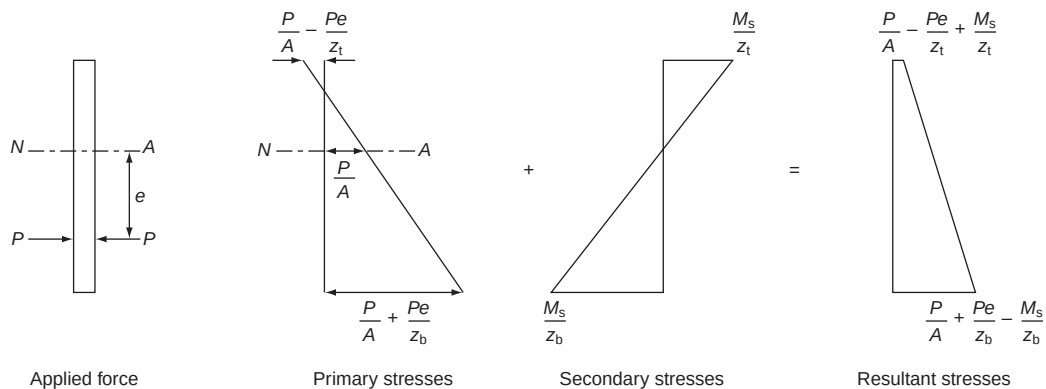


Figure 9.52 Prestress stresses on section

applied, the intermediate supports restrain the deck from vertical movement, and secondary moments and shears occur. The primary and secondary moments both set up longitudinal bending stresses on the section, as indicated in Figure 9.52, and the resultant stress is taken into the serviceability stress analysis.

Where the deck is built integral with abutments or piers, the restraint generated will again set up secondary effects from the prestressing, which must be accounted for in the design. An example of this is shown in Figure 9.53. The prestress tends to compress the portal beam, but this compression is resisted by the columns, generating secondary effects and reducing the prestress compressive forces on the beam. Similarly, the prestress tends to bend the portal beam under its primary effects, and this bending is again resisted by the columns.

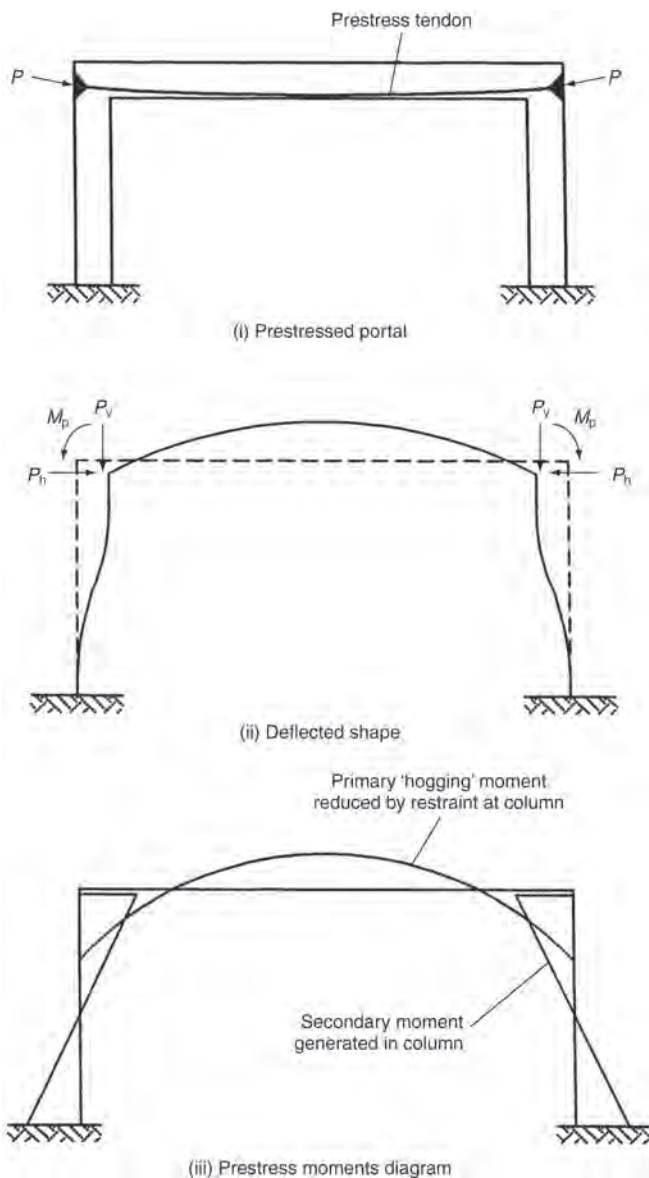


Figure 9.53 Prestress effect with integral bridge decks

Where post-tensioned tendons are used in curved decks, torsions are generated, owing to the secondary effects of the prestress. The intermediate supports along the structure resist the curved decks' tendency to twist, and where the prestress generates secondary moments along the deck and vertical reactions in the supports a complementary torsion is also present. An estimate of the torsion can be derived by computing the secondary moment diagram, assuming that the bridge is straight and dividing this by the horizontal radius of the deck. This M/R diagram is then applied as a load to the structure and the shears generated along the deck are the torsions that will be present in the original structure. Provided that the tendons are symmetrical about the vertical axis of the section, the primary effects of the tendons are not affected by deck curvature; however, where this is not the case, such as with external tendons between anchor and deviator points, a transverse bending and further torsions are generated.

Modern analysis software now allows each individual prestress tendon to be incorporated into the analysis model (Figure 9.54), installed and stressed as part of a stage-by-stage analysis. The friction losses, elastic shortening, concrete creep and shrinkage and tendon relaxation are all automatically accounted for. The software also models the articulation of the bridge, including any integral piers and abutments and the deck curvature, to give an accurate derivation of the secondary effects at each stage of construction, which is then automatically incorporated into the global analysis and design.



Figure 9.54 Tendons modelled in modern analysis software

Prestress force and losses

Tendons can be stressed up to 80% UTS, although some codes limit the jacking load to 75% UTS, and after the jack releases the strand to a maximum stress of 70% or 75% UTS, for post-tensioning or pre-tensioning, respectively. Prestressing bars are normally used straight and encounter little friction

losses. When a multi-strand tendon is pulled by a jack, the movement is resisted by friction along the duct, and the force F at any point along the tendon, x metres from the stressing anchor, is given as

$$F = F_0 e^{-(\mu\theta + kx)} \quad (9.1)$$

Typical values for the coefficients are shown in Table 9.2.

	k	μ
Bare strand in metal ducts	0.001–0.002	0.2–0.3
Bare strand in UPVC ducts	0.001	0.14
Bare strand in HDPE ducts	0.001	0.15
Greased and plastic sheathed strand in polyethylene ducts	0.001	0.05–0.07

Table 9.2 Typical values for the coefficients

For external tendons, or pre-tensioned strands, $k=0$ where the tendons are free from any restraint.

The equation for the force in the tendon along its length is sometimes written as

$$F = F_0 e^{-\mu(\theta + kx)} \quad (9.2)$$

in which case the value of k is adjusted accordingly.

When the tendon is released by the jack, a length of reverse friction is set up as the strand pulls in and the wedges grip. Figure 9.55 gives a typical force profile along the tendon, while the typical pull-in at lock-off for multi-strand tendons could be up to 7 mm.

The extension of the tendon during stressing can be calculated from the force profile:

$$\text{Extension} = \sum F d_x / A_t E_t = (\text{area under force profile}) / A_t E_t \quad (9.3)$$

The extension is usually checked during the stressing process to ensure that the prestress is achieved as expected. Too-small extensions can indicate excessive friction or a blockage in the duct, while too-large extensions can indicate strand slippage at anchors or strand breakage, although that is usually obvious from the loud noise made.

The friction loss along the tendon greatly depends on the tendon profile, and for tendons longer than 40 m excessive losses can occur; however, this can be compensated for by double end stressing, where the tendon is jacked from both ends.

With post-tensioning, when each tendon is installed, the elastic shortening of the concrete causes losses in the tendons already stressed. The losses can be calculated as

$$l_E = \sigma_c (E_t / E_c) \text{ MPa} \quad (9.4)$$

where σ_c is taken as the increase in stress in the concrete adjacent to the tendon, occurring after the new tendon has been stressed.

For pre-tensioning, the full prestress force will generate elastic shortening of the concrete and give a constant loss in each tendon. For post-tensioning, each tendon at a section will suffer a different loss depending on when it is stressed in relation to the other tendons. For most designs, it is sufficient to calculate an average loss and apply this to all the tendons, with σ_c often taken as half the total final stress in the concrete, averaged along the tendon length.

Relaxation of the tendon will reduce the prestress force over a period of time, as will creep and shrinkage of the concrete. Typical relaxation percentages are given in Table 9.1.

Creep losses are

$$l_c = \phi \sigma_c E_t / E_c \text{ MPa} \quad (9.5)$$

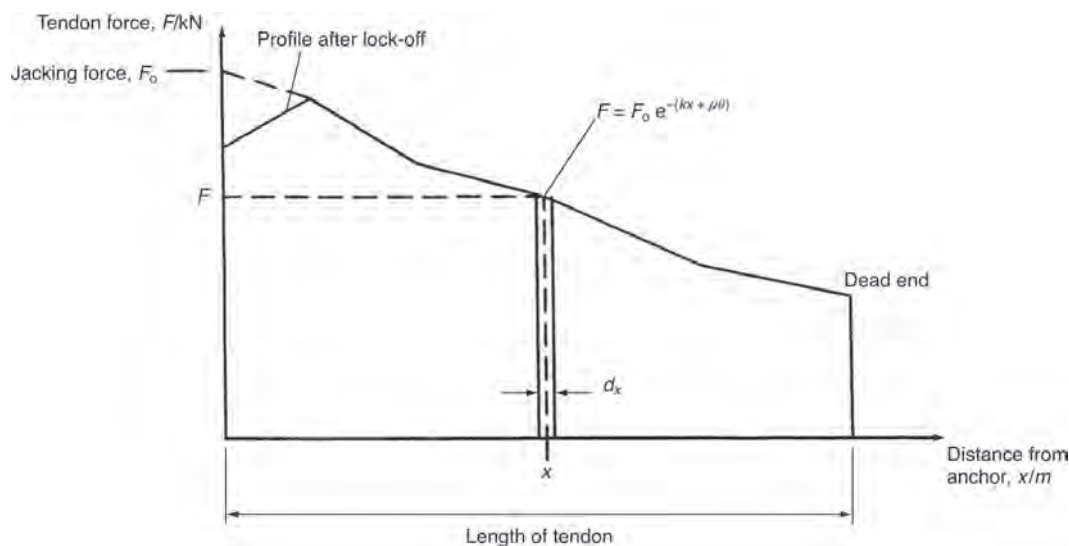


Figure 9.55 Typical tendon force profile

where σ_c is the total stress in the concrete adjacent to the tendon.
Shrinkage losses are

$$l_s = \varepsilon_{cs} E_t \text{ MPa} \quad (9.6)$$

Derivations of ϕ and ε_{cs} are given in Eurocode 2 (BSI, 2004), Part 1-1 Clause 3.1.4 and Annex B. In BS 5400, Part 4 (BSI, 1990), estimates of ϕ and ε_{cs} are given in Appendix C, with typical values of shrinkage and creep losses of standard bridge construction given in Clause 6.7.2.

External tendons are not bonded to the concrete and the elastic shortening, creep and shrinkage losses should be estimated by considering the total deformation between the anchorages or other fixed points as being uniformly distributed along the complete tendon length.

With modern analysis software, incorporating each individual tendon into a stage-by-stage analysis, with time-dependent effects included, allows the tendon force to be determined at every point along its length and at each stage of construction, with this being automatically incorporated into the global design.

Serviceability limit state stress check

In a prestressed deck, the longitudinal stresses or crack widths in the concrete are checked at each stage of construction, and throughout the structure's design life, so that under service loads they remain within allowable limits. Using Eurocode 2 (BSI, 2004) and the UK National Annex, with the normal environment that bridges are exposed to, the following limits apply.

- Compressive stresses:
 - $0.6f_{ck}$ to limit cracking in most environments
 - $0.45f_{ck}$ under quasi-permanent actions above which non-linear creep needs to be considered.
- Tensile stresses and crack widths in concrete:
 - for bonded tendons, concrete within 100 mm of a tendon to be in compression under the frequent loading combination
 - elsewhere, a maximum crack width of 0.2 mm under quasi-permanent actions
 - with unbonded tendons, a maximum crack width of 0.3 mm under quasi-permanent actions.
- Stresses during construction:
 - tension $f_{ctm}(t)$ under the quasi-permanent load combination
 - limit state of cracking should be verified under the quasi-permanent combination of actions
 - durability and final appearance of the completed structure should not be affected, which would tend towards adopting the same criteria as for the completed structure.

Eurocode 2 is currently under revision and these limits are expected to change when the revised documents are issued.

Reference should be made to the latest version of Eurocode 2 to carry out any designs or checking.

Using BS 5400, the following limits apply.

- Compressive stresses:
 - $0.5f_{ci} < 0.4f_{cu}$ with triangular stress distribution
 - $0.4f_{ci} < 0.3f_{cu}$ with uniform stress distribution.
- Tensile stresses:
 - 0 MPa with permanent loads and normal live load
 - $0.45\sqrt{f_{cu}}$ full loading and other effects on pre-tensioned members
 - $0.36\sqrt{f_{cu}}$ full loading and other effects on post-tensioned members.
- Tensile stresses during construction:
 - 1 MPa during erection with prestress and coexistent dead and temporary loads
 - $0.45f_{cu}$ for pre-tensioned members under any loading
 - $0.36f_{cu}$ for post-tensioned members under any loading.

With precast segmental construction, it is normal that no tension is allowed at the joints during either the construction stages or in the completed structure; some countries require a minimum compression of 1 MPa to be maintained with epoxied joints, or 2 MPa with dry jointed construction.

The stress levels should be checked at all the critical stages in the structure's life, including

- at transfer of prestress to the concrete
- during construction, with temporary loads applied
- at bridge opening, with full live load
- after long-term losses in the prestress, and full creep redistribution of moments have occurred.

With external tendons, to allow for tendon replacement, the design of the deck should be checked for the situation when any two tendons are removed. The live loading applied may be reduced to reflect traffic management systems that could be implemented.

Deflections

For normally proportioned prestressed concrete decks and stresses within the allowable limits, deflections are usually not critical and do not need to be checked, other than to confirm the precamber values to be catered for during construction.

Vibrations and fatigue

The vibration from traffic or wind seldom creates a problem and for most prestressed concrete decks it is not necessary to

consider vibrations further; however, if external tendons are used, the individual tendons can be subjected to vibrations if their natural frequency is similar to that of the deck or the traffic passing over.

For either internal or external tendons, the fluctuation in the direct stress in the tendons caused by live and other loading is very small and fatigue is usually not critical; however, with external tendons it is necessary to check that the tendons are not subjected to vibrations that are likely to give rise to fretting of the strands or bending stresses that could cause fatigue problems. This is normally achieved by ensuring that the frequency of the free length of tendons, between anchors or deviator points, is not the same as the natural frequency of

the deck or the traffic using the bridge. Approximate values for frequencies are

$$\text{Frequency of external tendon} = (\pi/L_T) \sqrt{(F/m_t)} \text{ Hz} \quad (9.7)$$

$$\text{Frequency of deck} = (k_f^2/(2\pi L_{sp}^2)) \sqrt{(EI/m_d)} \text{ Hz} \quad (9.8)$$

where k_f depends on span continuity and is π for simply supported spans and 3–4.5 for a continuous deck.

The vibration frequency of live load is often taken as 1–3 Hz for highway traffic, while for rail traffic the behaviour of track supports, track stiffness and natural frequencies of the train bogey leads to a more complex behaviour.

To prevent vibrations in highway bridges, it is normally

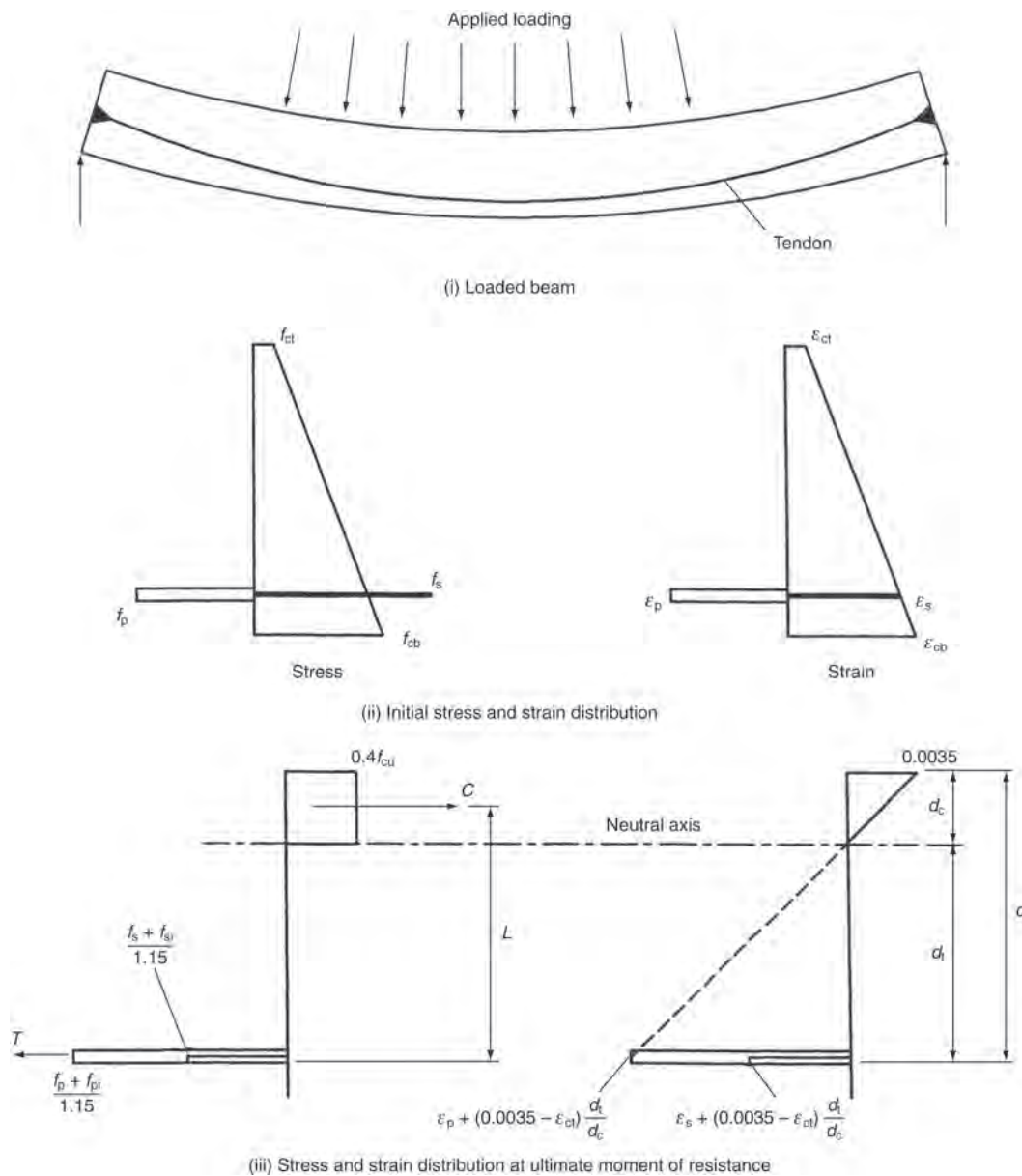


Figure 9.56 Ultimate moment of resistance

sufficient to limit the free length of the tendon to 12 m or less, which should result in the tendon frequency being different from that of the deck and traffic.

Ultimate moment

As the applied bending moment on a prestressed beam increases, the compression on one side goes up, while on the other side the concrete goes into tension. When the tensile strength of the concrete is exceeded, cracking occurs and the tensile stress in the concrete is transferred to either the prestressing tendons or any non-prestressed reinforcement present. As the applied moment further increases, the crack in the concrete opens and propagates across the section, with a pure couple set up between the compression in the concrete and tension in the tendon and non-prestressed reinforcement. The maximum moment is reached when either the concrete or tendon and reinforcement fail. To ensure that there is a sufficient factor of safety against failure, the ultimate moment of resistance of any section along the deck must exceed the bending moment generated by the ultimate loads. It is normally sufficient only to check key points along the deck, such as over the pier and at midspan.

The ultimate moment of resistance is derived by determining the failure condition of the section being considered, with the compression and tension chords being balanced by adjusting the height of the neutral axis, as indicated in Figure 9.56(iii).

When designing to Eurocode 2 (BSI, 2005), the ultimate moment of resistance under the normal persistent and transient situations is considered, while material partial factors γ_c and γ_s are applied with 1.5 for concrete and 1.15 for prestress tendons and reinforcement. Under accidental situations, such as vehicle impact, the factors become 1.1 and 1.0, respectively.

For concrete, Eurocode 2 recommends the maximum design compressive strength, f_{cd} , of $0.67f_{ck}$, while the UK National Annex adopts $0.57f_{ck}$, which is equivalent to $0.45f_{cu}$. This design compressive strength is then applied to the section as a uniform stress over a reduced depth of compression. The maximum strain in the concrete is limited to 0.0035. Referring to Figure 9.56(iii), the stress in the concrete is represented by a rectangular stress block, with the stress being f_{cd} for concrete of strength up to f_{ck} of 50 MPa and $[1 - (f_{ck} - 50)/200] \times f_{cd}$ for concrete strengths up to $f_{ck} = 90$ MPa. The depth of the rectangular stress block is taken as $0.8 \times d_c$ for concrete of strength up to f_{ck} of 50 MPa and $[0.8 - (f_{ck} - 50)/400] \times d_c$ for concrete strengths up to $f_{ck} = 90$ MPa.

The stress in the prestressing and reinforcement on the tension side of the element subjected to the moment is derived from the consideration of strain compatibility with the adjacent concrete, with the maximum stress limited to the characteristic strength divided by the material factors noted previously. For prestressing steel, the stress-strain profile can be simplified, with a constant horizontal profile of $0.87f_{p0.1k}$ after that stress is reached, and similarly with the reinforcement present when it reaches a stress of $0.87f_{yk}$.

With external, unbounded, tendons, the increase in stress in the tendon is determined by considering the overall deformation of the structure, rather than the strain compatibility at any particular section. The overall length increase of each tendon is determined, and the stress increase averaged over the length of the tendon. Eurocode 2 allows the simplified approach of adopting an increase in stress in the unbounded tendons of 100 MPa without considering the overall structure deformations in detail, although this may not be conservative in some circumstances, such as long straight tendons passing through several spans.

When designing to BS 5400 (BSI, 1990) the approach is similar to that when using Eurocode 2, with some small adjustments. The ultimate moment of resistance is derived by considering a balance of tensile force in the tendon and compressive force in the concrete, as shown in Figure 9.56, for internal tendons. In the case of rectangular sections, or flanged sections with the neutral axis in the flange, the compressive stress in the concrete is taken as an average of $0.4f_{cu}$ over the depth of compression, while the concrete strain is limited to 0.0035. The increases in stress in the tendon, f_{pi} , and non-prestressed reinforcement, f_{si} , are derived by considering the increase in strain caused by the bending within the beam and can be obtained by reference to the tendon stress-strain curves, similar to Figures 2 and 3 in BS 5400 Part 4 (BSI, 1990), which are used with $\gamma_m = 1.15$.

To determine the ultimate moment of resistance, an iterative approach is often used, with a first estimate of d_c taken, from which C and T can then be calculated:

$$C = 0.4f_{cu}d_cb_s \quad (9.9)$$

$$T = A_t[(f_p + f_{pi})/1.15] + A_p[f_s + f_{si})/1.15] \quad (9.10)$$

and then d is adjusted and C and T recalculated until $C = T$, giving the ultimate moment of resistance, TL .

The American bridge design code (AASHTO, 2007) requires that the factored moment at a section, $M \leq \phi M_r$, where $\phi = 1.0$ for conventional concrete construction, reduced with precast segmental construction to 0.95 where fully bonded tendons are used and 0.9 where external, unbonded tendons are used.

For external tendons unbonded from the concrete over long lengths, the change in stress under ultimate loads is derived by considering the increase in tendon length between its fixed points as the deck deflects and then using this to estimate an average strain change. The increase in stress in the tendon is thus limited and ultimate failure is normally governed by the compression in the concrete, which can give rise to a brittle failure mechanism. The estimate of the ultimate moment of resistance follows the same procedure as for internal tendons, although for long tendons installed over several spans $f_{pi} = 0$ MPa is normally taken. For short tendons anchored within one span, f_{pi} can be significant, depending on the tendon layout and deck arrangement.

When the ultimate moment of resistance of a section with external tendons is being considered, as the deck deflects, the tendon will remain a straight element between the nearest restraints, while at a point away from the anchors or

deviators the tendon moves closer to the compression face with the lever arm, L , being reduced. This can become significant if the free length of external tendons is longer than a quarter of the span length and the tendons are not restrained near the critical points.

To allow the future replacement of external tendons, the section should have adequate capacity to carry a reduced live loading with any two of the external tendons removed.

Ultimate shear

Shear need only be considered at the ultimate limit state, when the shear resistance must exceed the shear forces generated by the ultimate loads applied. The shear in the deck is checked adjacent to each pier and at regular intervals along the spans, and reinforcement, normally in the form of links, provided as required to give adequate resistance when combined with the concrete resistance. It is usual to assume that all the shear acts on the webs of a section and only the web resistance is taken into account in the design.

For haunched decks, such as with long-span box girders, the longitudinal bending in the deck gives rise to compression in the bottom slab, which acts parallel to the soffit, as shown in Figure 9.57. This compression has a vertical component that acts against the shear forces and can be deducted from the applied shear force, V , in the design.

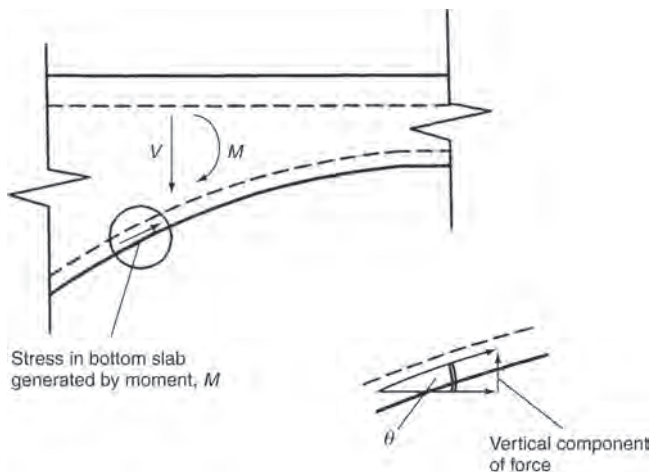


Figure 9.57 Resistance to shear in haunched decks

The presence of the prestress force on the section increases the shear resistance of the concrete; where the tendons are inclined, the vertical component of the prestress usually counters the applied shear force.

When designing to Eurocode 2, the vertical component of force from inclined prestress tendons is treated as part of the load. Eurocode 2 recommends a favourable partial factor of 1.0, while the UK National Annex recommends 0.9, reducing the prestress force to take into account possible variations in the losses in the tendon.

In deriving the shear resistance of the concrete without considering the reinforcement, the prestress increases the resistance by $k_1\sigma_{cp}b_wd$ to give the total resistance of the concrete as

$$V_{Rd,c} = [C_{Rd,c}k(100\sigma_{lfck})^{1/3} + k_1\sigma_{cp}]b_wd \quad (9.11)$$

with a minimum of

$$V_{Rd,c} = (v_{min} + k_1\sigma_{cp})b_wd \quad (9.12)$$

The definitions and values of $C_{Rd,c}$, k , ρ_l and v_{min} that make up the plain concrete resistance can all be found in Eurocode 2:

- k_1 is recommended as 0.15
- σ_{cp} is limited to $0.2f_{cd}$.

Where internal post-tensioned tendons are placed within ducts in the webs of the concrete, allowance must be made for the reduction in effective web width. With non-grouted ducts or prior to grouting the tendons, as well as grouted tendons with plastic ducts or unbonded tendons, the web width b_w is reduced by $1.2\times$ the outer duct diameter, while for grouted tendons in metal ducts $0.5\times$ the outer duct diameter is deducted. For grouted tendons in metal ducts where the outer diameter is less than $0.125b_w$, the full value of b_w is taken.

For sections uncracked in bending, defined as when the flexural tensile stress in the concrete is less than $f_{ctk,0.05}/\gamma_c$, the shear resistance is given by

$$V_{Rd,c} = [Ib_w/S] \times \sqrt{[(f_{ctd})^2 + \alpha_1\sigma_{cp}f_{ctd}]} \quad (9.13)$$

where $\alpha_1 = 1.0$ for post-tensioned tendons and pre-tensioned tendons outside of the transmission zone, and the ratio of the distance from the start of the transmission to the upper bound transmission length within the transmission zone.

This uncracked shear resistance limit is not applied to sections nearer to a support than where a 45° line drawn from the edge of the support intersects the section neutral axis.

The ultimate shear resistance at any section requiring shear reinforcement is derived using a truss analogy, with a similar approach adopted for prestressed concrete as for reinforced concrete, with the formulae applied adopted to suit. The shear is taken on a compression strut in the concrete, which is assumed to be inclined between 45° and 22° to the horizontal, with tension and compression chords in the concrete. Where reinforcement is required to increase the shear resistance, this usually takes the form of vertical bars placed between the tension and compression chords. Where shear reinforcement is provided in bridge structures, it is usually perpendicular to the member axis; in the following formulae, the angle of the shear reinforcement has been taken as 90° .

The design resistance of the reinforcement to be added to the shear resistance of the concrete is given as

$$V_{Rd,s} = (A_{sw}/s)z f_{ywd} \cot \theta \quad (9.14)$$

$$Z = 0.9d \quad (9.15)$$

where θ is the assumed angle of compression strut.

From this, it is seen that the smaller the angle of the compression strut with the horizontal the greater the resistance of the reinforcement provided, as there are more legs of the links crossing the potential failure plane. This results in a reduction in the vertical reinforcement required, but is offset by an increase in the reinforcement to cater for the additional longitudinal force, ΔF_{ld} , associated with the applied shear force, which is given by

$$\Delta F_{ld} = 0.5 V_{Ed} \cot \theta \quad (9.16)$$

The maximum tensile force in the longitudinal reinforcement is limited to that required to cater for the maximum bending moment along the deck. At the point of maximum moment, the shear force is minimal.

Where the concrete resistance is adequate to cater for all the applied shear, a minimum percentage of shear reinforcement equal to $(0.08 \sqrt{f_{ck}})/f_{yk}$ is required.

The maximum shear force on any section must be limited to prevent crushing of the concrete in the compression strut, with the maximum shear resistance of the section, $V_{Rd,max}$ given by

$$V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd} / (\cot \theta + \tan \theta) \quad (9.17)$$

where the coefficients α_{cw} and v_1 are given in Eurocode 2 but are modified in the UK National Annex.

It is seen that if a shallow angle of the compression strut is chosen, this will reduce the maximum shear resistance that the section can generate. The choice over the angle of the compression strut, between 45° and 22° , will depend on the governing factors in the section. A larger angle will allow thinner webs to be adopted in the section, while smaller angles will reduce the amount of shear reinforcement needed.

Eurocode 2 does not differentiate between internal and external tendons and applies the same shear design approach to both types of prestressing.

When designing to BS 5400 Part 4 (BSI, 1990), the shear capacity at any section is the sum of V_c , the ultimate shear resistance of the concrete, and V_s , the ultimate shear resistance of the reinforcement present.

With internal tendons, V_c is the lesser of V_{co} , the ultimate shear resistance of a section uncracked in flexure, and V_{cr} , the ultimate shear resistance of a section cracked in flexure:

$$V_{co} = 0.67 b h \sqrt{f_t^2 + f_{cp} f_t} + V_p \quad (9.18)$$

$$V_{cr} = 0.037 b d \sqrt{f_{cu}} + (M_{cr}/M) V \quad (9.19)$$

$$M_{cr} = (0.37 \sqrt{f_{cu}} + f_{pt}) (I/y) \quad (9.20)$$

The values for V_p , f_{cp} and f_{pt} are usually based on the prestressing forces after all losses have occurred and are multiplied by the partial safety factor, $\gamma_n = 0.87$. Where the vertical component of prestress, V_p , resists shear at the section being considered, this is usually ignored in calculating V_{cr} .

Standard BS 5400 Part 4 (BSI, 1990) requires a minimum shear reinforcement to be provided, such that

$$(A_{sv}/S_v)(0.87 f_{yv}/b) = 0.4 \text{ MPa} \quad (9.21)$$

When V exceeds V_c , shear reinforcement is required, such that

$$(A_{sv}/S_v) = (V + 0.4 b d_t - V_c) / (0.87 f_{yv} d_t) \quad (9.22)$$

Longitudinal reinforcement should be provided in the tensile zone, such that

$$A_{sl} \geq V/2(0.87 f_{yl}) \quad (9.23)$$

This quantity can include both the non-prestressed reinforcement and any bonded prestressing tendon not being utilised at the ultimate limit state for other purposes.

The maximum shear force V is limited to $0.75 \sqrt{f_{cu}} b d$ to prevent excessive principal stresses occurring in the concrete.

For internal post-tensioned tendons within ducts in the webs of the concrete, allowance must again be made for the reduction in effective web width. Prior to grouting the tendons, the full duct width is deducted from b ; after grouting, two-thirds of the duct diameter is deducted.

When checking the ultimate shear capacity at the ends of pre-tensioned beams, allowance must be made for the loss of prestress over its anchorage length, L_a , as defined previously in the chapter. It is normally sufficient to consider the prestress force as varying linearly over this length and, by applying the formula given here, derive V_c . If the shear resistance based on the beam being simply reinforced, ignoring the prestress, is greater than that calculated assuming a prestressed beam, the larger value can be taken.

Standard BS 5400 Part 4 (BSI, 1990) does not cover external tendons, while BD 58/94 (Highways Agency, 1994) requires that sections with unbonded external tendons are designed as reinforced concrete columns, subjected to an externally applied load, with

$$v = (V - 0.87 V_p) / b d \quad (9.24)$$

where v should not exceed 5.8 MPa:

$$\text{when } v \leq \xi_s v_c, \text{ then } A_{sv} \geq 0.4 b s_v / 0.87 f_{yv} \quad (9.25)$$

when $v > \xi_s v_c$, then

$$A_{sv} \geq b s_v (v + 0.4 - \{1 + [0.05(0.87 P)/A_c]\} \xi_s v_c) / 0.87 f_{yv} \quad (9.26)$$

where v_c and ξ_s are given in BS 5400 Part 4 (BSI, 1990), Tables 8 and 9, respectively.

AASHTO's approach to load factor design differs from BS 5400 Part 4 in that, for shear, $V \leq \phi(V_c + V_s + V_p)$, in which V_c is $0.083 \beta \sqrt{f'_c} b d$. In this equation, β is a factor indicating the ability of diagonally cracked concrete to transmit tension and, where at least the minimum shear reinforcement is provided, varies between 1.5 and 6.32, depending on the applied shear stress and the longitudinal strain in the section:

$$V_s = a_{sv} f_{yv} d (\cot \theta + \cot \alpha) \sin \alpha / S_v \quad (9.27)$$

where θ and α are the inclination of the diagonal compressive stress and inclination of shear reinforcement respectively,

$$V_s = (a_{sv} f_{yv} d / S_v) \geq 0.644 \sqrt{f'_c} b d \quad \text{and} \quad d \geq 0.8 h \quad (9.28)$$

The strength reduction factor, ϕ , is taken as 0.9 for internal tendons and 0.85 for external tendons.

For external tendons, AASHTO treats shear design in the same way as for internal tendons when calculating V_c and the shear reinforcement to be provided.

With external tendons, the shear capacity should be adequate to carry a reduced live loading with any two tendons removed.

Ultimate torsion

Torsion does not normally govern the dimensions of the concrete members but will require additional reinforcement in the section and is considered at the ultimate limit state.

When designing to Eurocode 2, the torsional resistance is considered on the basis of a thin-walled closed section. Solid sections are represented by equivalent thin-walled closed sections, while open beam sections are considered as a series of thin-walled sections, totalled to get the overall torsional resistance.

The shear stress in any wall is given by

$$T_{ed}/(2A_k t_{ef,1}) \quad (9.29)$$

The shear force, $V_{ed,1}$, in any wall is given by

$$\tau_{t,1} t_{ef,1} z_i \quad (9.30)$$

The effects of both the shear and torsion are combined and the design carried out as for the ultimate shear resistance.

The additional longitudinal reinforcement per metre around the perimeter required for the torsion is given by

$$A_{sl} = T_{ed} \cot \theta / (2A_k f_{yd}) \quad (9.31)$$

This additional longitudinal reinforcement is added to the other tension reinforcement required but can be reduced in the compression areas of the section, subject to the compressive force available.

When considering the maximum capacity of a section, the torsion and shear resistance are considered together, such that

$$T_{ed}/T_{Rd,max} + V_{ed}/V_{Rd,max} \leq 1.0 \quad (9.32)$$

For a closed thin-walled box structure, the design torsional resistance moment $T_{Rd,max}$ is given by

$$T_{Rd,max} = 2v\alpha_{cw}f_{cd}A_k t_{ef,1} \sin \theta \cos \theta \quad (9.33)$$

where v and α_{cw} are those values adopted in the shear design.

When designing to BS 5400 Part 4 (BSI, 1990), in determining the torsional capacity it is necessary to calculate the torsional shear stresses generated by the ultimate loads; where these stresses exceed V_{min} as given in Table 10 of BS 5400 Part 4, reinforcement is provided by means of transverse links and longitudinal bars. The stresses generated by torsion and shear are added and the sum

must not exceed $0.75\sqrt{f_{cu}}$ and must be less than 5.8 MPa.

For rectangular sections, the torsional shear stress, v_t , is given by

$$v_t = [2T_u/h_{min}^2(h_{max} - h_{min}/3)] \quad (9.34)$$

and torsional reinforcement provided, such that

$$A_{st}/S_v T_u / [1.6x_i y_i (0.87f_{yv})] \quad (9.35)$$

$$A_{sl}/S_l \geq A_{st}(f_{yv}/f_{yl})/S_v \quad (9.36)$$

Where T and I sections are used, the torsion is considered as acting on the individual rectangular elements, with the section divided up to maximise the sum of $(h_{max}h_{min}^3)$ of each rectangle. Each rectangle is then designed to carry a proportion of the torsion based on its value of $(h_{max}h_{min}^3)$ in relation to the sum of the values for all the rectangles, reinforcement being determined as for normal rectangular sections and detailed to tie the individual rectangles together.

For box sections:

$$v_t = T_u/2h_{wo}A_o \quad (9.37)$$

and reinforcement is provided such that

$$A_{st}/S_v \geq [T_u/2A_o(0.87f_{yv})] \quad (9.38)$$

$$A_{sl}/S_l \geq A_{st}(f_{yv}/f_{yl})/S_v \quad (9.39)$$

Where a part of the section is in compression, this compressive force may be used to reduce the longitudinal reinforcement required. This reduction in A_{sl}/S_l is given by $f_c h_{wo}/0.87f_{yl}$.

For precast segmental decks where no reinforcement passes through the joints, and where the joints should be in compression at all times, the longitudinal torsional stresses need to be countered by a residual longitudinal compression from the prestress equal to $T_u/2h_{wo}A_o$.

Longitudinal shear between web and flanges

As the bending moments change, the flow of stress through the section gives rise to longitudinal shear between the different parts of the deck section. This is usually checked at the critical slab-web interfaces, as indicated in Figure 9.58. The shear

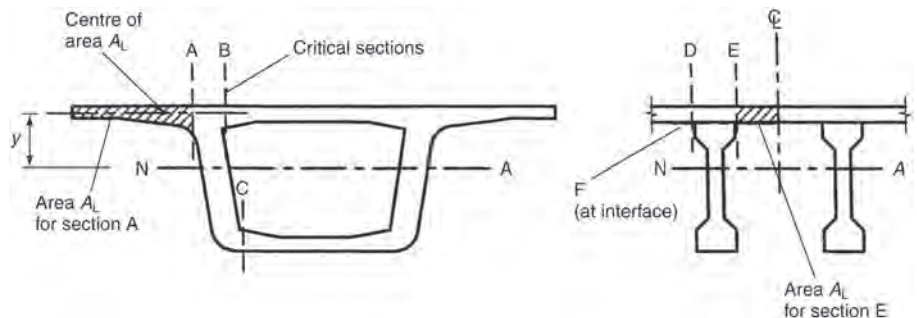


Figure 9.58 Longitudinal shear

along the interface is derived by considering the change in the normal force on the flange along the element. By calculating the compressive stress on the flange at the ends of a selected length of deck, the shear force can be determined and the shear stresses checked.

The ultimate longitudinal shear force per unit length is given by

$$V_L = V(A_L y / I) \quad (9.40)$$

where A_L and y refer to the area of concrete outside of the section being considered.

When designing to Eurocode 2, the shear reinforcement required across the interface is given by

$$(A_{sf} f_{yd} / s_f) \geq v_{Ed} h_f / \cot \theta_f \quad (9.41)$$

The angle θ_f is the chosen angle of the compression strut in the flanges and is limited to between 45° and 26.5° for compression flanges and between 45° and 38.6° for tension flanges.

To prevent crushing of the compression struts in the concrete, the maximum shear stress must be such that

$$v_{Ed} \leq v_{fd} \sin \theta_f \cos \theta_f \quad (9.42)$$

When combining longitudinal shear and transverse bending, the reinforcement provided must be greater than either A_{sf} , as described, or 50% of this value added to the transverse bending requirement.

When designing to BS 5400 Part 4 (BSI, 1990), the concrete is considered to provide some resistance to the longitudinal shear, with reinforcement required to ensure that V_L is less than both:

$$k_1 f_{cu} L_s \quad (9.43)$$

and

$$v_L L_s + 0.7 A_s f_y \quad (9.44)$$

Values for v_L and k_1 are given in BS 5400 Part 4, Table 31. For precast beam-and-slab construction, a minimum area of reinforcement of 0.15% of the contact area should be provided across the interface between the beam and the slab; BS 5400 Part 4 allows reinforcement that is provided for other purposes to be utilised to resist the longitudinal shear.

AASHTO (2007) considers a clamping force provided by the reinforcement and any applied compression across the interface, combined with a cohesion and aggregate interface when deriving the nominal shear resistance. This is then multiplied by a resistance factor before comparing with the factored interface shear forces.

$$V_{ni} = c A_{cv} + \mu (A_{vf} f_y + P_c) \quad (9.45)$$

with

$$V_{ni} \leq K_1 f'_{c A_{cv}} \quad \text{or} \quad V_{ni} \leq K_2 A_{cv} \quad (9.46)$$

The cohesion, c , friction factor, μ , and factors K_1 and K_2 are typically 2.8, 1.4, 0.25 and 10.3 MPa, respectively, for monolithic concrete, with other values for different situations given in AASHTO Section 5.

Partial prestressing

Partial prestressing refers to the situation where prestress is provided, but in insufficient quantities to achieve a fully prestressed design, and reliance is put on the reinforcement to limit cracking of the concrete in the tensile zones. Several countries, including Denmark and Australia, have successfully adopted this approach in their bridge designs in the past although, historically, partially prestress bridges have not been accepted in the UK.

The approach taken by Eurocode 2 is to treat all reinforced and prestressed structures the same, to effectively create partially prestressed structures to some degree. The requirement for prestressed concrete bridges in normal environments is to ensure that the concrete around the tendon remains in compression under the frequent loading combination; outside of that, the crack widths are limited by the provision of reinforcement.

In practice, most prestressed concrete bridges will have prestressing tendons near the surface of the concrete, such that the extreme fibres of the deck at critical locations (e.g. top over piers and bottom in midspan) will need to be in compression under the frequent loading combination but that away from the critical positions, where the prestress tendons are not near the surface, partially prestressed situations can arise. This does not affect the shear and torsion designs under Eurocode 2.

With BS 5400 Part 4 (BSI, 1990), partially prestressing is classified as Class 3, and is not currently adopted for bridge-works where this code and the UK BD documents are adopted.

When designing with BS 5400 Part 4, for fully prestressed structures, the design is based on the concrete being uncracked under service loading, either as Class 1, where the stresses are in compression under all loading conditions, or as Class 2, where, under transient loading, small tensions are allowed, which are kept below the tensile strength of the concrete. For partially prestressed structures, the philosophy is to allow the concrete to crack under service loading and to limit the crack widths to the normal allowable limit for reinforced concrete. This is often related to a hypothetical allowable tension in the concrete of up to 6 or 7 MPa when carrying out the longitudinal stress check. The ultimate moment and torsion design is carried out in the normal manner, while for ultimate shear the concrete resistance is calculated as cracked in flexure, with

$$V_{cr} = [1 - 0.55(f_{pe}/f_{pu})] v_c b d + M_o (V/M) \quad (9.47)$$

$$M_o = f_{pt} (I/y) \quad (9.48)$$

and any vertical component of prestress ignored.

The values of f_{pe} and f_{pt} are based on the prestressing force after all losses have occurred and multiplied by the partial safety factor, $\gamma_f = 0.87$, and for the purpose of this equation f_{pe} should be $\geq 0.6 f_{pu}$, while v_c is given in BS 5400 Part 4, Table 8 (BSI, 1990), in which the value of A_s is the total area of prestressed or non-prestressed reinforcement in the tensile zone.

A fatigue check of the prestress is needed to ensure that the service stress fluctuation in the strand is not critical for the level of stress present. Normally, checking to ensure the stress range in the prestress is below 120 MPa is sufficient.

Precamber

Deflections of the concrete deck occur under the self-weight and from the weight of the permanently applied loads, followed by further movements owing to long-term creep of the concrete and losses in prestress. The deck must be cast and erected so that the theoretical profile is achieved on completion. The adjustment made to the profile during casting to achieve the desired shape is called the precamber; this will be affected by the construction sequence and concrete properties.

The creep effects will cause the deck to change its profile over its life; it is usual to aim to achieve the desired alignment at the time of bridge opening, although long-term changes in deflections should be checked to ensure that they are not excessive.

Construction sequence and creep analysis

The way the bridge is built affects the moments and shears generated in the structure; this needs to be fully taken into account during the design. The structure should be checked for strength and stability, and serviceability stresses assessed at each stage of construction, with the final moments and shears derived to reflect the construction sequence. For example, Figure 9.59 shows the dead load bending moments in a four-span deck constructed in stages, with the final moments after creep being between the as-built moments and the moment as if the deck were built instantaneously.

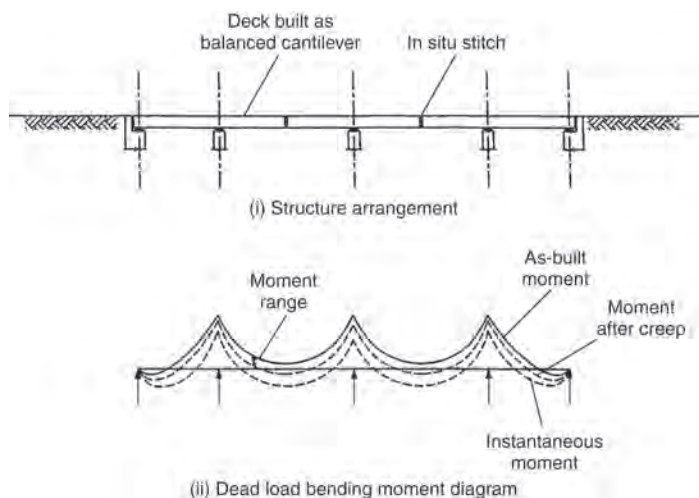


Figure 9.59 Creep redistribution of moments

When a concrete structure's static system is changed during construction, creep of the concrete will modify the as-built bending moments and shear forces towards the 'instantaneous' moment and shear distribution, the amount of the change being dependent on the creep factor, ϕ , of the concrete:

$$\phi = \text{creep strain/elastic strain} \quad (9.49)$$

The value of ϕ varies, depending on the concrete constituents and details, environmental conditions and age of concrete; this variation can be from 1.3 to 3 or more. For precast construction, ϕ is normally around 1.6, while for in situ construction it would normally be between 2.0 and 2.5. Figure 3.1 of Eurocode 2 (BSI, 2005) and Appendix C.2 of BS 5400 Part 4 (BSI, 1990) give detailed derivations of ϕ when using the different codes.

Where the change to the static system is sudden, such as when connecting cantilevers with a midspan stitch, the modification to the moments is

$$M_{\text{final}} = M_{\text{as-built}} + (1 - e^{-\phi})(M_{\text{inst}} - M_{\text{as-built}}) \quad (9.50)$$

Where the change is gradual, such as the differential shrinkage between precast beams and in situ top slab, the modification to the moments generated if the change was instantaneous, M_a , becomes

$$M_{\text{final}} = M_a[(1 - e^{-\phi})/\phi] \quad (9.51)$$

Shears are modified in a similar way to the moments.

Temperature effects

Changes in effective temperature of the deck will cause it to expand or contract, while differential temperature gradients through the concrete result in stresses that need to be considered in the prestress design.

Under a uniform temperature change, the change in deck length is $\Delta L = \Delta t \alpha L$.

The coefficient of thermal expansion, α , is typically $12 \times 10^{-6}/^\circ\text{C}$ for normal concrete or $9 \times 10^{-6}/^\circ\text{C}$ with limestone aggregates.

Where a deck is free to expand or contract, this overall change in effective temperature will not give rise to any forces in the structure, although the movement does need to be allowed for in the bearing and expansion joint design. When a restraint exists that restricts free movement, such as integral decks or a number of fixed piers, forces and stresses are generated throughout the structure and must be taken into account in the prestress design.

Differential temperature effects are generated through the concrete section, as the outer surface heats or cools more quickly than the rest. Eurocode 1, Part 1-5 (BSI, 2003) provides two approaches for the temperature differential through a concrete bridge deck; the UK NA requires approach 2 to be adopted. Standard BS 5400 Part 2, Figure 9 (BSI, 2006) defines a series of simplistic temperature gradients that can be applied to different structural types. Figure 5.60 illustrates how the temperature gradients generate stresses and forces within the deck. The equivalent forces generated by the temperature differential give rise to an out-of-balance axial force and moment. Provided the deck is not restrained, the force and moment will relieve at the ends and the stresses will be adjusted accordingly.

to give the resultant stress distribution across the section. These stresses are then catered for in the design when considering the serviceability stress check of the prestressed concrete.

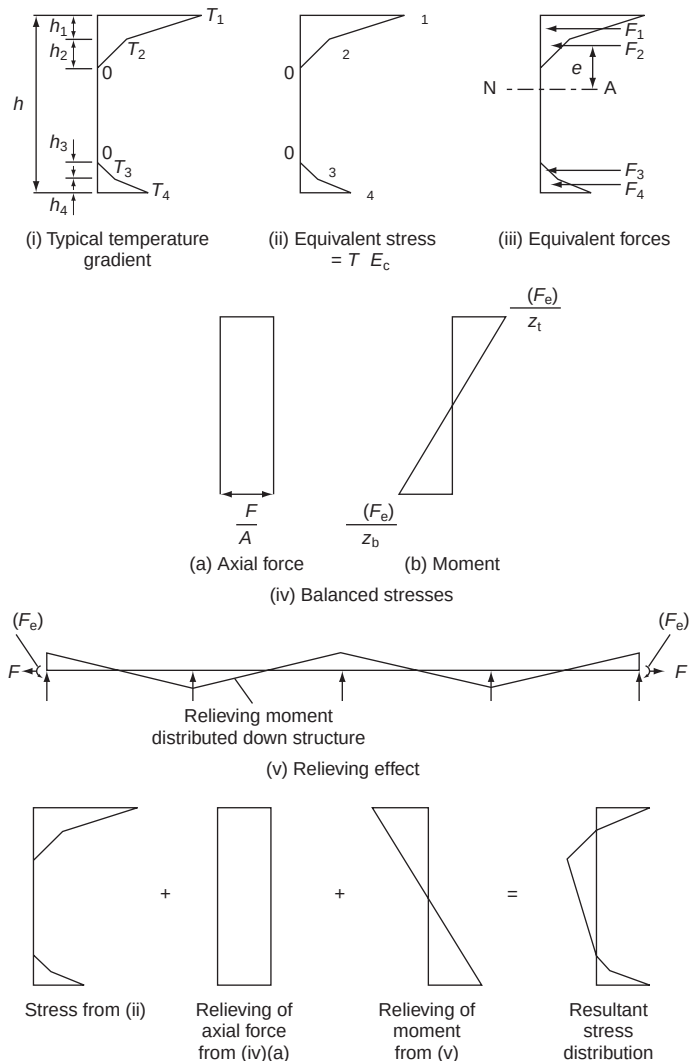


Figure 9.60 Differential temperature stresses

The moment effect gives rise to shear forces in the spans; these should be considered during the ultimate shear design where appropriate.

Analysis

Modern computer techniques are increasingly being used to analyse and design prestressed concrete bridges. The latest generation of software takes into account the 3D nature of bridges and combines the prestressing tendons with the structural analysis, as well as simulating the stage-by-stage construction and long-term effects, which is essential when considering the design of prestressed concrete bridges. Analysis using these

software packages includes all the prestress losses, creep and shrinkage of the concrete and combines it with the self-weight, superimposed dead loads, live loads and temperature loading to simplify the design process.

A typical model for a precast segmental box girder deck built by the balanced cantilever technique is shown in Figure 9.61, which was generated using the MIDAS software package. Several stages of the analysis are shown as the deck is built up and the prestress installed.

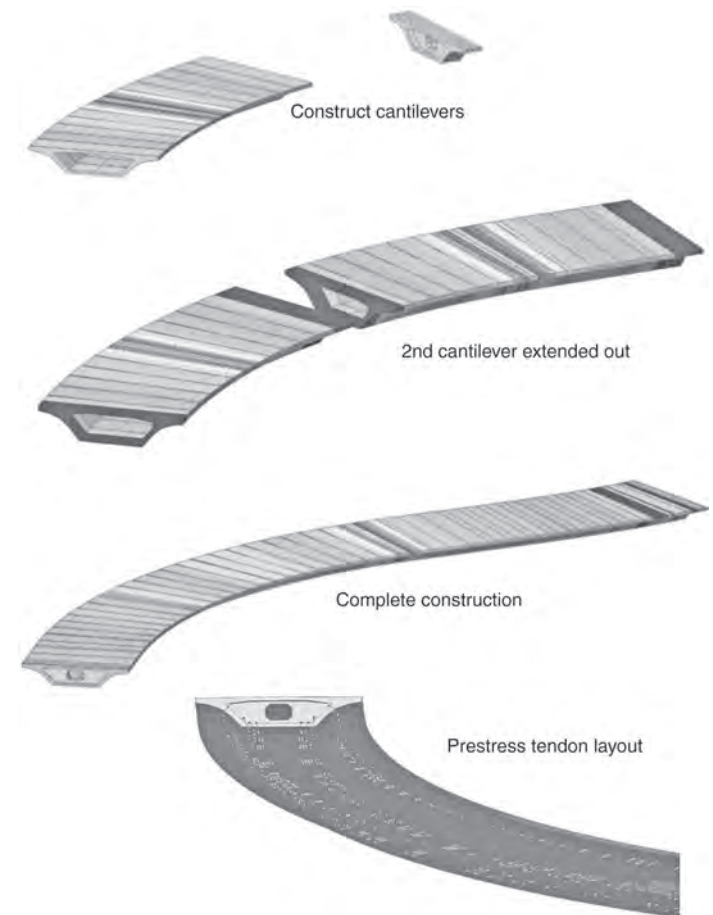


Figure 9.61 MIDAS analysis model for box girder deck

When using this sophisticated analysis software, care should always be taken to understand how the analysis works and to check the output to ensure that it is reasonable and correct for the design being undertaken.

Analysis and design of deck types

Voided slab decks

Analysis of a voided slab deck (Figure 9.5) is easily achieved with sufficient accuracy using a grillage model. It is convenient to place the longitudinal grillage members to match the layout of the prestress tendons or groups of tendons, with

transverse members placed over the supports and at suitable spacing between them, to give an evenly balanced grid pattern. The section properties of both the longitudinal and transverse members are based on the deck width to midway between the members, with the resultant voided section taken longitudinally and the section at the centre of the void taken for the full width transversely. This is normally found to give sufficiently accurate results for use in the design. When assigning the torsional stiffness to the grillage members, it is sufficient to give both longitudinal and transverse members the same value of twice the I value of the longitudinal members. The longitudinal prestress design is carried out by considering the bending moments and shear forces generated from the imposed loads on the longitudinal grillage members and designing the prestress to balance the serviceability limit state stresses, with checks to ensure adequate ultimate limit state strength in the normal way.

Beam-and-slab decks

The design of beam-and-slab decks (Figure 9.7) involves the superimposing of many different effects to build up the overall design state. Dead load, superimposed dead load and live load effects, both longitudinally in the beams and transversely in the deck slab, may be derived from a grillage analysis, which will give satisfactory results for most standard beam-and-slab arrangements. For the grillage analysis, the deck is modelled as a series of discrete members, both longitudinally and transversely. Longitudinally, it is simplest to provide a grillage member along the line of each beam web, with section properties to include a portion of the top slab up to midway to adjacent webs and, similarly, a portion of the bottom slab, where U-beams or box beams are used. Where box structures are present and are represented by individual grillage members for each web, the torsional stiffness of the box is calculated and a quarter of this is assigned to each web, with the remaining half of the torsional stiffness assigned to the transverse members. Transversely, grillage members are positioned at each diaphragm location, with section properties to match the diaphragm beam, and also at regular intervals between, with section properties to match the length of the top and bottom slabs between, where appropriate. The location of the transverse members should result in a ratio of the spacing of longitudinal members to transverse members of between 1:1 and 1:2 to achieve reasonably accurate load distribution.

Where unusual beam-and-slab arrangements are used or secondary effects, such as distortions or transverse bending in the beams, become significant, three-dimensional finite-element models should be used for deriving the load effects. Shell elements may be used to build up the webs, slabs and diaphragms of the three-dimensional structure, and the dead, superimposed and live load applied to the model to give the forces and moments at the critical sections for the beam design.

A full account of the construction and stressing sequence needs to be allowed for in the design. When the beams are cast, they are supported by the formwork along their total length;

however, on applying the prestress, the beams tend to lift up over their length, and when removed to the storage yard they are usually supported at the ends only, giving a typical stress profile at midspan, as indicated in Figure 9.62(i). At this stage, the beams are subject to their own dead weight and the applied prestress and the resulting stresses at the top and bottom of the beam may be accurately derived. The common problem at this stage is to ensure that the compression in the bottom and tension in the top of the beam do not exceed allowable limits. Once erected, the top slab is cast with the weight being carried by the beam section alone, as indicated in Figure 9.62(ii).

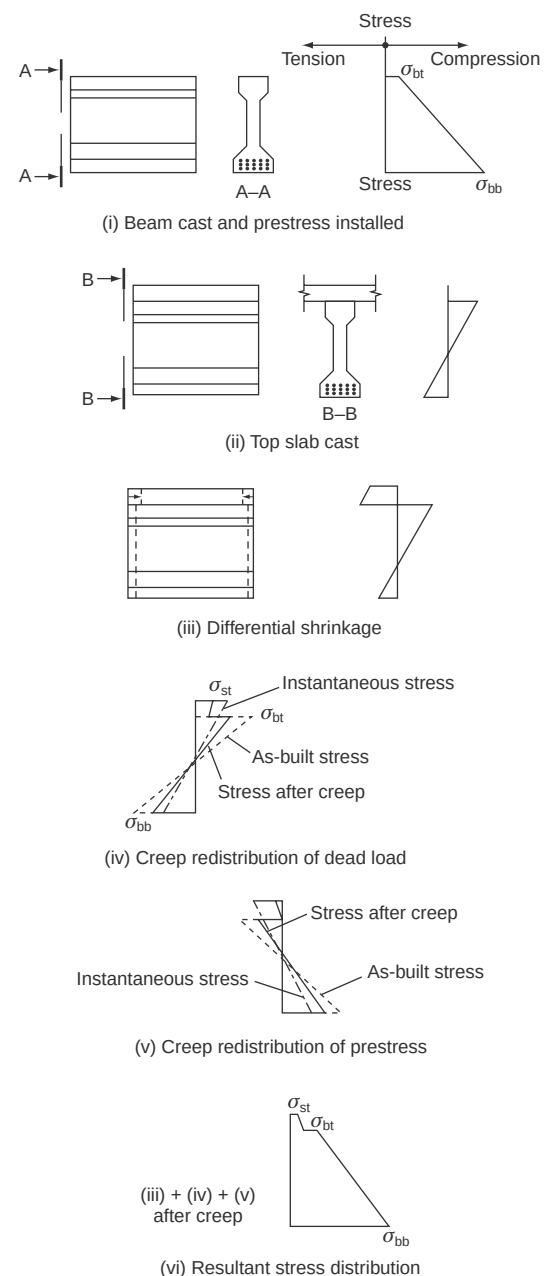


Figure 9.62 Longitudinal stress distribution in beams

When the top slab concrete has hardened, creep on the concrete will redistribute the dead load stresses and prestress forces across the section. The superimposed and live loads are carried by the combined section of the beam and top slab.

The creep redistribution of the dead load and prestress stresses may be estimated by considering the as-built condition, where all the load is carried on the beam and the stresses are a combination of Figures 9.62(i) and 9.62(ii), and the theoretical 'instantaneous' condition, where the full dead load and prestress is applied to the composite section. The stresses will creep from the as-built condition towards the instantaneous condition, as indicated in Figures 9.62(iv) and 9.62(v), with the final stress at any level in the section being given by

$$\sigma_{\text{final}} = \sigma_{\text{as-built}} + (1 - e^{-\phi})(\sigma_{\text{inst}} - \sigma_{\text{as-built}}) \quad (9.52)$$

The loss of prestress forces due to relaxation in the strand, and concrete creep and shrinkage that occur after casting of the top slab, will affect the stresses in both the slab and beam; these long-term prestress losses should be considered in two phases. For the first phase, before the deck slab is cast, the prestress losses should be estimated and the prestress force in the beam reduced accordingly. In the second phase, the remaining losses that occur after casting the slab should be estimated and applied as a 'tensile' force to the composite section at the position of the centroid of the strand or tendons.

When the top slab is cast, the concrete in the beam will already have completed a large proportion of its shrinkage, resulting in differential shrinkage occurring between the slab and beam. The deck slab tries to shrink more than the beam and is restrained by it, creating tension in the slab and a combined compressive force and sagging moment in the beam. The equivalent differential shrinkage force F_s , assuming the top slab was fully restrained, may be estimated as

$$F_s = \Delta_s A_s E_c [(1 - e^{-\phi})/\phi] \quad (9.53)$$

with $(1 - e^{-\phi})/\phi$ reducing the effect from creep of the concrete.

The stresses in the section may be derived by adding the 'fully restrained' stress to the 'released' stress in the composite section:

- tensile stress from force F_s acting on slab alone at the centroid of slab
- stresses generated from a compressive force F_s acting on the composite section, applied at the centroid of the slab, giving a stress profile similar to Figure 9.62(iii).

The stresses generated at the top of the slab, the top of the beam and the bottom of the beam, as indicated in Figure 9.64(vi), are checked at each stage of construction as well as in the permanent condition after opening and again after the long-term effects have occurred.

When the beams are fully connected longitudinally over the piers to make the deck continuous, the dead load and prestress effects will be redistributed, owing to creep of the concrete. The creep from the prestress load on the concrete may cause a hogging deflection of the beam at midspan and a sagging

restraint over the pier, which may lead to excessive cracks in the soffit in this region if the effect is not fully considered and suitable reinforcement provided. The final moment at the pier from the crept dead load and prestress may be derived by a similar approach to that described previously in this chapter, with

$$M_{\text{final}} = M_{\text{as-built}} + (1 - e^{-\phi})(M_{\text{inst}} - M_{\text{as-built}}) \quad (9.54)$$

where ϕ is based on the residual creep left in the beam at the time of casting the connection and $M_{\text{as-built}}$ is equal to zero where the connections are made after erecting the beam and casting the top slab. The moment calculated as if the deck were built 'instantaneously' as continuous, M_{inst} , should include the prestress secondary moment as well as the prestress primary moment and dead load. The differential shrinkage between the top slab and the beam will generate a secondary hogging moment in the deck at the pier after the connection has been made; this must also be considered in the design.

The design for ultimate moment, shear and torsion is carried out in the normal manner, while the check of the longitudinal shear along the interface between the slab and beam must include the forces generated by the differential shrinkage and creep effects.

During lifting and transporting of beams, they may be subjected to additional loads, resulting in a change to the bending moments, which must be considered in the design. Care must be taken during handling to minimise any impact or dynamic loading, while lifting or temporary support positions should be located as near to the permanent support position as possible. Where long slender beams are used, transverse instability may occur, either during lifting or when placing the concrete for the top slab; suitable restraint will need to be provided.

In situ multi-cell box girder decks

Where thin webs and slabs are being used, the box section may significantly deform and distort under the imposed loading, and the deck analysis is usually most accurately carried out using a three-dimensional finite-element model, with shell elements for each web and slab section. The moments and shears in each element in each direction may be taken directly from the model and used for both the longitudinal prestress design and the transverse reinforcement design. For the longitudinal design, the deck section is divided into a series of I-beams consisting of a web and associated top and bottom slabs up to the midpoint of each cell. Prestress layouts are then designed for each web to balance the stresses derived from the finite-element analysis and give adequate ultimate strength.

In situ single-cell box girder decks

Traditionally, for a single-cell box girder, the longitudinal and transverse designs have been considered separately, although it is now becoming more commonplace to establish three-dimensional finite-element models that combine the effects.

The deck may be modelled longitudinally as a two-dimensional or three-dimensional frame by standard structural analysis software, and the moments and shears derived for dead load, superimposed dead load and live load, as well as for secondary effects, such as support settlement. Creep effects of the dead load and the prestress and temperature effects may be determined as described earlier in the chapter.

Transversely, the box is subjected to bending and shear by the dead and superimposed dead loads and the live loads on the top slab. A three-dimensional finite-element model with shell elements should be set up, similar to that shown in Figure 9.63, covering sufficient length of the box to allow the longitudinal distribution of the live load and the three-dimensional behaviour of the box to be accurately modelled. Once the moments and shears around the box have been calculated, reinforcement is determined as necessary.

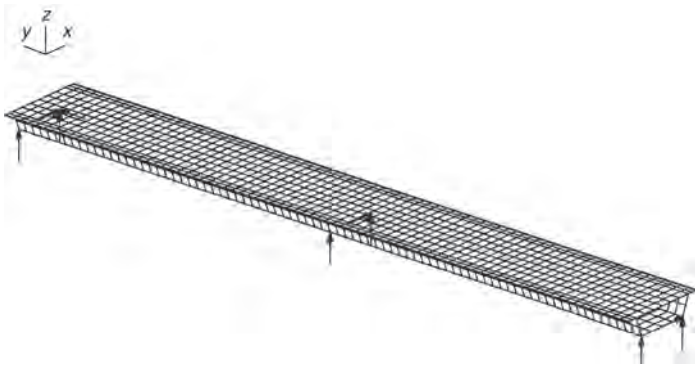


Figure 9.63 Typical finite-element model of box girder

Under torsional loading, concrete box girders warp, with their cross-sections undergoing out-of-plane displacements. The sections undergo torsional and distortional warping, while the applied loads may also cause distortion to the cross-section. These effects are fully described in the Cement and Concrete Association technical report on concrete box beams (Maisel and Roll, 1974). If a full three-dimensional finite-element model is set up over the full length of the deck, the warping effects may be directly modelled, along with both the longitudinal and transverse effects.

For normal-proportioned box girders, the prestress tends to balance the dead and live load moments; shear lag is not a problem and need not be considered further. For wider or slimmer decks with unusual proportions, shear lag may be significant and needs to be allowed for in the design. In these cases, the shear lag will modify the stress distribution across the section and peak stresses will need to be determined for use in the serviceability longitudinal stress check; however, shear lag need not be considered in the ultimate limit state. The shear lag effect may be determined from a full three-dimensional finite-element model of the structure, although care is needed in modelling the prestress forces to give an accurate representation of their effect.

Precast segmental box girder decks

Precast segmental boxes are usually very similar to in situ single-cell box girder decks, with the main difference being the joints between the segments.

With matchcast segments erected with epoxy in the joints, the design of the concrete box in its completed condition is similar to that for an in situ box girder. The presence of the joints creates a discontinuity in the longitudinal reinforcement and the joints should be kept in compression under all loading conditions. The epoxy creates a bond of greater strength than the concrete itself and the ultimate shear checks may be carried out as if the deck is monolithic; however, where designing to BS 5400 at wide, cast in situ, concrete joints, such as in situ stitches connecting the different parts of the deck, the shear force under ultimate loads should not be greater than

$$0.7(\tan\alpha)0.87P \quad (9.55)$$

where $\tan\alpha$ depends on the interface and, for roughened faces, may be taken as 0.7 during erection and 1.4 after completion of the bridge and P is the horizontal component of the prestress force after all losses.

The Eurocodes and AASHTO treat ultimate shear at wide, cast in situ, joints in the same way as interface shear, although normally the stitches do not have reinforcement continuous across the interfaces and the shear capacity is reliant on the compressive force present in the deck.

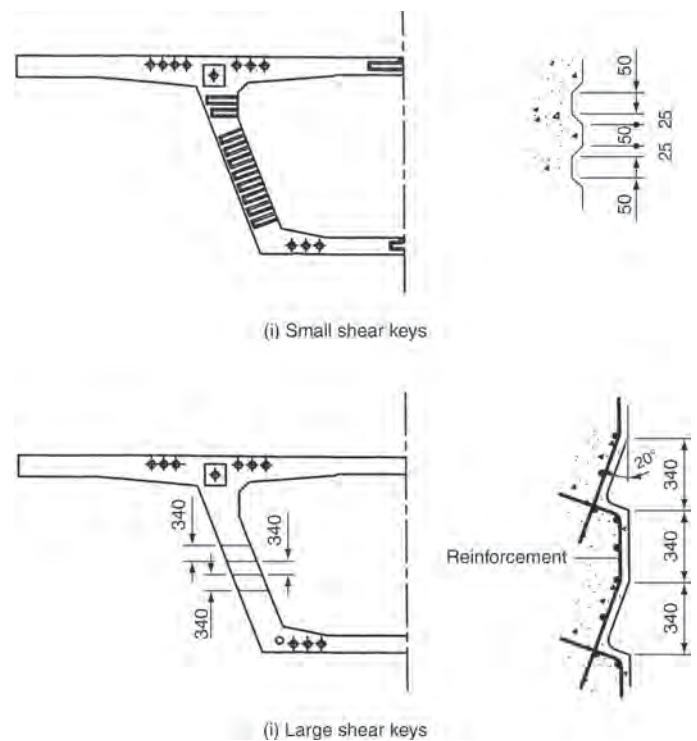


Figure 9.64 Shear key arrangement

Shear keys are provided on the surface of the joint to help align the segments during erection and to transfer the shear forces before the epoxy has set. Either large or small shear keys may be used, as shown in Figure 9.64; both are designed to prevent a shear failure under the temporary erection loads.

Considering the shear friction concept as given in AASHTO (2007), the capacity of the shear key is estimated using a coefficient of friction of 1.4, resulting in a shear resistance of

$$V_k = 1.4[A_{sk}f_{pk} + A_r(0.87f_y)] \quad (9.56)$$

Standard BS 5400 (BSI, 1990) does not cover the situation associated with the design of the shear keys, while the Eurocode (BSI, 2004, 2005) takes them into account when assessing the interface shear capacity, although in the temporary condition, before the epoxy has set, the cohesion on the interface should be taken as zero.

The large shear keys may be designed as a reinforced concrete corbel where appropriate reinforcement is provided; for the small shear key, reinforcement is not normally required.

Incrementally launched box girder bridges

For incrementally launched box girder bridges, the design needs to cater for the range of moments and shears occurring at each deck section during the launching procedure. As the deck moves forward over the piers, each section is subject to a change in moment and shear. At some stage, each section of deck is over a pier, and subject to hogging moments, or in midspan, and subject to sagging moments, and the prestress design needs to cater for the full range of moments, as indicated in Figure 9.65. During launching, a first-stage prestress is usually arranged to give a constant compressive stress across the section, normally of the order of 5 MPa. This prestress would often consist of straight tendons placed in the top and bottom slabs, anchored on the construction joint between segments and coupled or lapped to extend through each new length of deck cast. After completion of launching, a second-stage prestress is installed with a draped profile along the spans to balance the bending moment profile generated by the deck in its final position.

The full length of the box section also needs to be strong enough to resist the shear forces and the temporary bearing load under the webs, as the deck passes over the piers. The webs are normally kept a constant thickness and the web or bottom slab corners stiffened throughout to distribute the local loads from the temporary bearings during the launch.

Unevenness of the concrete surface and differential settlement of the piers and temporary supports generate additional moments and shears in the deck during the launching operation; these need to be allowed for in the design.

As the deck is launched, the friction in the temporary bearings applies a load to the top of the piers. The temporary bearings are aligned parallel to the deck, rather than horizontal

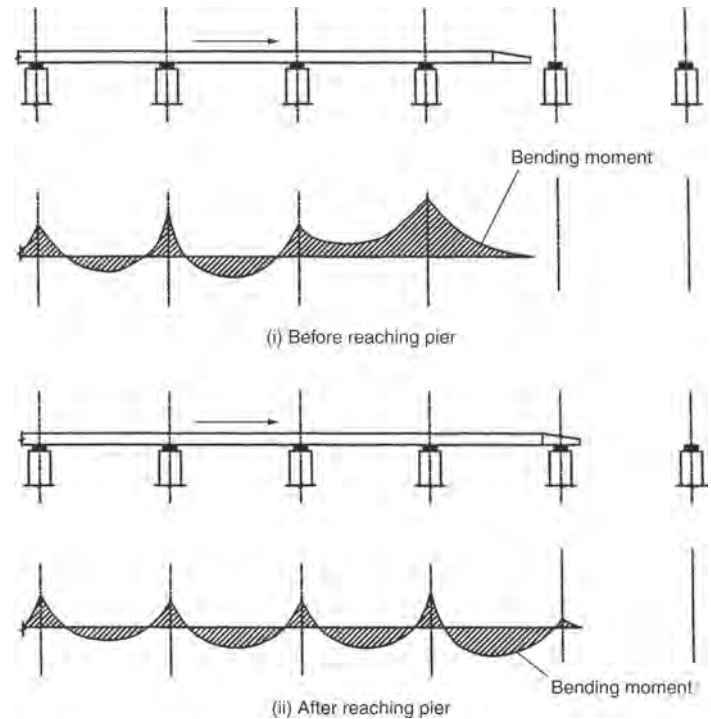


Figure 9.65 Bending moment range in deck during launching

like the permanent bearings, and this induces a further horizontal load on the pier. The piers must be designed to resist these horizontal loads in combination with the coexistent vertical loads, although the effects may be reduced by providing stays or guys to the top of the piers.

Design of details

Anchorage

With pre-tensioned strands, the force transfer into the concrete is achieved through the bond between the two materials. At the end of the strand, it slips into the concrete as the bond gradually builds up the force transferred, until the total force in the strand has been taken up by the concrete over a transmission length L_t .

When designing to BS 5400, with $F_o \leq 75$ UTS and $f_{ci} \geq 30$ MPa, this length is defined as

$$L_t = k_t D_t / f_{ci} \quad (9.57)$$

where k_t can be taken as 600 for plain, indented or crimped wire with a wave height of less than $0.15D_t$, 400 for crimped wire with a total wave height greater than $0.15D_t$, 240 for 7-wire standard and super-strand or 360 for 7-wire drawn or compacted strand.

With the Eurocodes, a similar formula is adopted, although more variables are considered, to give

$$l_{pt} = \alpha_1 \alpha_2 \phi \sigma_{pm0} / f_{bpt} \quad (9.58)$$

$$f_{bpt} = \eta_{p1} \eta_{l1} f_{ctd(t)} \quad (9.59)$$

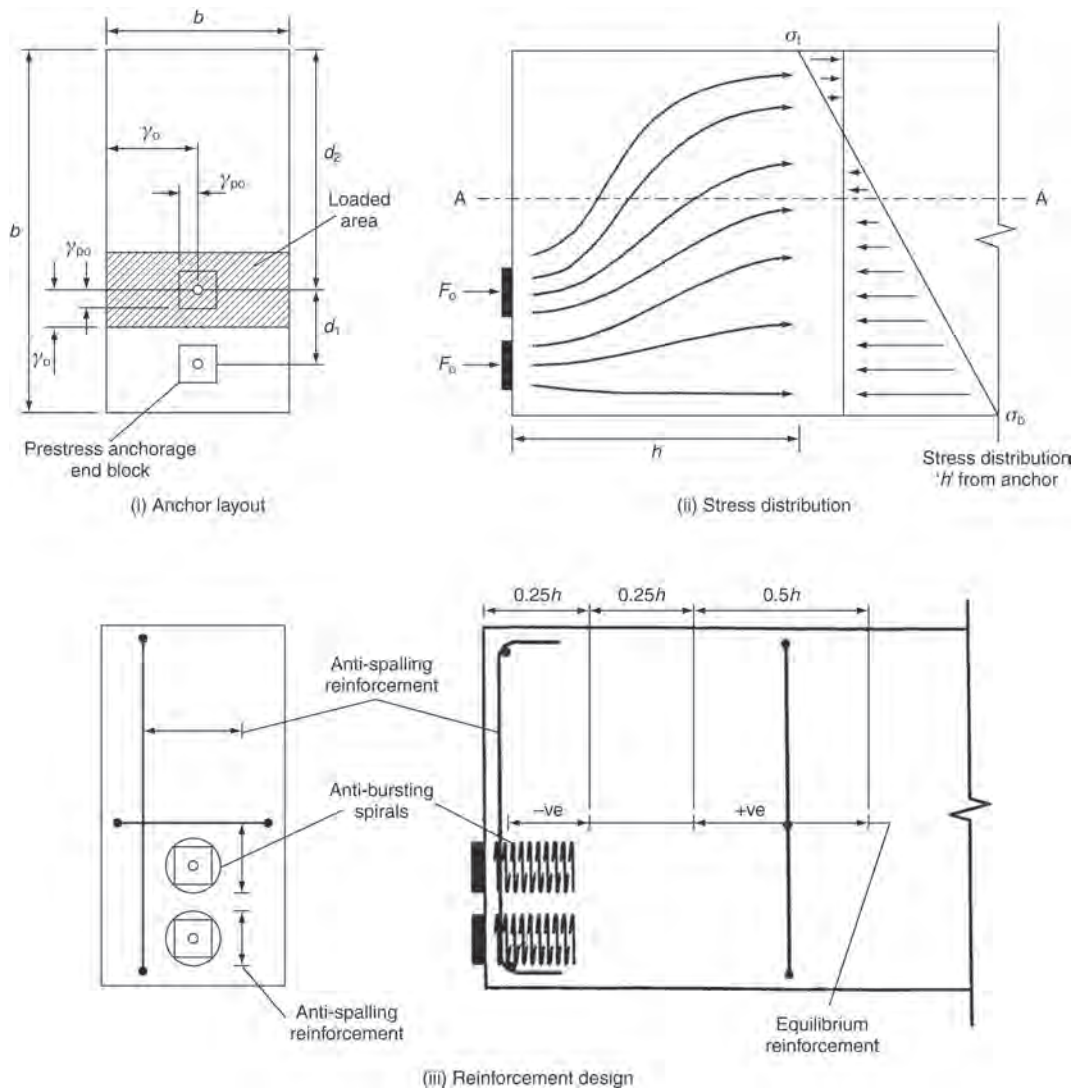


Figure 9.66 End block design

The worst case of 0.8 or $1.2l_{pt}$ is taken for the design, while α_1 is 1.0 for gradual release and 1.25 for sudden release; α_2 is 0.19 for 7-wire strand; η_{p1} is 3.2 for 7-wire strand and η_1 is 1.0 for good bond condition and 0.7 otherwise.

When post-tensioned tendons are anchored, they apply a large concentrated force to the concrete, which needs to be contained. CIRIA Guide No 1 (Clark, 1976) describes the behaviour and design of anchor blocks, with a typical arrangement as shown in Figure 9.66. Behind each anchor, splitting forces and tensile stresses occur and reinforcement needs to be provided as follows.

Bursting reinforcement is provided as a spiral or series of links around each individual anchor, with the area of reinforcement given as

$$A = F_{bst}/0.87f_y \quad (9.60)$$

where F_{bst} depends on the end block arrangement and is given by

$$y_{po}/y_o: \quad 0.3 \quad 0.4 \quad 0.5 \quad 0.6 \quad 0.7$$

$$F_{bst}/F_o: \quad 0.23 \quad 0.20 \quad 0.17 \quad 0.14 \quad 0.11$$

and the denominator, $0.87f_y$, should be replaced by a stress of 200 MPa or less to control cracking when the concrete cover to the reinforcement is less than 50 mm.

Bursting reinforcement is placed between $0.2y_o$ and $2y_o$ from the anchor face and encloses a cylinder or prism with dimensions 50 mm larger than the face of the anchor block. The bursting forces in the two principal directions should be determined and sufficient reinforcement provided to suit. The loaded area used for determining y_o is taken as symmetrical about the anchor, extending to the nearest edge of the concrete or to the midpoint of any adjacent anchors.

Spalling reinforcement is provided to prevent concrete spalling off the end face around the anchor, with the area of reinforcement given as

$$A = 0.04F_o / 0.87f_y \quad (9.61)$$

The stress in the reinforcement ($0.87f_y$) should be kept to below 200 MPa to control cracking, while the bars should be placed as near to the end face as possible and anchored around the concrete edges.

Where the anchor is positioned non-symmetrically on an end face, additional spalling stresses are set up and additional reinforcement is provided in the unsymmetrical direction, such that

$$A^1 = 0.2[(d_1 - d_2)/(d_1 + d_2)](F_o / 0.87f_y) \quad (9.62)$$

Equilibrium reinforcement is provided to maintain the overall equilibrium of the end block within the bridge deck arrangement. The force from the anchor block is assumed to have fully spread out at a distance h from the anchor face. By considering the concrete block with the anchor force on one side and the stress distribution on the other, the equilibrium of any horizontal plane, such as A–A in Figure 9.66(ii), is checked and vertical reinforcement provided to resist the out-of-balance moment, using a lever arm of $h/2$ to calculate the quantity of reinforcement needed. The reinforcement is then placed over the distance $0.25h$ or $0.5h$, as indicated in Figure 9.66(iii), depending on the direction of the out-of-balance moment.

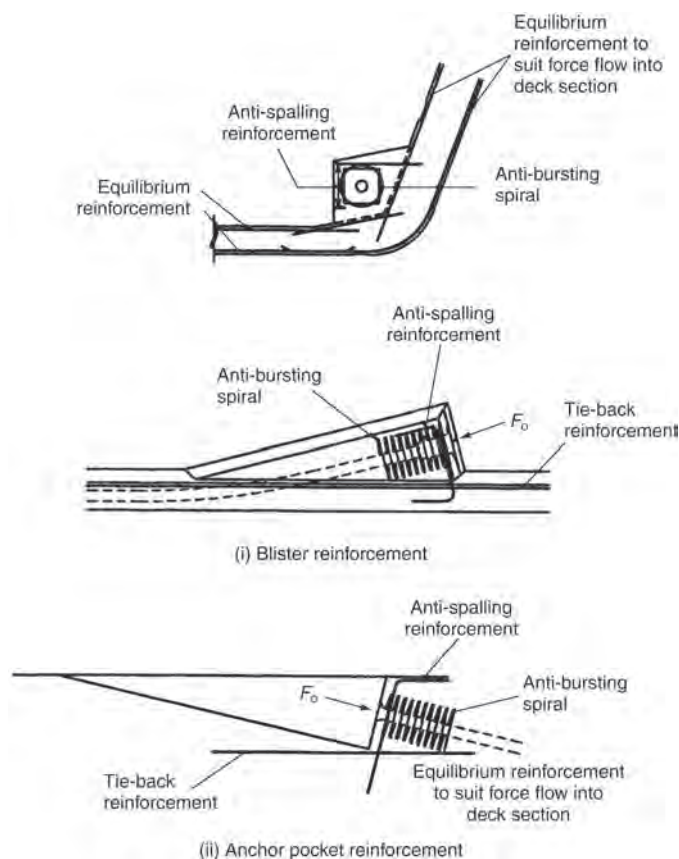


Figure 9.67 Anchor reinforcement

The shear on each horizontal plane should also be checked to ensure that the shear stress does not exceed

$$(2.25 + 0.65pf_y) \text{ MPa} \quad (9.63)$$

where p is the ratio of the reinforcement crossing the plane, and is taken as A_s/bh .

When designing to Eurocode, the local reinforcement for a post-tensioned tendon anchorage is often prescribed by the manufacturer and based on the European Technical Approval Guidelines (ETAG) and the arrangements adopted in the approval process. The containment of the spread of forces can be based on a strut and tie model or CIRIA Guide No 1 (Clark, 1976) could be adopted as an appropriate representation.

Anchor blisters

Figure 9.67(i) shows the additional reinforcement needed to tie the tendon into the main body of concrete where anchors are placed on blisters, or concrete blocks are cast on the side of the concrete member.

The bursting and spalling reinforcement quantities are calculated as for standard anchors, but the additional tie-back reinforcement is required to prevent cracks occurring behind the anchor, owing to the tensile forces generated to achieve strain compatibility in the concrete around the blister or anchor block. It has been traditional to provide tie-back reinforcement to cater for 50% of F_o , although finite-element analysis can show that significantly less reinforcement than this is needed in some cases and it is becoming more common to provide reinforcement to cater for 25% of F_o .

Equilibrium effects occur as the force distributes across the section and into the webs and slabs, requiring reinforcement to be provided, as described previously.

Anchor pockets

Figure 9.67(ii) shows an arrangement where recesses or pockets are provided in the concrete to anchor the tendons. The anchor forces tend to cause cracking of the concrete behind the recesses and similar tie-back reinforcement should be provided as for the anchor blisters, as well as the normal bursting and spalling reinforcement, while equilibrium reinforcement is again needed in the webs and slabs to restrain the force as it spreads out. The use of anchor pockets on the top surface of decks is discouraged in some countries, owing to concerns over the ingress of water through the resultant construction joint; if used, they should be well constructed and fully protected with additional waterproofing measures.

Ducts

Duct sizes are governed by the practicalities of having space to thread the tendon through and to allow the grout to flow freely

around the strands or bars, with the area of a duct normally not less than twice the tendon area.

With internal tendons, the ducts have to be supported and held in place during the concreting operation. This is normally done by installing additional reinforcement around the duct and fixing this to the normal reinforcement cage. The spacing of those supports will depend on the duct size and stiffness and is normally between 0.5 and 1.0 m.

To ease concrete placing and prevent delaminating of the adjacent concrete, ducts cast into the concrete should have a minimum clear spacing of not less than the greater of the following:

- duct internal diameter
- aggregate size + 5 mm
- 35 mm.

The concrete cover to the duct should not be less than 50 mm, although this depth may need to be increased when the ducts are curved, to prevent the tendon bursting out of its concrete.

With curved tendons, the strand presses against the edge of the duct and exerts a pressure on the concrete, which causes splitting forces that must be considered. The splitting force on the concrete depends on the force in the tendon and the radius of the curve. When the radius is large enough, no additional reinforcement is needed as the tensile strength of the concrete is sufficient to counter the forces generated. Where the radius of the duct is significantly less than the value given in Tables 36 and 37 of BS 5400 Part 4 (BSI, 1990) for the tendon force and cover or duct spacing, reinforcement should be provided to restrain the tendon, as shown in Figure 9.68.

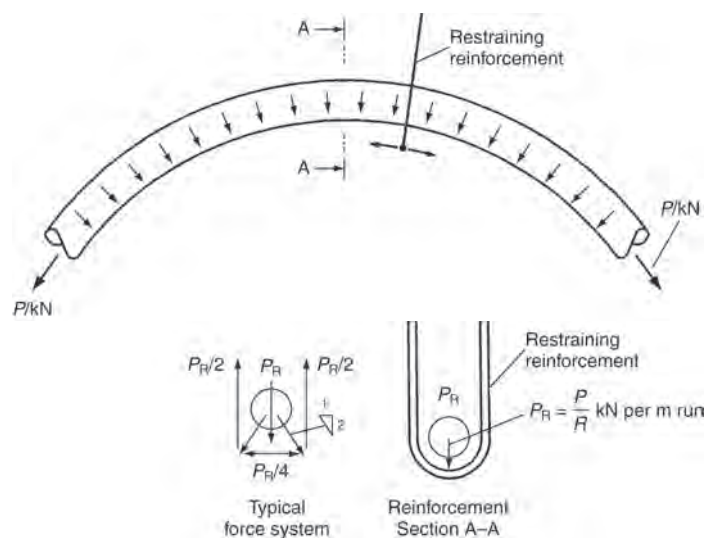


Figure 9.68 Restraint of curved tendon

Where a curved tendon exerts a force on the deck section, it is sometimes necessary to provide additional reinforcement to counter the bursting and bending effects, such as where

the tendons placed inside the bottom slab of a haunched box girder give a resulting downward force, as indicated in Figure 9.69. The tendon will tend to pull out of the concrete section and needs to be tied back, while it also generates transverse bending and shears in the bottom slab and tension in the webs, which need to be reinforced against.

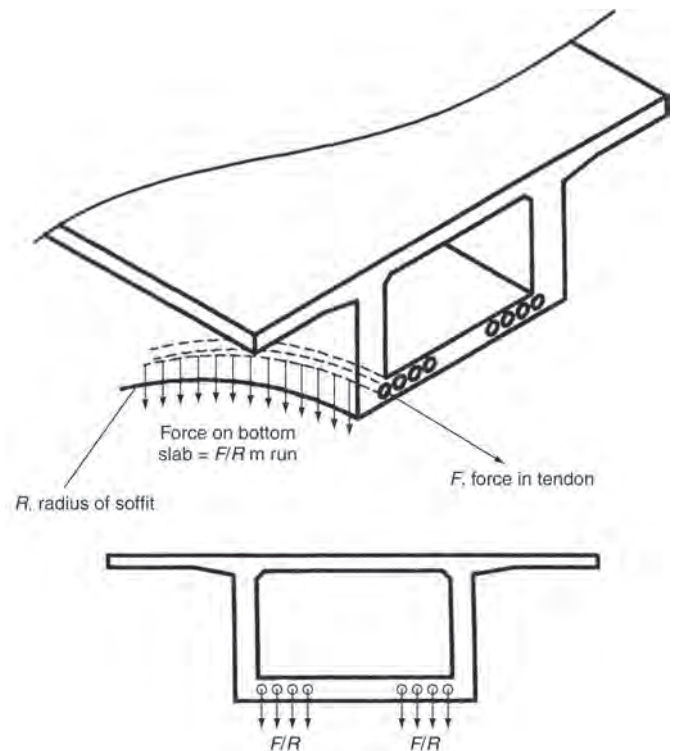


Figure 9.69 Force on curved soffit

Where the ducts are curved, the tendon will move to the edge of the duct, resulting in the tendon centre line being offset from the duct centre line. This offset will vary depending on the tendon and duct sizes and arrangement and must be taken into account when considering the tendon eccentricity in the longitudinal prestress design.

Typical design offsets and duct radii for tendons are indicated in Table 9.3. With strands, the tendons can follow a profile with fairly tight curves, the minimum radii often being governed by the ability of the duct to bend. Individual strands can be bent to radii as little as 0.5 m.

Typical tendon size: mm	Duct size internal diameter: mm	Typical tendon offset: mm	Minimum radii for internal ducts: m	Minimum radii for external ducts: m
7 × 15 strand	65	10	3	2.5
19 × 15 strand	90	17	5	3.0
27 × 15 strand	110	20	7	5.0

Table 9.3 Typical duct details

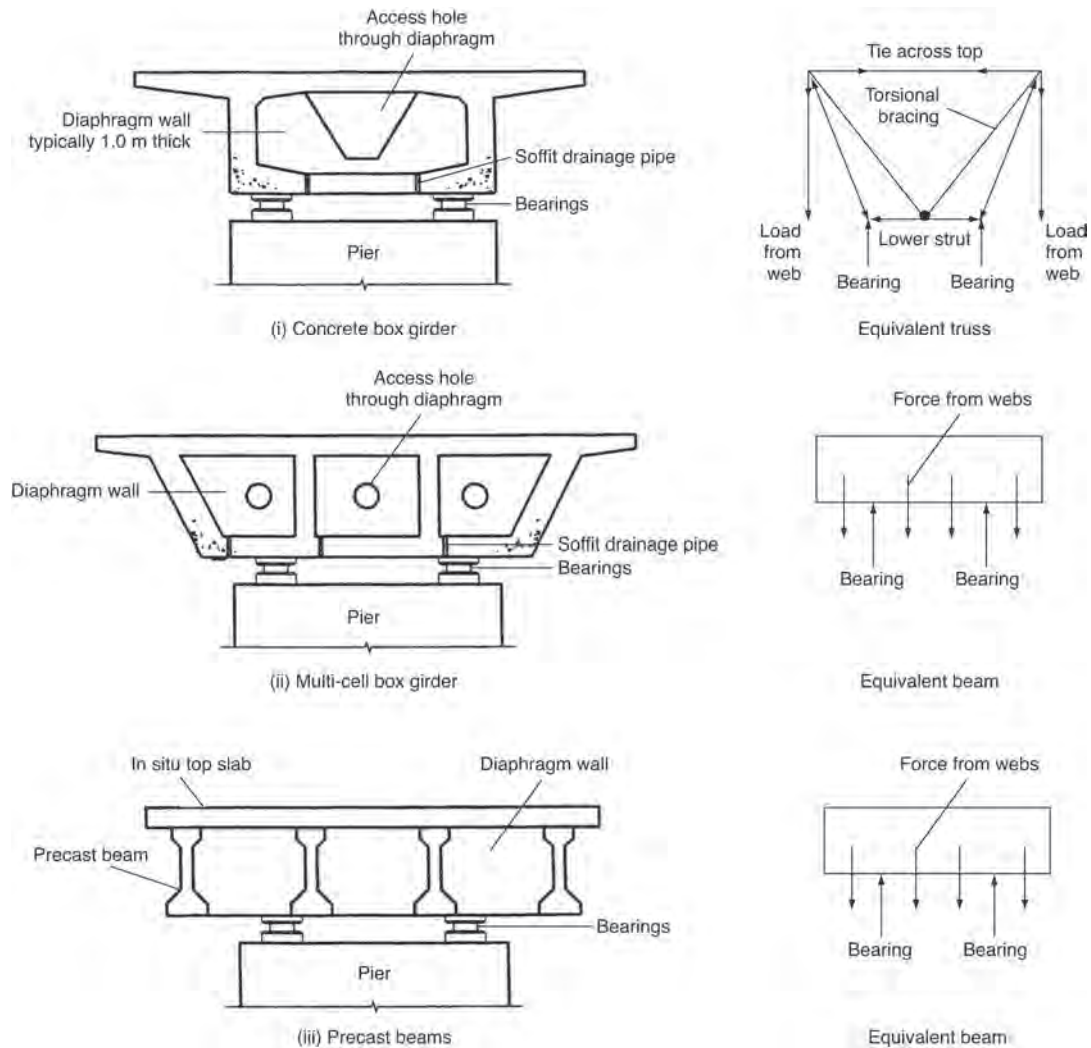


Figure 9.70 Diaphragm arrangements.

Diaphragms

Diaphragms are generally used in the deck at points of support to transfer the load from the webs into the substructure beneath. Typical diaphragm arrangements are shown in Figure 9.70. For the arrangement shown in Figure 9.70(i), a truss analogy is normally used to model the force transfer from the webs into the bearings. To ensure that all the load is picked up and transferred to the truss at the top of the webs, ‘hanging reinforcement’ needs to be provided in the form of vertical bars in the web and diaphragm. The vertical force is then held by a ‘tie’ across the top of the diaphragm and a strut towards the bearing. Where significant torsion exists in the deck, it is also necessary to consider the horizontal forces present in the top and bottom slabs, and provide reinforcement to tie these together.

In Figures 9.70(ii) and 9.70(iii), the diaphragm will behave as a beam and bending moments and shears can be determined and reinforced against in the normal way. Where a web is

directly over a bearing, the load will go straight into the support; elsewhere, ‘hanging reinforcement’ will be needed to take the force from the web up to the top of the diaphragm beam.

The concrete above the bearings needs to contain the high loads applied and can be designed in a similar manner to an anchor end block, with spalling and bursting reinforcement provided accordingly.

Deviators

Where deviators are used to deflect external tendons to give the desired profile, they can be subject to large forces that need to be tied into the deck section. Figure 9.71 shows several different types of deviator arrangement that can be used. The force applied to the deviator is equal to the force in the tendon multiplied by its ‘angle change’ (in radians) and the deviator should be designed to cater for this load. In the UK, where BD 58/94

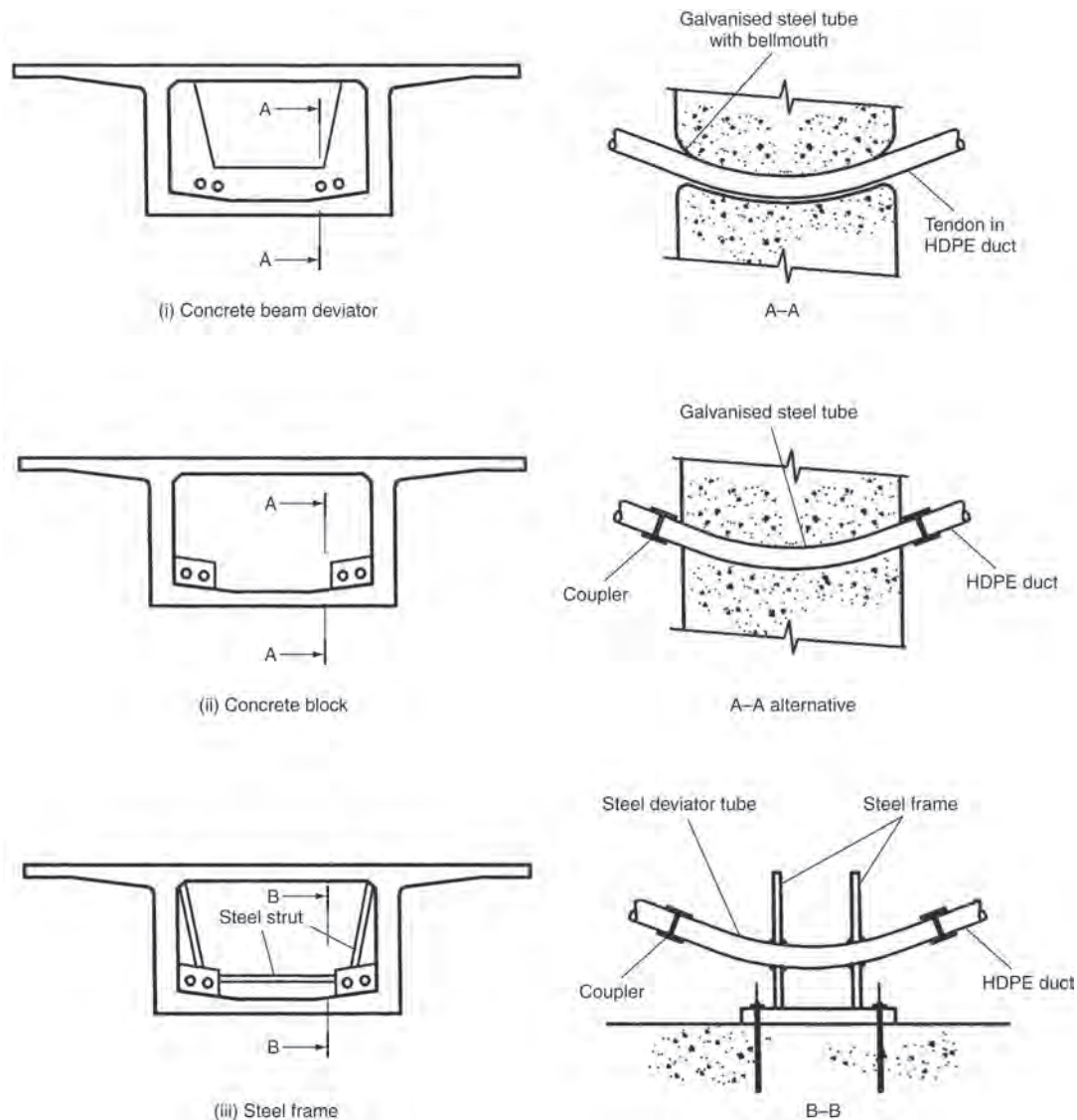


Figure 9.71 Deviator arrangements

(Highways Agency, 1994) is used, the design is carried out at the ultimate limit state, with the applied ultimate force based on the characteristic strength of the tendon. Elsewhere, the design is often based on the direct load applied by the tendons at the serviceability limit state.

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Appendix I. Definitions

Bonded tendon	The tendon is bonded to the concrete member, by being cast in as for pre-tensioning or through grout for post-tensioning.
External tendon	The tendon is placed outside the concrete section.
Internal tendon	The tendon is placed within the concrete section.
Post-tensioning	The tendon is stressed after the concrete has hardened, with load transfer during the stressing operation.
Prestressing tendon	A wire, strand, bar or bundle of strands used to prestress the concrete.
Pre-tensioning	The tendon is stressed before placing the concrete, with load transfer occurring after the concrete has set.
Unbonded tendon	The tendon is not connected to the concrete other than at the anchor positions, allowing the tendons and concrete to act independently along the tendon length.

Appendix II. Notation

A or A_1	area of reinforcement to be provided
A_c	area of concrete section
A_{cv}	area on concrete engaged in interface shear ($=b_{vi} \times L_{vi}$)
A_L	area of concrete considered for longitudinal shear check
A_o	area enclosed by median wall lines around the box
A_p	area of non-prestressed reinforcement
A_r	area of reinforcement across failure plane
A_s	area of slab
A_{sk}	area of shear key
A_{sl}	area of longitudinal reinforcement
A_{st}	area of leg of link around section
A_{sv}	area of shear reinforcement
A_t	cross-sectional area of tendon
A_{vf}	area of interface shear reinforcement
b	breadth of member or web
b_s	width of slab

b_{vi}	shear interface width
C	compressive force generated in concrete at ultimate moment capacity
D_t	nominal diameter of tendon
d	distance from tendons to compression face
d_c	depth of compression in concrete at ultimate moment capacity
d_t	distance from reinforcement to compression face
d_1	larger dimension from line of action of anchor force to the boundary on non-symmetrical prism
d_2	smaller dimension from line of action of anchor force to the boundary on non-symmetrical prism
E_c	28 day secant modulus of elasticity of concrete
E_t	modulus of elasticity of tendon
F	force in tendon at point being considered
F_{bst}	anchor bursting force
F_o	force applied by jack at anchor
F_s	force generated in top slab due to differential shrinkage
f_{bpt}	bond strength for pre-tensioned strand
f_c	stress in concrete at point considered
f_{ci}	cube strength of concrete at age being considered
f_{ck}	28 day cylinder strength of concrete
$f_{ck,cube}$	28 day cube strength of concrete
f_{cp}	compressive stress from prestress at centroid
$f_{ctd(t)}$	design tensile strength at time of release of pre-tensioned strand
f_{ctm}	tensile strength of concrete
f_d	stress due to unfactored dead load at tensile face of section subject to M_{cr}
f_p	initial stress in tendon
f_{pc}	stress due to prestress only at the centroid of the tendons
f_{pe}	effective prestress after all losses
f_{pi}	stress increase in tendon
f_{pk}	average compressive stress over shear keys
f_{pt}	stress due to prestress only at the tensile face
f_{pu}	characteristic strength of tendon
f_s	initial stress in non-prestressed reinforcement
f_{si}	increase in stress in non-prestressed reinforcement
f_t	tensile strength of concrete, taken as $0.24\sqrt{f_{cu}}$
f_y	characteristic strength of reinforcement
f_{yl}	characteristic strength of longitudinal reinforcement
f_{yv}	characteristic strength of link reinforcement
h	overall depth of member
h_{max}	larger dimension of the section
h_{min}	smaller dimension of the section
h_{wo}	web or slab thickness
I	second moment of area of section
K_L	longitudinal shear coefficient
k	wobble coefficient
k_i	concrete bond coefficient
k_t	coefficient dependent on type of tendon
L	length of deck
L_s	width of longitudinal shear failure plane
L_{sp}	length of span of deck

L_T	free length of tendon	V_d	shear force at section due to unfactored dead load
L_t	transmission length for anchorage of pre-tensioned strand	V_k	ultimate shear resistance of shear key
L_{vi}	shear interface length	V_L	longitudinal shear force per unit length
l	lever arm at ultimate moment	V_{ni}	nominal interface shear resistance
l_c	losses of stress in tendon due to creep of concrete	V_p	vertical component of prestress
l_E	losses of stress in tendon due to elastic shortening of concrete	V_s	ultimate shear resistance provided by reinforcement
l_{pt}	basic value of transmission length for pre-tensioned strand	ν	shear stress in concrete due to ultimate loads
l_s	losses of stress in tendon due to shrinkage of concrete	ν_c	ultimate shear stress allowed in concrete
M	moment at section due to ultimate loads	ν_L	ultimate longitudinal shear stress in concrete
M_a	moment generated by change applied 'instantaneously'	ν_t	torsional shear stress
$M_{as-built}$	moment as constructed	x	distance of point being considered from tendon anchor
M_{cr}	cracking moment of section	x_i	smaller centre-line dimension of torsion link
M_{final}	final moment after creep effects	Y or y'	distance in section from N–A to point being considered
$M_{inst.}$	moment if the structure is built instantaneously	y_i	larger centre-line dimension of torsion link
M_o	moment necessary to produce zero stress in concrete at tensile face	y_o	half length of side of anchor block
M_p	primary moment from prestress on section	y_{po}	half length of side of loaded area
M_r	ultimate moment of resistance at a section	y_t	distance in section from N–A to tensile face
M_s	prestress secondary moment	z_b	elastic sectional modulus referred to bottom face
m_d	mass per metre run of deck	z_t	elastic sectional modulus referred to top face
m_t	mass per metre run of tendon	α	coefficient of thermal expansion of the concrete (°C)
m_1	moment due to unit restraint moment applied at pier 1	γ_m	partial safety factor for strength
m_2	moment due to unit restraint moment applied at pier 2	γ_s	partial factor for reinforcing or prestressing steel
N–A	neutral axis of section	Δ_l	change in deck length due to Δ_t
P	total unfactored prestress force acting on section	Δ_s	differential shrinkage strain between in situ slab and precast beam
P_c	compressive force normal to interface plane	Δ_t	change in effective temperature
P_h	horizontal force from prestress tendon	ϵ_{cb}	strain in concrete at bottom fibre
P_v	vertical force from prestress tendon	ϵ_{ct}	strain in concrete at top fibre
p	ratio of reinforcement, A_r/bh	ϵ_{cs}	shrinkage strain deformation of the concrete
S_l	spacing of longitudinal reinforcement	ϵ_p	strain in prestress tendon
S_v	spacing of link reinforcement	ϵ_s	strain in non-prestressed reinforcement
T	tensile force generated at ultimate moment	θ	total angle change (in radians) in the tendon over distance x
T_s	torsional moment due to serviceability loads	μ	friction coefficient
T_u	torsional moment due to ultimate loads	ξ_s	depth factor for shear, given in Table 9 of BS 5400 Part 4 (BSI, 1990)
V	shear force due to ultimate loads	$\sigma_{as-built}$	stress in section due to construction sequence
V_c	ultimate shear resistance of concrete at section	σ_c	stress in concrete adjacent to prestress tendon
V_{co}	ultimate shear resistance of concrete uncracked in flexure	σ_{final}	final stresses in section
V_{cr}	ultimate shear resistance of concrete cracked in flexure	σ_{inst}	stress in section if built instantaneously
		σ_{pm0}	tendon stress at time of release
		ϕ	creep factor
		ϕ	AASHTO strength reduction factor
		ϕ	nominal diameter of strand

Chapter 10

Steel bridges

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Steel has been used extensively over the last 120 years to construct a large majority of the world's greatest bridges. This chapter, on the design of steel bridges, gives an introduction to the different types of steel bridges in use, followed by guidance on the behaviour of steel plates loaded in both shear and compression. Careful attention is given to assist understanding of the load displacement behaviour of stiffened plates, which are the fundamental components of both plate girders and box girders. Steel plate girders are used extensively in the construction of medium-span bridges and steel box girders form the decks of all modern, innovative suspension bridges. The behaviour and design of bolted and welded connections is also included in this chapter, along with guidance on the fatigue appraisal of steel bridges.

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Introduction

Structural steel is an extremely versatile material, eminently suited for the construction of all forms of bridges. The material, which has a high strength-to-weight ratio, can be used to bridge a range of spans from short through to very long (15–1991 m), supporting the imposed loads with the minimum of dead weight.

Steel bridges normally result in light superstructures, which in turn lead to smaller, economical foundations. They are normally prefabricated in sections in a factory environment under strict quality control, transported to site in manageable units and bolted together in situ to form the complete bridge structure. Using this construction method, the erection of a steel bridge is usually rapid, resulting in minimal disruption to traffic, a very important factor if traffic delays, be it road or rail, are properly assessed in the construction project.

Rolled steel sections, the largest manufactured in the UK being a 1016 mm × 305 mm × 487 mm (486.7 kg/m) universal beam, are economical for short-span highway bridges and, when designed to act compositely with a concrete deck, are capable of spanning 25–30 metres. For spans in the range of about 25–100 m, plate girders, again acting compositely with the deck, provide an economical solution. To optimise the concrete deck, which has to distribute wheel loads transversely across the bridge, it is usual to arrange for a plate girder spacing of around 3 m. For longer spans, exceeding 100 m, box girders are the favoured choice. Although box girders have a higher fabrication cost than plate girders, box girders have substantially greater torsional stiffness and, if carefully profiled, good aerodynamic stability. For very long spans, in excess of 250 m, stiffened steel box girders with an integral orthotropically stiffened steel top plate, forming the primary support for the running surface, provide a very economical lightweight solution. Figure 10.1 gives cross-sections taken through typical bridge structures using hot rolled, plate girder and box girder sections.

Truss bridges

Lattice truss structures have been used very successfully for both railway and highway bridges throughout the last 150 years. There are three main truss configurations in use today, namely, the Warren truss, the modified Warren truss and the Pratt truss, each of which can be used as an underslung truss, a semi-through truss or a through truss bridge. Figure 10.2 gives details of the three truss types, together with sections showing the differences between underslung, semi-through and through trusses.

With an underslung truss, as shown in Figure 10.2, the live loading from either the passage of vehicles or trains is carried directly by the top chord of the truss. Underslung trusses are used almost exclusively for rail bridges in situations where the depth of construction or clearance under the bridge is not critical. In semi-through trusses, vehicles or trains pass through the truss bridge but, owing to the height of the vehicles or trains relative to the upper chord, the transient live load projects above the top chord members. Consequently, in semi-through trusses, it is not possible for the top chords to be braced laterally and these chord members must obtain lateral stability from U-frame action, described later in the chapter. In through truss bridges, as shown in Figure 10.2, the vehicles or trains pass through the centre of the truss bridge and the clearance between the live load and the top chords is such that it is possible to brace the top chord members laterally.

Provided a truss bridge is detailed so that the live loading is effectively applied at the nodes of the structure, the members in the truss will primarily carry only axial tension or axial compression forces. The global bending moment acting on the bridge can be resolved into a couple formed from compression forces acting in the top chord and axial tension forces acting in the bottom chord. Similarly, the global shear force acting on the truss bridge is carried by the diagonal web members, again

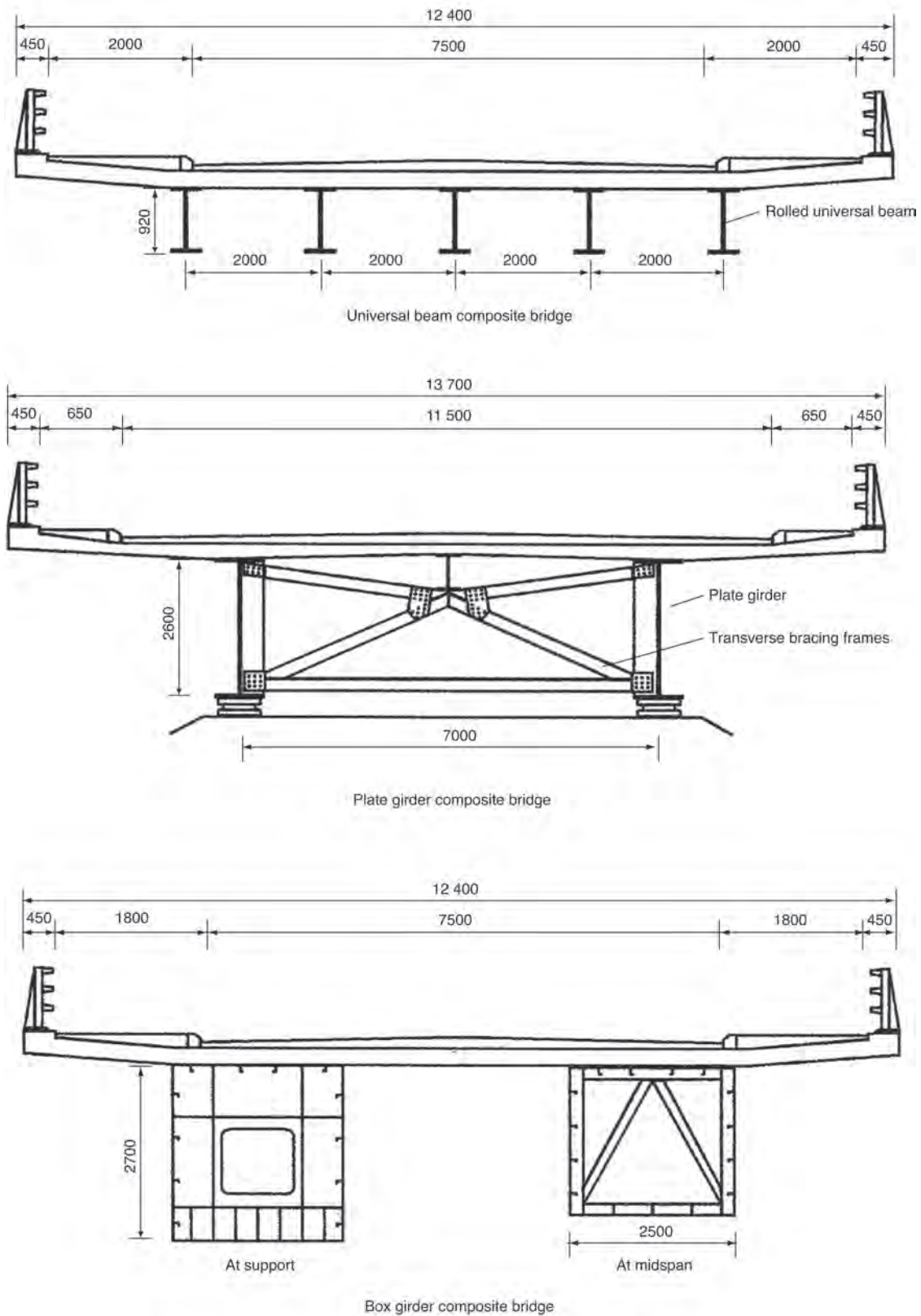


Figure 10.1 Cross-sections through typical composite bridges

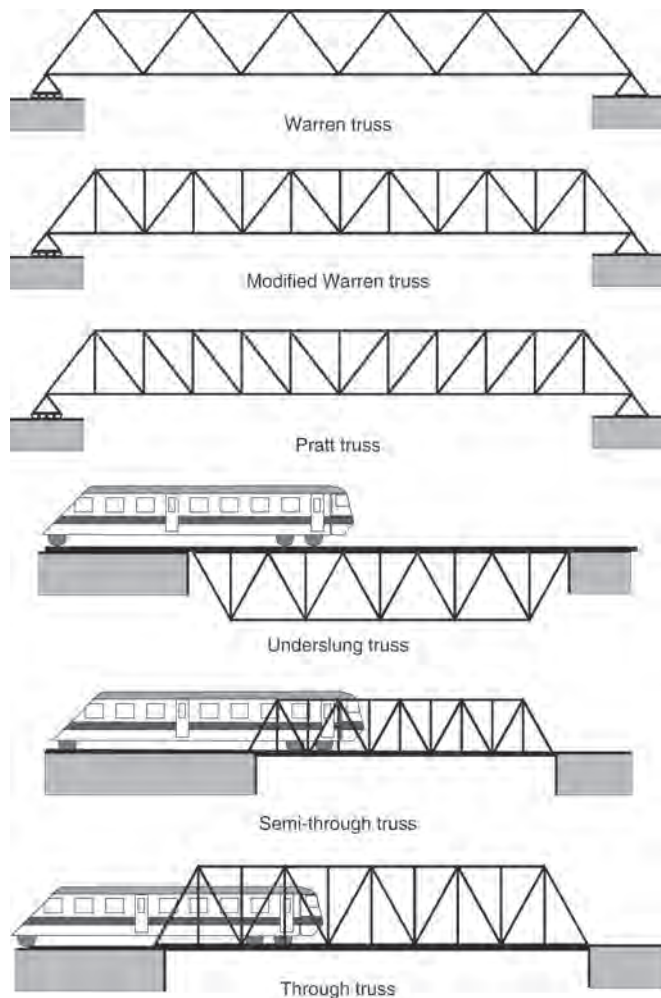


Figure 10.2 Typical truss bridges

either in axial tension or compression, depending on the truss configuration. In a Warren truss, the diagonal web members carry compression and tension alternately along the bridge. However, in a Pratt truss, all of the internal diagonal web members carry tension only, while the shorter vertical web members are loaded in compression.

Figure 10.3 shows a section through a typical through truss highway bridge and gives details of the terminology used in truss bridge design.

Analysis

Owing to the way in which truss bridges transmit the imposed loads to the foundations, through axial tension and axial compression forces in the members, it is acceptable to analyse these structures as pin-jointed assemblies, either as a two-dimensional plane truss or, preferably, as a three-dimensional space truss. This type of analysis assumes that member connections are pinned and consequently it is not possible for any of the

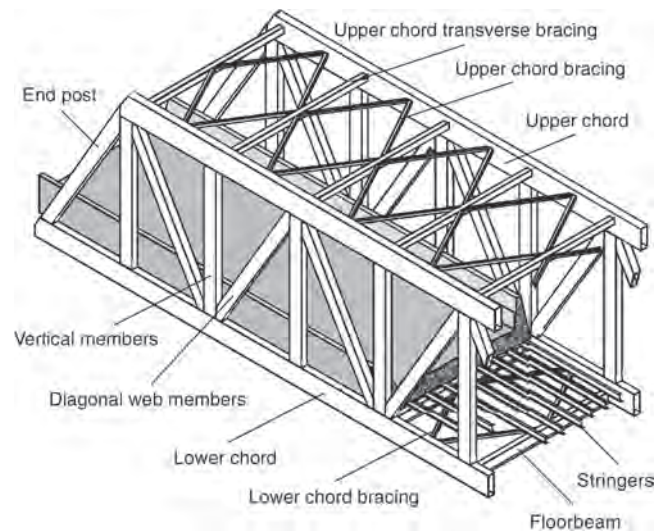


Figure 10.3 Truss bridge terminology (Chen and Duan, 1999)

truss members to attract moment or torsion forces. A two-dimensional plane truss analysis can be undertaken by hand, either by using equilibrium equations to resolve the forces at each joint in turn, or by using the 'method of sections' to free-body parts of the bridge truss, again using equilibrium equations to determine member forces.

The stiffness method can also be used to determine, first, node displacements, followed by member forces. Nowadays, truss bridges do not have pinned joints, the connections being either welded or bolted; however, analysing the structure as a two- or three-dimensional pin-jointed assembly will permit an accurate assessment of member axial loads but will over-predict the truss node displacements. To obtain a more realistic prediction of node displacements, together with an assessment of the secondary bending and torsion moments, which will be small but nevertheless present, because the joints are not pinned in reality, it is necessary to analyse the truss as a three-dimensional space frame with six degrees of freedom at each node. The secondary moments and torsions acting on the structure can influence the fatigue life of the bridge, especially if the truss is continuous, spanning several supports. Secondary forces, and hence stresses, can be minimised by ensuring that the neutral axis of all members meeting at a node intersect at a single point in three-dimensional space.

Members

Several different member types are suitable to use for the chords and web members of truss bridges. Rolled H-sections and rolled square hollow sections are suitable for the tension and compression chords and also for the web members of short-span highway trusses (30–50 m), whereas for longer highway truss bridges or trusses supporting railway loading, larger fabricated sections, such as a 'top hat' section or box section, will be required for the chords, as shown in Figure 10.4.

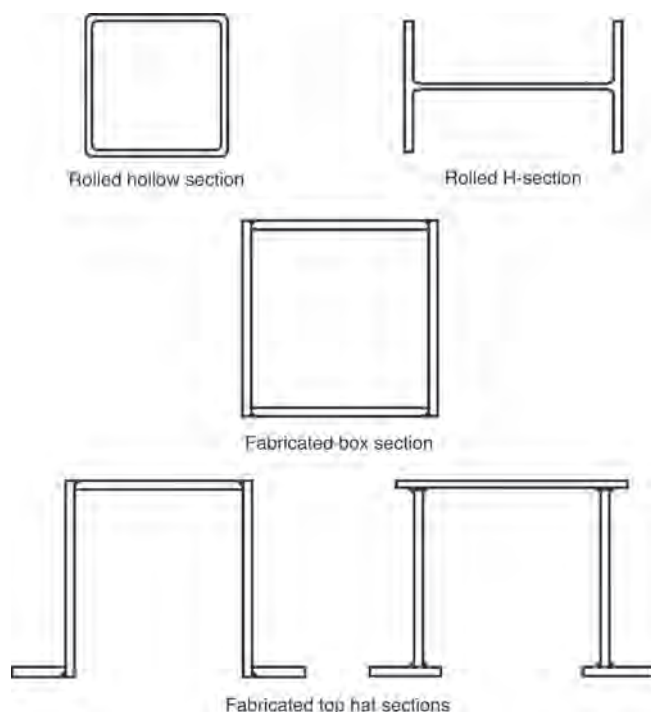


Figure 10.4 Typical chord members (Steel Construction Institute, 1985)

Open top hat sections will permit easy connection to H-section web members if the two section types being connected together are carefully sized so that the flanges of the H-section web members pass inside the top hat chord members. Top hat sections will require lacing and battening across the open side to ensure that the section is fully effective. Box sections form very efficient compression chord members, having improved lateral stiffness and aerodynamic stability compared with H- and top hat sections; however, connecting web members to box sections chords with very limited internal access may prove both awkward and expensive.

Compression members

Compression chord members, irrespective of the cross-section type, should be kept as short as practicable to maximise axial load capacity. Careful attention must be given to determine the appropriate effective lengths for buckling, both in the plane of the truss and normal to the plane of the truss. It is most likely that, owing to the lateral bracing running between the top chords and the differences in the radius of gyration about the two major axes of the member, the flexural buckling capacity for a chord will be different for in-plane and out-of-plane buckling. Effective lengths for in-plane and out-of-plane flexural buckling can be determined by undertaking an elastic critical buckling analysis of the whole truss; however, guidance on choosing the appropriate effective lengths can be obtained from Table 11, BS 5400 Part 3 (BSI, 2000) given in Table 10.1.

For simply supported underslung truss bridges, where the top compression chord is continuously supported by a steel or reinforced concrete deck, the chord members can be considered to be effectively restrained laterally throughout their length if the connection between the deck and the chord member under consideration is capable of resisting a lateral force, distributed uniformly along the member length, of 2.5% of the maximum force in the chord. Where the deck provides adequate lateral stability to the chord members, their effective length for buckling in the plane of the deck may be taken as zero. Where lateral restraint is provided by discrete cross members, provided the cross members have sufficient stiffness and are also capable of carrying at least 2.5% of the maximum force in the chord they are restraining, the effective flexural buckling length of the chord member in the lateral plane is equal to the spacing of the cross members.

For chord or web compression members whose cross-section is not doubly symmetric, such as top hat sections, which have only one plane of symmetry, failure can occur by flexural buckling about one of the principal axes, or by torsional buckling involving twisting about the shear centre, or by flexural–torsional buckling, which is a combination of flexural and torsional buckling, as shown in Figure 10.5.

Member	Effective length l_e		
	Buckling in plane of truss	Buckling normal to plane of truss when:	
		compression chord is effectively braced by lateral system	compression chord is unbraced
Chord	$0.85 \times$ Distance between intersections with web members	$0.85 \times$ Distance between intersections with lateral bracing members or rigidly connected cross-beams	See clause 12.5.1 of BS 5400 Part 3 (2000)
Web member			
Single triangulated system	$0.70 \times$ Distance between intersections with chords	$0.85 \times$ Distance between intersections with chords	Distance between intersections with chords
Multiple intersection system with adequate connections at all points of intersection	$0.85 \times$ Greatest distance between any two successive intersections	$0.70 \times$ Distance between intersections with chords	$0.85 \times$ Distance between intersections with chords

Table 10.1 Effective length l_e for compression members in trusses (BS 5400 Part 3 2000).

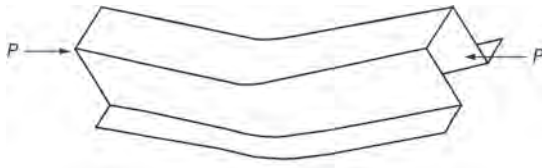


Figure 10.5 Flexural-torsional buckling of top hat compression members (Galambos, 1998)

Open sections that have their shear centre and centroid coincident, together with closed sections, are not subject to either torsional or flexural torsional buckling; however, they are still subject to flexural buckling.

Unbraced compression chords

In semi-through truss bridges, where vertical clearance requirements for the live load prevent the incorporation of a lateral bracing system for the top chords, the lateral restraint to the top chord is provided by U-frames, formed from transverse members in the bridge deck and vertical web members, as shown in Figure 10.6.

In these circumstances, the top chord compression members are considered to behave like a column braced at intervals by elastic springs whose stiffness is equivalent to the stiffness of the transverse U-frames. The buckling behaviour of a pin-ended column, restrained laterally at intervals by a series of springs, depends on the spring stiffness. If the spring stiffness is low, column buckling will take the shape of a single half-wave over the full length of the member, as shown in Figure 10.7.

Alternatively, if the spring stiffness is significant, nodes will form at the intermediate restraint points, and the buckled shape will exhibit a shorter wavelength, indicating a substantial increase in buckling capacity.

Early investigations into the behaviour of a column transversely braced at intervals by elastic springs were reported by Engesser (1885), who assumed that the elastic supports were equally spaced, each restraint had the same stiffness and the compression chord was straight, of uniform cross-section, equally stressed throughout its length and pinned, but rigidly held in position, at both ends. Engesser indicated that the stiffness required from a transverse U-frame in order to ensure that

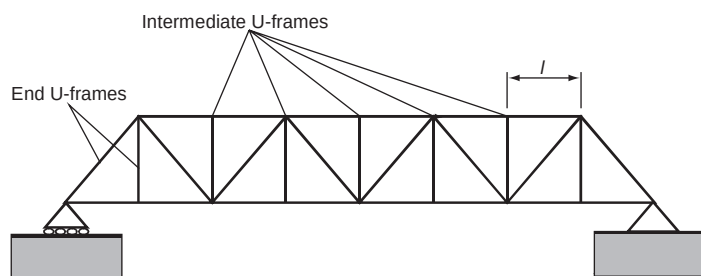


Figure 10.6 U-frames in a typical truss bridge (BS 5400 Part 3 2000)

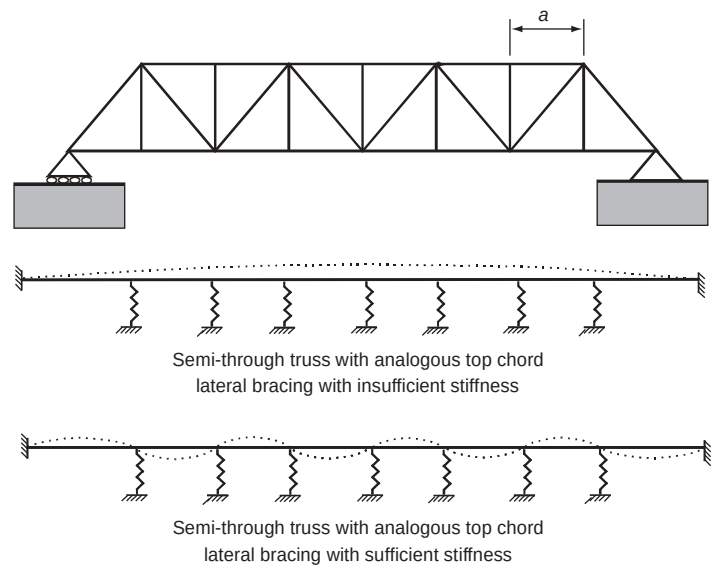


Figure 10.7 Top chord restraints for semi-through truss bridges

the complete truss compression chord supports an axial force of P_c is given by

$$C_{\text{req}} = \frac{P_c^2 l}{4EI} \quad (10.1)$$

where l is the panel length (Figure 10.6) and EI the flexural rigidity of the chord. Owing to the simplifying assumptions made by Engesser, the expression given in Equation 10.1 is not suitable for short-span semi-through truss bridges, which have only a small number of panels.

Engesser's investigation was reappraised by Hu (1952), who also studied the behaviour of elastically restrained compression chords in semi-through truss bridges. Hu stated that a suitable value for the transverse spring stiffness is given by

$$C = \frac{E}{h^2 \left[\left(\frac{h}{3I_c} \right) + \left(\frac{b}{2I_b} \right) \right]} \quad (10.2)$$

where the symbols correspond to the U-framework shown in Figure 10.8. It is important to appreciate that this equation neglects the contributions from the torsional stiffness of the compression chords and, in addition, does not include the transverse bending stiffness of the diagonal web members.

A similar expression for the transverse spring stiffness, the U-frame stiffness, C_d , is given in Table D.3 in EN 1993-2:2006 (BSI, 2006).

For design purposes, the effective length l_e for a compression chord restrained laterally by U-frames is given in BS 5400 Part 3 (2000) Clause 12.5.1 as

$$l_e = 2.5 k_3 (EI_c a \delta)^{2.5} \quad \text{but } l_e \text{ must not be less than } a \quad (10.3)$$

where a is the distance between the U-frames, as shown in Figure 10.7. The term I_c represents the maximum second moment of area of the chord about the weak Y-Y axis and the

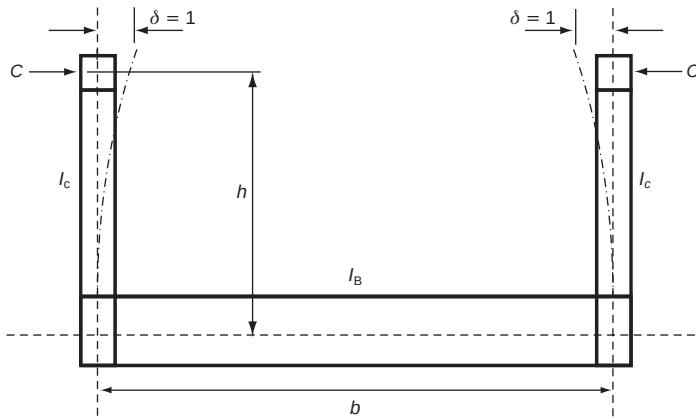


Figure 10.8 U-framework for semi-through truss (Galambos, 1998; Hu, 1952)

parameter k_3 depends on the degree of rotational restraint on plan, given to the ends of the chord member. If the compression chord is free to rotate in plan at both ends, k_3 must be taken as 1.0; however, if the compression chord is partially restrained against rotation in plan, at the points of support, the value of k_3 may be reduced to 0.85. The term δ (used in Equation 10.3) is given by

$$\delta = \frac{d_1^3}{3EI_1} + \frac{usd_2^2}{EI_2} + fd_2^2 \quad (10.4)$$

where the parameters d_1 , d_2 , I_1 , I_2 and s are shown in Figure 10.9.

For a semi-through truss bridge, the value of u can be taken as 0.5. The parameter f represents the flexibility of the connection between the cross-member and the verticals of the U-frame, expressed in radians per unit moment, and may be taken as 0.5×10^{-10} rad/N mm when the cross-member is bolted through unstiffened end plates or cleats, 0.2×10^{-10} rad/N mm when the cross-member is bolted through stiffened end plates or 0.1×10^{-10} rad/N mm when the cross-member is fully welded or bolted through stiffened end plates to a stiffened part of the

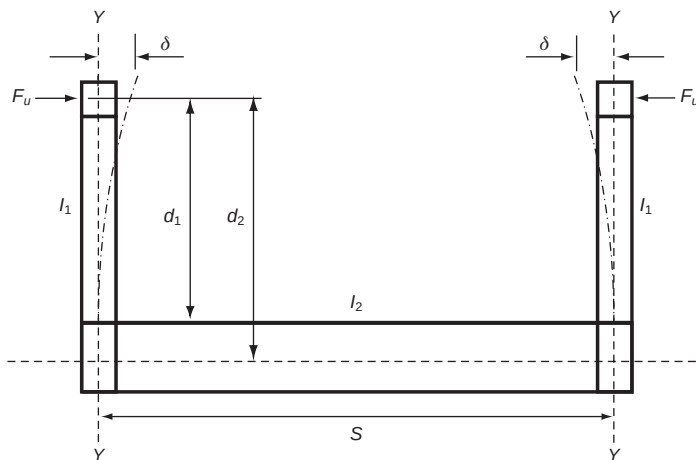
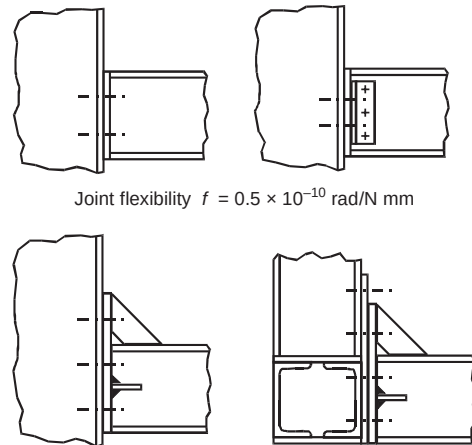
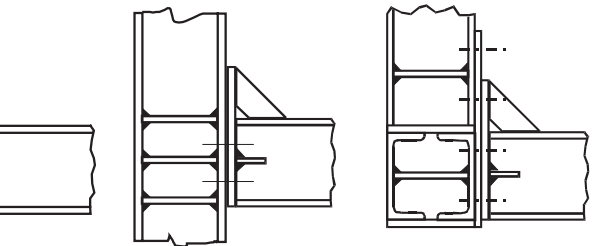


Figure 10.9 U-framework for semi-through trusses (BS 5400 Part 3 2000)



Joint flexibility $f = 0.5 \times 10^{-10}$ rad/N mm



Joint flexibility $f = 0.1 \times 10^{-10}$ rad/N mm

Figure 10.10 Joint flexibilities (BS 5400 Part 3 2000)

vertical member or stiffened section of the chord. Examples of typical cross-member to vertical connections and their respective flexibilities are given in BS 5400 Part 3 (2000) and shown in Figure 10.10.

Plate and box girder bridges

Introduction

As discussed previously in this chapter, one of the most common forms of steel (or composite) bridge, the plate girder, comprises steel plate elements welded together, often of relatively slender construction. These elements are found in the webs and flanges of plate girders and also in the stiffeners, although the latter are normally made from hot rolled sections. The box girder, somewhat less common, is found in longer-span bridges, either as a composite construction or, for very long spans, as an all-steel structure with stiffened steel decks. Such long-span girders may have additional support provided by cables, such as found in cable-stayed or suspension bridges (see Chapter 15). Very long-span bridges can have extremely complex cross-sections of aerodynamic shape with complex stiffening arrangements. Because of the slender nature of the individual plate components, they are prone to local buckling. In addition, other buckling modes might occur, such as lateral torsional buckling of a plate girder between points of lateral flange restraint. This is discussed later in this chapter.

To design the plated elements of a plate or box girder, it is desirable to have a degree of understanding of the behaviour of plates and the functions and effects of stiffening under various types of loading. The next section will explain the buckling behaviour of plates, including the effects of initial imperfections and boundary conditions representative of those found in bridge structures.

The behaviour of plates

Plates in compression

The reference case for an initial understanding of plate behaviour is a square plate, simply supported on all four sides and loaded by uniform in-plane compression along two opposite edges, as shown in Figure 10.11.

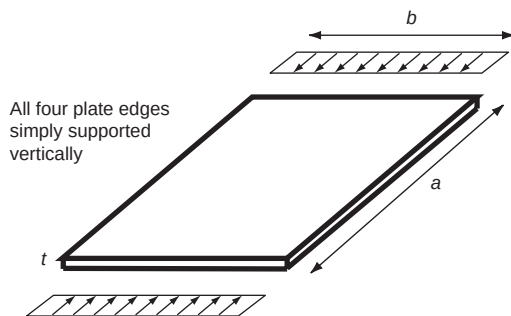


Figure 10.11 Simply supported square plate loaded by uniaxial compression

For a plate to be defined as slender, the in-plane dimensions of the plate, a and b , are significantly greater than the plate thickness t . The dimension b is normally the dimension transverse to the main component of in-plane loading; in the case of a laterally loaded plate, with loading applied transverse to the surface of the plate, it is normally taken as the smaller of the two dimensions. The dimensionless slenderness b/t is the most important slenderness in terms of the buckling capacity (and behaviour) of the plate and, while there are differences, it could be considered equivalent to the slenderness l/r for a column. The buckling behaviour of a plate is, in many ways, similar to a column, in that a perfect elastic plate loaded by increasing in-plane compression will carry all applied stresses as an in-plane elastic response with no out-of-plane deflection until the plate reaches the critical buckling stress (equivalent to the Euler stress). At this stress level, the plate will suddenly buckle out of plane but unlike the column, which is essentially in a state of neutral equilibrium after elastic buckling, the plate has a significant post-buckling reserve and the buckling resistance rises as the lateral deflections increase. This is shown in the plot of stress against lateral deflection shown in Figure 10.12.

Prior to buckling, the in-plane stresses in the plate will depend on the in-plane boundary conditions along the edges. If the longitudinal edges are free to move, they will move outwards because of Poisson's expansion. If prevented from

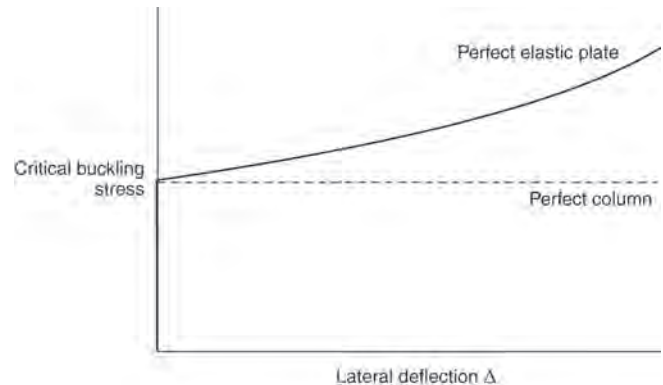


Figure 10.12 Post-critical reserve of a perfect elastic plate

moving, the resistance to displacement will cause transverse compressive stresses to be set up at the boundary. After buckling, the stress states will, because of the out-of-plane action, depend on both the in-plane and out-of-plane boundary conditions at the edges.

The critical buckling stress of the plate is a function of the plate slenderness (b/t), the plate aspect ratio (a/b), the boundary conditions and the nature of the loading. The general expression for the critical buckling stress of a plate is

$$\sigma_{cr} = k \frac{\pi^2 E t^2}{12(1 - \nu^2) b^2} \quad (10.5)$$

where E and ν are the material Young modulus and Poisson's ratio, respectively, and k is the buckling coefficient, which has a numerical value that is a function of the loading and the boundary conditions. For the uniaxially compressed plate with simply supported boundaries, k is generally taken to be 4, although the actual value of k varies with the aspect ratio, as shown in Figure 10.13.

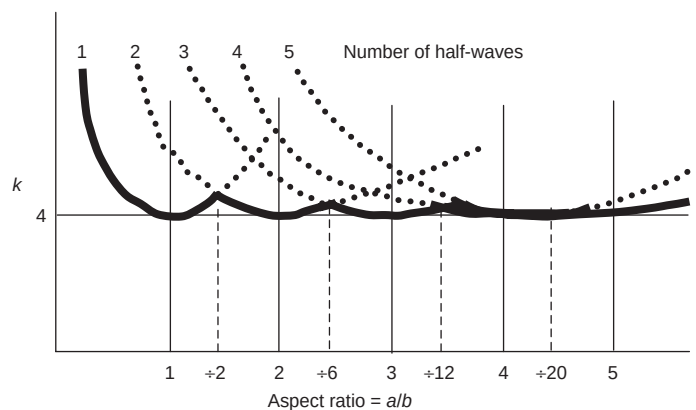


Figure 10.13 Variation of k with plate aspect ratio

This variation occurs because a perfect plate under in-plane compression will buckle into m square half-waves if the plate aspect ratio is an integer ($m = a/b$). This corresponds to the lowest energy mode; for other aspect ratios (e.g. 1.5) the plate will, in theory, have a larger elastic buckling stress. In practice,

this increase has no real significance because imperfections will interfere with this perfect behaviour. Buckling of a plate with an aspect ratio of 3:1 is shown in Figure 10.14.

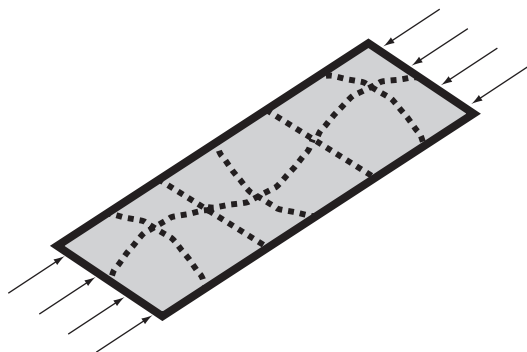


Figure 10.14 Buckling of a plate with a 3:1 aspect ratio

Longitudinal edge support conditions	Diagrammatic representation	k value
Both edges simply supported		4.00
One edge simply supported, the second fixed		5.42
Both edges fixed		6.97
One edge simply supported, the second free		0.425
One edge fixed, the second free		1.277

Table 10.2 Buckling coefficients for different boundary conditions

Table 10.2 shows values of k for boundary conditions relevant to bridge design.

The free edge cases with their much lower value of k (for example, 0.425 for one edge free and one edge simply supported) have a particular relevance for the behaviour of some stiffer cross-sections. This will be discussed later.

After critical buckling of the perfect plate, lateral deflections increase with the applied loading (in a square half-wave for the case being considered) and because the plate does not form a developable surface, membrane tensions are set up, which resist the growth of deflection and cause the post-buckling reserve mentioned before. Bending moments and twists are also set up in the plate, owing to out-of-plane action, and there is a coupling between in-plane and out-of-plane behaviour. The form and magnitude of the in-plane tensions depend on the transverse boundary condition along the unloaded edge. A Poisson's effect will cause compressive stresses to develop near the plate corners, while the deflection of the plate will

cause tension stresses to develop near the centre. If the boundary is completely unrestrained, however, the stresses cannot anchor at the boundary and any development of transverse stresses has to self-equilibrate within the plate; hence, the post-buckling reserve is modest. If the longitudinal plate edge is fully restrained against transverse movement, large edge stresses can develop and a significant net boundary tension can occur, with benefit to the post-buckling reserve. An intermediate condition exists, thought to approximate to the condition of an internal compression panel of a multistiffened flange of a box girder, where the edge of the panel is allowed to move but constrained to remain straight by adjacent panels. In this case, boundary stresses are possible but the central tensions have to self-equilibrate with the corner compressions because the boundary is unable to carry a net force. This boundary stress state is shown in Figure 10.15.

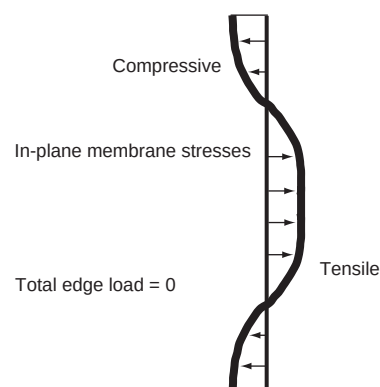


Figure 10.15 Transverse boundary membrane stresses for a constrained plate edge

The applied compression may also vary from the uniform condition shown in Figure 10.11 as the plate buckles, depending on the in-plane boundary conditions on the loaded edges. The uniform stress condition is not maintained, for example, if the loaded edge remains essentially straight, either because of the presence of an edge beam or, in practice, because of the presence of longitudinally adjacent panels. In this case, the buckling of the panel causes a decrease of the stress at the centre of the edge, as shown in Figure 10.16.

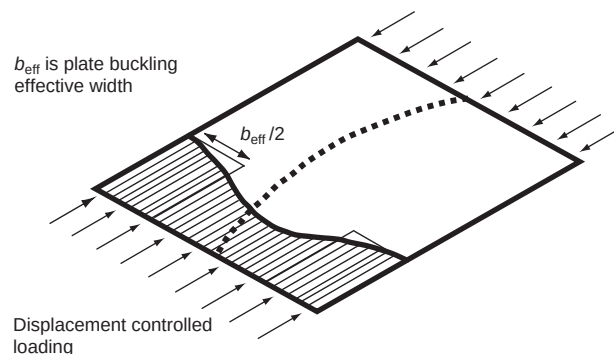


Figure 10.16 Distribution of stresses in a panel, owing to buckling

This stress reduction occurs because of a reduction in in-plane stiffness along the centre line of the plate, caused by the lateral deflection. It is an out-of-plane effect and should not be confused with the reduction in in-plane stresses at the centre of a flange in a box girder caused by the in-plane behaviour associated with shear lag. This will be discussed later in this chapter. The reduction in stress caused by buckling action gives rise to a concept often used in design, called the buckling effective width. In this simple representation, the actual stress distribution in the plate is replaced by an idealised distribution, which is of uniform magnitude (equal to the maximum of the actual distribution) but with a width, less than the actual width of the plate, known as the effective width. This width is evaluated so that the total force carried by the plate is the same as for the actual response. Figure 10.16 shows this idealisation. A commonly used design approach defines failure as the point at which the stress for the effective width reaches the yield stress.

Figure 10.17 shows the form of the effective width for a plate girder and simple box girder that might be used in a design approach.

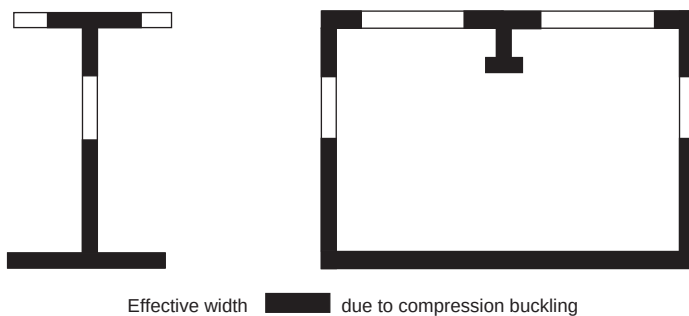


Figure 10.17 Effective elements in a plate girder and box girder due to compressive buckling (thicknesses exaggerated)

So far, the discussion has related to the behaviour of a perfect plate. In reality, no structural element or system is perfect. In particular, for the type of element being considered for bridge design, the most important imperfections can be considered to be of two forms: geometric imperfections and residual stresses. Geometric imperfections occur because of handling and fabrication processes. Because of the nature of plate buckling, the most important imperfections are out-of-plane deflections of the plate, which may or may not be sympathetic with the buckling mode. If they are sympathetic, they will tend to reduce the initial buckling resistance; if they are not sympathetic, they may even enhance the buckling load by resisting the development of buckling in the lowest critical mode. The second form of imperfection present in fabricated girders is a residual stress distribution, caused primarily by the welding process. This is not dissimilar to the residual stresses caused by hot rolling, for example in an H-section column member, but is somewhat different in form. As a plate edge is welded, for example, to an adjacent plate or to a stiffener, the metal has to cool from a molten state at the location of the weld. This differential cooling introduces in-plane stresses,

which are compressive near the centre of the plate and tensile around the weld location, which is the last region to cool. If the plate is slender, the compressive stress is non-linear, owing to out-of-plane deflection. The tensile stress will be at yield because of the weld material having been in a molten state. The state of residual stress is often idealised by a tensile block of width ηt at yield with a uniform compressive stress over the remainder of the plate width. This is not unlike the effective width idealisation. The tensile and compressive stress blocks must be self-equilibrating because no external forces are acting on the plate at this stage. Hence

$$(b - 2\eta t) \sigma_{RC} = 2\eta t \sigma_y \quad (10.6)$$

where σ_{RC} is the compressive residual stress.

The build-up of compressive residual stresses actually causes some additional out-of-plane imperfection in the plate panel, which tends to be in the critical buckling mode shape.

For practical fabrication, the geometric imperfection is around 1/200th of the plate width for a typical mild steel plate. Design codes often provide an equation that relates the fabrication tolerance to the plate slenderness and yield strength.

The compressive residual stress level σ_{RC} is typically around 10% of the yield stress for bridge fabrication, although it can be as high as 30% for heavy welding, such as sometimes used in heavy steelwork, for example, in ship construction. The effects of geometric imperfections and residual stresses are, however, not directly summative (with a significant imperfection present, a small residual stress level will only produce a modest additional reduction in strength); hence, many codes allow for residual stresses by providing a sensible conservative fabrication tolerance requirement, as residual stresses are not easily monitored.

The effect of geometric imperfections on the elastic response of plates, as for columns, is to initiate a gradual loss of plate stiffness from the onset of loading. This is shown in Figure 10.18, where the gradual increase in the out-of-plane deflection can be seen as the load is increased.

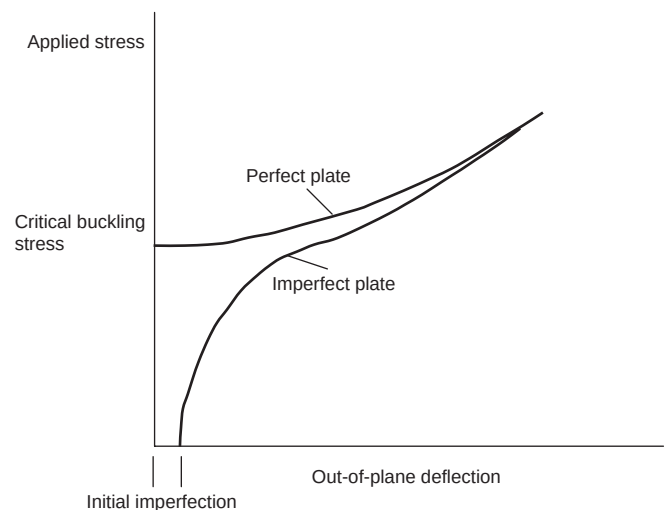


Figure 10.18 Effect of geometric imperfection on elastic buckling response

As with a column, because the post-critical response is stable, the elastic buckling curve eventually becomes asymptotic to the critical buckling curve as the deflections become large. The level at which this occurs depends on the magnitude of the initial imperfection.

The stress–strain or end shortening response of the plate is also affected by the level of the geometric imperfection. This is shown in Figure 10.19, where it can be seen that the bilinear response of the perfect plate changes to a gradual non-linear response.

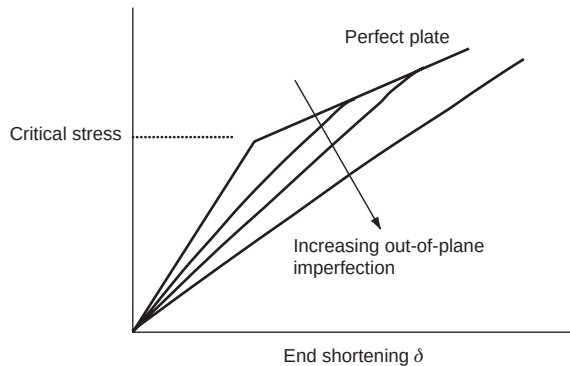


Figure 10.19 Load–end shortening response of an elastic plate

In this case, all curves start from zero strain or displacement because the initial imperfection is orthogonal to the end shortening. Residual stresses do not directly affect the elastic response because there is no limit to the stress level that can be attained.

While the elastic buckling response and, in particular, the elastic critical buckling value provide a reference for design (indeed codes have, in the past, applied imperfection and material factors to the critical buckling stress to obtain a design resistance value), with limit state design it is the ultimate collapse load of the plate panel that is important to the designer. This ultimate performance is affected by the non-linear response of the material, the degree of which varies, depending on the plate slenderness. Three ranges of slenderness can be defined, depending on the relationship between the yield stress and the critical buckling stress. Stocky plates can be defined as those where $\sigma_{cr} \geq \sigma_y$, slender plates as those where $\sigma_y \geq \sigma_{cr}$ and plates of intermediate slenderness as those where $\sigma_{cr} \approx \sigma_y$. The effects of geometric imperfections and residual stresses are different, depending on which range the slenderness falls.

For a stocky plate, failure occurs as an in-plane phenomenon with material yield controlling collapse. Out-of-plane deflection is negligible; hence, elastic critical buckling is irrelevant to the response. Geometric imperfections of normal magnitudes therefore have no influence on the response and the collapse stress–strain curve follows that of the material response, typically approximating a bilinear curve with a plateau, as shown in Figure 10.20.

Residual stresses, on the other hand, do influence the response because at a stress approximating to $\sigma_y - \sigma_{RC}$ the central area of the panel yields and the in-plane stiffness reduces.

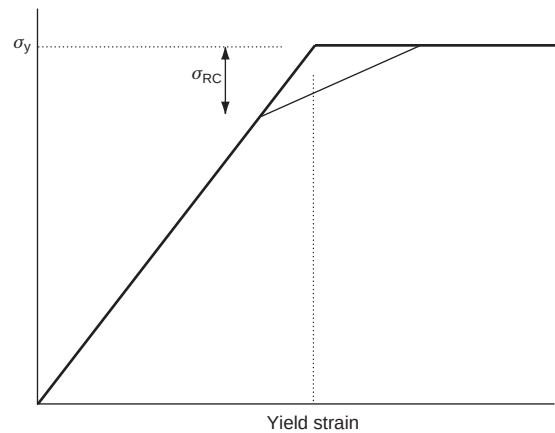


Figure 10.20 Stress–strain response of a stocky plate

However, because of the stable nature of the stocky plate, as the load increases, the stresses redistribute from the centre to the edge strips and the plate eventually reaches the same value of collapse load. This is also shown in Figure 10.20. Hence, the design value for such a plate is equal to the yield stress. Even when present, work hardening of the steel is ignored in the design because it generally occurs at high values of strain and hence affects the collapse behaviour at a late stage, particularly if buckling is taking place. In any event, the precise nature of the material stress–strain response would not generally be known at the design stage.

For a slender plate, the behaviour is dominated by the out-of-plane response, with buckling and hence elastic behaviour being dominant. As in-plane stress levels are relatively low and yield occurs as an out-of-plane extreme fibre phenomenon, particularly concentrated at the central zone of the plate, residual stresses are of little importance. While geometric imperfections will affect the behaviour and reduce the plate in-plane stiffness, because of the post-buckling stability, the sensitivity of the collapse behaviour to imperfection levels will be modest and the ultimate load will be dictated by the interaction between buckling deflections and out-of-plane yielding, gradually reducing the plate stiffness until it eventually reaches yield. Figure 10.21 shows the resulting stress–strain behaviour; it can be seen that the collapse stress is greater than the critical buckling stress because of the post-critical reserve.

A plate of intermediate slenderness, as for a column where the Euler stress and yield stress are approximately equal, is in the region of maximum imperfection sensitivity. This is because both buckling and yield interact in determining the collapse behaviour and ultimate load of the panel. However, plates are only moderately sensitive to imperfections because of their inherent stability. This sensitivity is a function of both the type of loading and the boundary conditions. For example, fully supported laterally loaded plates are insensitive, while plates with free edges loaded in in-plane compression are relatively sensitive. Plates in shear and in-plane bending, also relevant to the design of bridge girders, are also relatively insensitive, for reasons that will be described later.

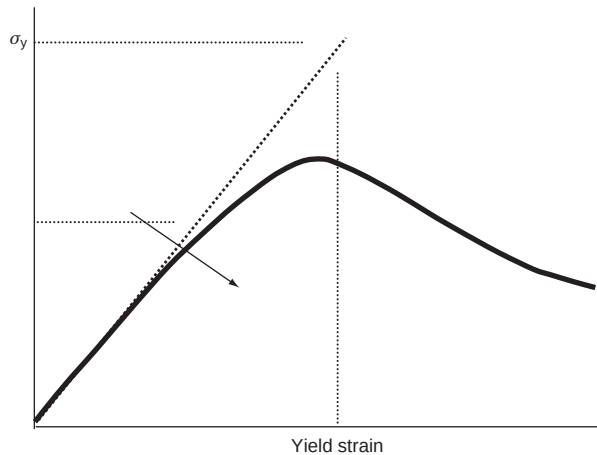


Figure 10.21 Stress-strain response of a slender plate

Considering the description of the slenderness zones, the strength slenderness relationship for a plate is shown in Figure 10.22.

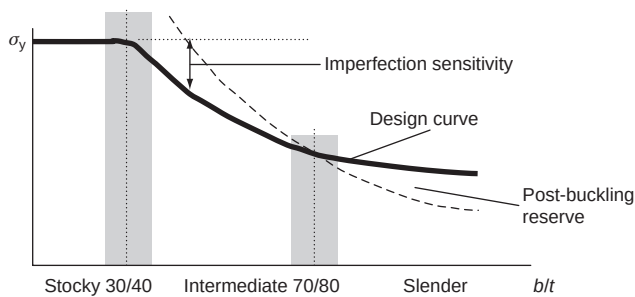


Figure 10.22 Strength slenderness curve for a plate panel in compression

The imperfection-sensitive area can be seen for intermediate slenderness and the post-buckling strength can be seen for high slenderness. For grade S275 steel, the zone limits occur around values of b/t of 30–40 and around 80.

The point of maximum imperfection sensitivity can be defined for a particular yield stress by equating the yield stress to the critical buckling stress. For a S275 steel yield stress of 275 N/mm² and $E = 205\,000$ N/mm², this gives

$$\sigma_y = 275 = \frac{k\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2 = 4 \frac{\pi^2 205000}{12(1-0.3^2)} \left(\frac{t}{b}\right)^2 \quad (10.7)$$

which gives $b/t = 52$.

In Eurocode 3 (BSI, 1992), the ultimate strength curve for in-plane compression is given in terms of a non-dimensionalised slenderness $\bar{\lambda}_p$, where

$$\bar{\lambda}_p = \sqrt{\frac{\sigma_y}{\sigma_{cr}}} \quad (10.8)$$

This curve is shown in Figure 10.23.

Approximate expressions, given in Part 1.5 of the Eurocode, define the buckling effective width factor ρ :

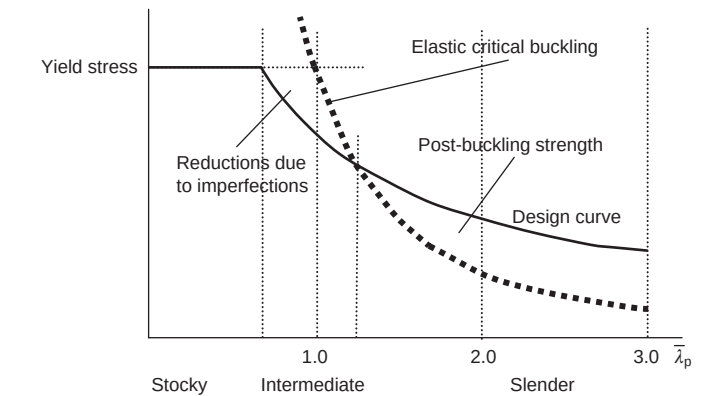


Figure 10.23 Compression panel ultimate strength curves in Eurocode 3 (BSI, 1992)

$$\rho = 1 \quad \text{when} \quad \bar{\lambda}_p \leq 0.673 \quad (10.9)$$

$$\rho = (\bar{\lambda}_p - 0.22) / \bar{\lambda}_p^2 \quad \text{when} \quad \bar{\lambda}_p > 0.673 \quad (10.10)$$

It was mentioned already that the critical buckling coefficient for a plate with a free edge with the other edges simply supported is very low ($k=0.425$). The buckling behaviour of such a plate is also significantly more unstable than for a plate supported on four sides, as it does not have the post-buckling reserve of the latter. During buckling, such a plate has a tendency to shed a high proportion of its direct stress and, if acting as a stiffener to a stiffened plate, the inertia of the section also reduces, as the stiffening element twists towards the attached plating. For this reason, strict limits are normally applied to the slenderness of outstand elements (flats, angles, tees and bulb flats) acting as stiffeners in plated structures. These limits are typically based on the elastic critical buckling behaviour of the elements, factored by an additional safety factor to allow for non-linear behaviour.

The buckling behaviour of such elements depends on their outstand slenderness and, in particular, on whether they are buckling elastically or plastically, but in either case the mode of local buckling is not unlike the lateral torsional buckling that can affect a beam member. If elastic, the critical mode is with the outstand in a half sine wave between supports, while if plastic buckling occurs, it is as a short wave S-shaped buckle, which can exhibit well-defined yield lines. These buckling modes are shown in Figure 10.24.

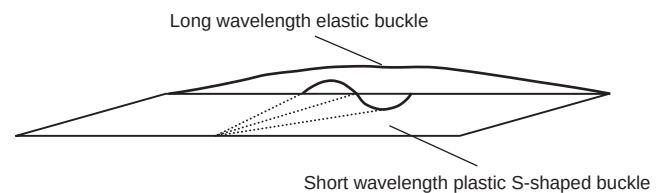


Figure 10.24 Buckling modes for a plate with one unsupported edge

Plates in shear

Figure 10.25 shows a plate panel loaded by in-plane shear along its edges.

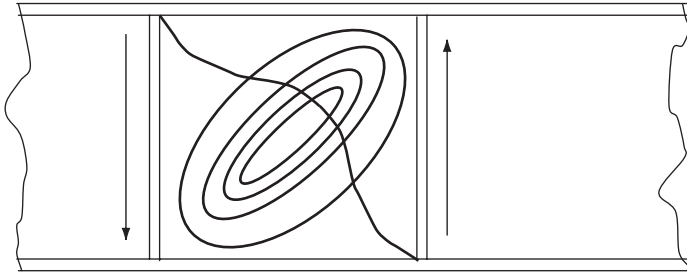


Figure 10.25 Plate in shear, showing the diagonal buckling mode

A perfect elastic plate will, as for compression, carry load by in-plane action alone prior to reaching its elastic critical buckling stress in shear. The plate will distort in-plane with stretching of one diagonal and shortening of the other, creating orthogonal tension and compression stresses. It is the compressive component of the stress state that causes buckling; hence, the buckling mode is of a diagonal form, as shown in the figure.

The single buckle shown occurs in a square plate but, like a plate in compression, the number of waves varies with the plate aspect ratio. Unlike the compression case, however, the critical buckling coefficient, and hence load, is significantly affected by the plate aspect ratio. The critical buckling expression (Equation 10.5) can be approximated as $k = 5.34 + 4(b/a)^2$ for $b/a \leq 1$ (Figure 10.26). This gives $k = 9.34$ for a square plate loaded in shear. The true expression is discontinuous with wave number.

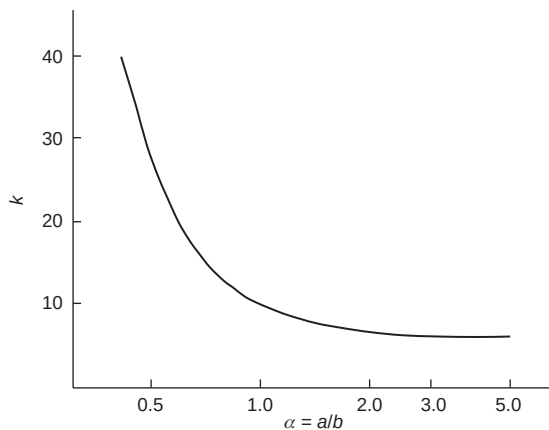


Figure 10.26 Buckling coefficient for a plate in shear

The higher value of 9.34, compared with the value of 4 for compression, reflects the presence of the tensile stresses inherent in the loading. The consequence of this is that plates can be designed with a higher slenderness in shear compared

with compression and still be economic. This can be seen by comparing the slenderness values at which the critical stress and yield stress are equal (the point of maximum imperfection sensitivity). The yield stress in shear is $\tau_y = \sigma_y / \sqrt{3}$. This can be appreciated from the governing von Mises yield equation:

$$\sigma_{VM}^2 = \sigma_{xx}^2 + \sigma_{yy}^2 - \sigma_{xx}\sigma_{yy} + 3\tau_{xy}^2 \quad (10.11)$$

Therefore, by equating the shear yield stress to the critical buckling stress with $k = 9.34$ for the same material values as used previously:

$$\frac{275}{\sqrt{3}} = 9.35 \frac{\pi^2 205000}{12 \times 0.91} \left(\frac{t}{b}\right)^2 \quad (10.12)$$

This gives $b/t = 95$, compared with the value of 52 for compression. A typical range of slenderness for the design of a panel in shear is $60 \leq b/t \leq 180$, with the stockier panels being used in the presence of combined compression and the more slender panels being used in the presence of tension.

A perfect elastic plate in shear undergoes two phases of behaviour. The first, prebuckling, corresponds to an in-plane response, where diagonal tensile and compressive contributions are equal. The second occurs after buckling, where the compressive stresses stabilise and begin to fall and the tensile stresses continue to increase, in principle up to the yield value. This post-buckling behaviour can be thought of as a band of diagonal tension, the vertically resolved component of which gives the panel additional shear capability. This behaviour leads to a simplified design process for webs called tension field design. The diagonal band is shown Figure 10.27.

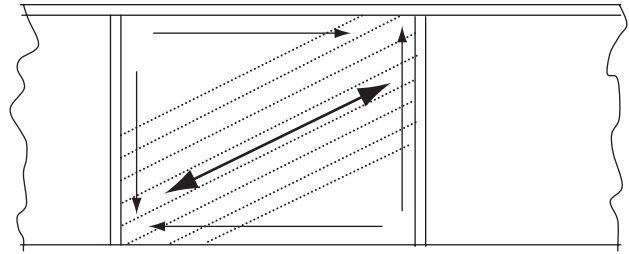


Figure 10.27 Tension field band in a post-buckled shear panel

Where the panel is adjacent to a flange or other panel, the tensile stresses can anchor off the boundary, increasing this contribution to the capacity. A restrained panel will have a strength close to the shear yield strength, even for very slender panels, and will effectively be imperfection insensitive. In fact, the tension field response means that all shear panels have a low imperfection sensitivity.

Figure 10.28 shows shear strength curves, illustrating the low sensitivity of strength to slenderness when the panel is restrained. Because the critical buckling coefficient is significantly affected by the aspect ratio, the latter also has an important effect on the collapse stress; therefore, different curves for different aspect ratios are presented.

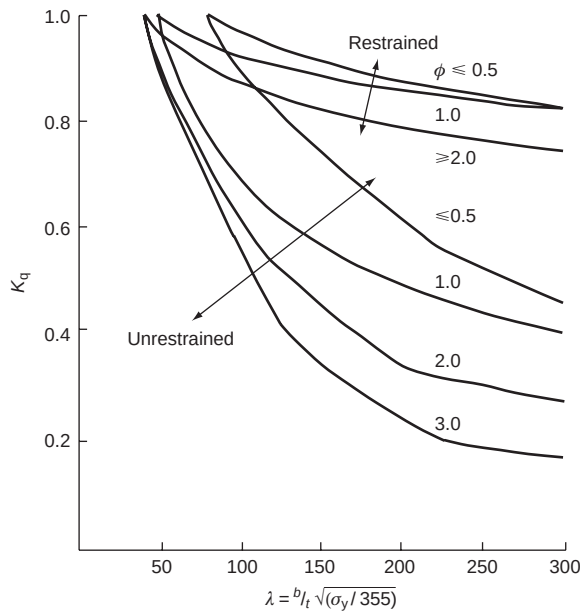


Figure 10.28 Shear panel ultimate strength curves (BSI, 2000)

This contrasts with the code curves for compression, which were independent of aspect ratio. The value of k_q (the value of k for shear) is multiplied by the shear yield stress to obtain the collapse shear stress, which is assumed to act uniformly down the panel. However, as will be seen later, an idealised tension field approach, rather than a panel strength approach, is used in some design codes, where the panel extends for the full depth of the web.

Plates under in-plane bending

The buckling coefficient for a plate in bending is very significantly influenced by the fact that half (in linear response) of the load is applied in tension. Figure 10.29 shows the resulting buckling mode for such a plate; it can be envisaged that the plate would buckle at a stress equivalent to the buckling stress of a stockier compression plate, that is, an effective width appreciably less than the breadth of the actual plate.

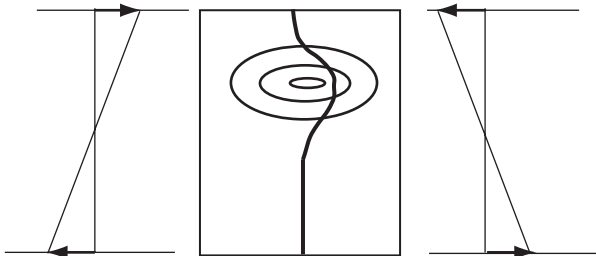


Figure 10.29 Buckling mode for a plate loaded by in-plane bending

In fact, the value of k for a plate in equal bending (k_b) is 23.9 and, like the compression plate, has curves with minima that are independent of the aspect ratio, although these minima

occur at aspect ratios somewhat less than in direct compression.

Again, because of the presence of the tension, the plate behaviour in bending is somewhat less sensitive to imperfections; consequently collapse loads, because of the variation of k_b with aspect ratio, are also insensitive to the plate aspect ratio. One further important phenomenon occurs in a plate whose edges are effectively held straight during the buckling response. As for shear, as the plate deflects, the compressive stresses tend to stabilise and decline, while the tensile stresses continue to increase. The neutral axis, therefore, shifts towards the compression side of the panel as the moment increases. While only stocky panels will reach the in-plane plastic moment capacity M_p , panels with intermediate slenderness, as large as $b/t=100$ and beyond, will exceed the in-plane yield moment value M_y . This redistribution of stress towards a plastic type distribution can be seen in Figure 10.30.

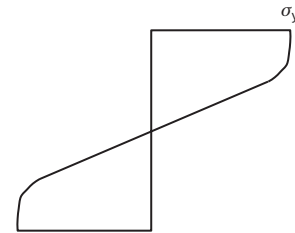


Figure 10.30 Redistribution of bending stresses in a stocky panel

The strength of a panel in bending is referenced to the yield stress value as for compression by the ultimate load coefficient k_b , as shown in Figure 10.31.

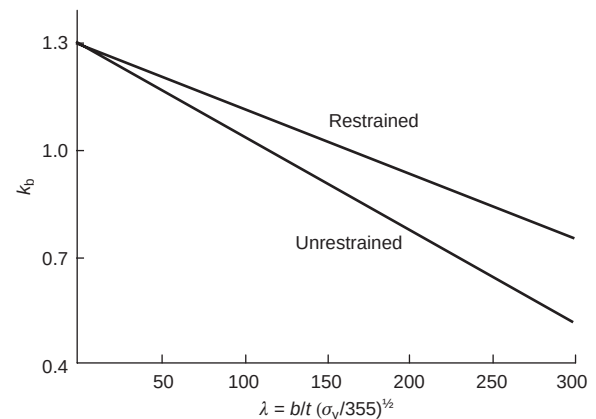


Figure 10.31 In-plane bending panel ultimate strength curves (BSI, 2000)

This is referenced to a linear stress distribution. To reflect this redistribution and provide strengths above M_y , the values of k_b increase above 1.0 to reflect the section shape factor.

In Eurocode 3 Part 1.5 (BSI, 2019), the same governing equations are used as for compression (Equations 6.9 and 6.10), with λ defined in terms of the bending critical stress and $k_b=23.9$. The value of ρ produced defines an effective width for the compression zone of the web, which is distributed in the

ratio 0.4:0.6 between the effective area of the flange near the web and the area next to the tensile zone when evaluating an effective section. The tensile zone is taken to be fully effective. The basis of the effective section approach within Eurocode 3 Part 1.5 is described later.

Panels under lateral loading

The concept of critical buckling has no relevance to panels loaded exclusively by lateral rather than in-plane loading. As the lateral load is applied, the panel will increasingly deflect out of plane, and out-of-plane moments will increase. As large-deflection behaviour occurs, membrane tensions will begin to resist increased deflection and may become very large, particularly if they can anchor off the boundaries. Behaviour will be limited by these membrane tensions reaching yield; hence, the panel collapse load is insensitive to imperfections. Of course, the significant deflections caused by the lateral loading could have a major effect on the stability of the plate under any coincident in-plane destabilising stress components, as they would essentially have the effect of large imperfections.

However, while relevant to, for example, the design of ship hulls, the only significant design case where lateral loading could be relevant to the element design in a bridge is for wheel loading on an orthotropic bridge deck. Even in this case, certain codes ignore the lateral load in the context of the design of the flange plate panels, only making an allowance for it in terms of an added in-plane stress contributing to the panel stress when considering yield in terms of serviceability.

Plates under combined in-plane stresses

The influence of plate slenderness and aspect ratio on the behaviour of a plate panel loaded in a combined stress state, for example, uniaxial compression and shear, reflects the influence they exert on the individual load cases. In addition, there is an interaction between the individual buckling modes that might enhance or detract from the collapse loads under the individual stress components, essentially dependent on whether the modes from the individual components are in sympathy. The elastic critical buckling interaction between biaxial compression, bending and shear is

$$\sqrt{\left(\frac{\sigma_{cx}}{\sigma_{cx,cr}}\right)^2 + \left(\frac{\sigma_{cy}}{\sigma_{cy,cr}}\right)^2} + \left(\frac{\sigma_b}{\sigma_{b,cr}}\right)^2 + \left(\frac{\tau}{\tau_{cr}}\right)^2 = 1 \quad (10.13)$$

where σ_{cx} , σ_{cy} , σ_b and τ are the applied compressive stresses in the x - and y -directions, the applied bending stress and the applied shear, respectively, and the 'cr' values are the corresponding critical stresses for the panel for each individual type of loading.

This shows that compression has the greatest influence on interactive buckling, while bending and shear have a smaller influence. It also shows the effect of biaxial compression in reducing the elastic critical buckling load.

No exact interaction equation exists for the ultimate collapse strength of a panel under a combined stress state. Figure 10.32 illustrates this behaviour by showing the collapse stress interaction diagram for combined compression or tension and shear.

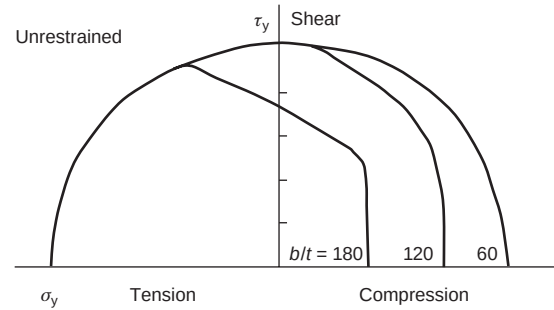


Figure 10.32 Collapse interaction diagram for applied tension or compression and shear

For a stocky panel that exhibits no buckling, the response follows the von Mises yield curve given by Equation 10.11 and is circular in form. It can be seen that the tensile stresses have a beneficial effect on shear buckling for large levels of shear and small levels of tension; for high levels of tension, buckling has no influence, even for modest levels of shear or slender panels, and the yield surface is again reached. For slender panels and large levels of compression, it can also be seen that shear has little influence until the levels of shear approach around 60% of the shear collapse values.

Figure 10.33 shows the corresponding diagram for bending moment and shear. There is little influence of shear on the buckling behaviour for shear values less than about 50% of shear yield and little influence of bending on buckling behaviour for bending values less than about 40% of M_u , where M_u is the collapse moment in the absence of shear.

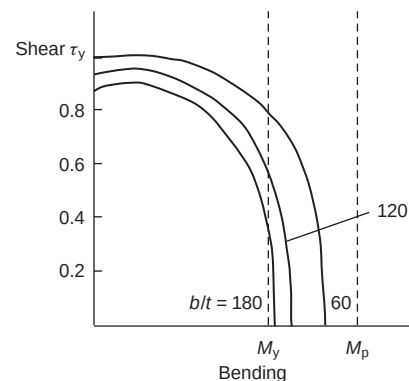


Figure 10.33 Collapse interaction diagram for applied bending and shear

An empirical curve fitting is used to represent these interaction diagrams, to relate the strength under individual stresses to the combined stress state. This interaction equation was based on that for critical buckling, defined as

$$\sqrt{\left(\frac{\sigma_{cx}}{k_c \sigma_y}\right)^2 + \left(\frac{\sigma_{cy}}{k_c \sigma_y}\right)^2} + \left(\frac{\sigma_b}{k_b \sigma_y}\right)^2 + \left(\frac{\tau}{k_q \tau_y}\right)^2 = 1 \quad (10.14)$$

where k_c are the appropriate values using the value of b transverse to the applied direct stress.

Design of plate girders

General design considerations

Plate girders are fabricated by welding flanges to a web plate, as shown in Figure 10.34.

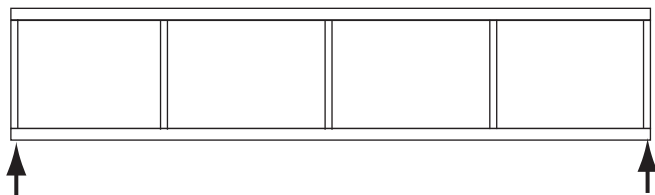


Figure 10.34 Typical form of a plate girder, showing possible stiffener arrangements for a transversely stiffened girder

The flanges are generally significantly thicker than the web because of the lower buckling capability of unsupported plates supported on one edge, as described previously. Occasionally, the unsupported edges of the flange are themselves stiffened by an outstand, but this is not often done in bridge design as it adds extra fabrication cost to the beam.

If the girder is to be used compositely with a concrete deck, the top flange will generally be narrower and will carry shear connectors for composite action with the concrete slab. In such a case, the width of the flange must be sufficient for the construction condition of carrying the wet concrete before composite action is achieved.

Cross-girders may be welded or bolted between adjacent girders, for example, for a rail bridge, where they support the deck that carries the track. In all cases, the main girders need to be appropriately designed to resist lateral torsional buckling. In the former case, lateral torsional buckling can only occur during construction as the concrete deck, once acting compositely, will provide full restraint to the steel top flange. In such a case, restraint is normally provided by cross bracing at appropriate intervals bolted to outstands welded to the girders. Where the girders are connected by cross-beams near the bottom of the main girders, U-frame action, whereby the cross-beams provide a moment restraint stiffness, can be used to prevent lateral torsional buckling of the girders, as described earlier. The design of lateral torsional buckling restraint using U-frame action is illustrated in Figure 10.35.



Figure 10.35 Lateral torsional buckling restraint provided by cross bracing and U-frame action

Plan bracing will also be needed between adjacent girders. If adjacent spans are continuous rather than simply supported, cross bracing may be needed near the supports to prevent lateral torsional buckling of the bottom flange, which is in compression in this region.

Lateral torsional buckling occurs because the low torsional and transverse stiffnesses of the girder, compared with the main vertical stiffness, allow failure by sideways and twisting deformations of the girder, even when loaded vertically. The section will be particularly prone to this form of failure if the vertical loading is applied to the compression flange, providing an increasing eccentricity of load as the girder deflects sideways. It is prevented through the measures discussed previously, by restricting sideways movement of the compression flange either directly through bracing or indirectly through the U-frame action.

Because they can be fabricated from plates of any width and thickness, plate girders can be used for much longer spans than can be achieved using beams from hot rolled sections, with their restricted sizes. Indeed, with plate girders, spans of over 200m are possible, often with haunches provided near continuous supports to increase moment capacity. Fabrication can be particularly effective where stiffening is kept to a minimum and welding is carried out automatically.

Plate girders are generally relatively deep to provide the moment resistance; webs therefore generally need only to be thin to resist the applied shear. This leads to relatively thin web design. In contrast, there is little benefit in having thin flanges, so these are designed to reduce buckling problems. This tends to lead to webs that require stiffening, at least over the supports, to prevent crippling caused by the high point loads and to allow the shear in the web to be transferred as compression down the bearing stiffeners into the support. Transverse stiffeners are also often provided for longer spans to enhance the buckling capacity of the web, principally by increasing its shear capability. In a minority of cases, longitudinal web stiffeners are also provided to increase the web buckling capacity by reducing the slenderness of the panels within the depth of the girder, as shown in Figure 10.36.

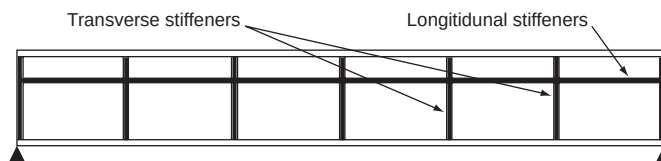


Figure 10.36 Plate girder stiffened by longitudinal and transverse stiffeners

However, this should be considered carefully, as it significantly increases fabrication complexity both by preventing the use of automatic welding and also by introducing a significant number of complex cutting and welding operations at the connection between transverse and longitudinal stiffeners. It is worth noting that intermediate transverse stiffeners are often curtailed short of the tension flange in order to provide better fatigue resistance. This is shown in Figure 10.37.

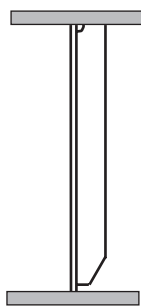


Figure 10.37 Curtailment of transverse web stiffeners to improve the fatigue response

Such curtailment does not affect the buckling enhancement of the stiffening, as the latter still provides out-of-plane bending support to the web plate.

In longer-span girders, it is possible to vary the cross-section in the longitudinal direction to match more closely the variation of moment or shear along the span. Flange thickness can be varied with full-strength butt welding, providing a smooth external flange surface. Flange width can also be varied. The possibility of a haunched girder has already been mentioned. Rather than vary the web thickness, it is more usual to vary transverse stiffener spacing or introduce a longitudinal stiffener over part of the span, for example, in the compression region over a continuous support. With modern fabrication, it is also possible to use different yield strength steels for different sections along the bridge to achieve a variation in capability. In all cases, the benefit of better matching the resistance of the structure to the applied loading needs to be weighed against any increased fabrication complexity, and hence cost, that might ensue.

Where holes are required in the girder web to accommodate services, it is necessary to provide some form of stiffening around the openings. A rule-of-thumb design approach is to replace the material removed from the depth of the web plate by the area of the stiffeners but the designer must ensure that the remaining web section is capable of carrying the applied shear. Figure 10.38 illustrates such openings.

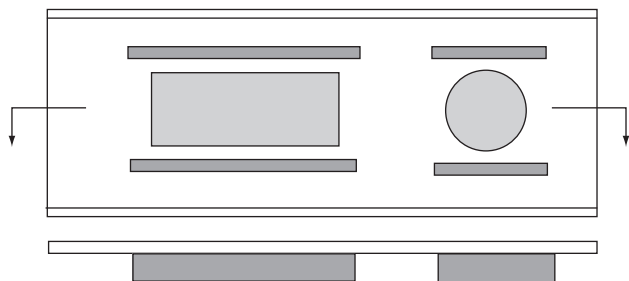


Figure 10.38 Plate girder with openings in the web

It is possible to define a range of practical dimensional proportions that are typical of bridge construction. Plate girders with spans up to around 1000 m have been used as suspension

structures but one notorious example, namely the Tacoma Narrows Bridge, with a span of 853 m, failed disastrously in 1940 because of its flexibility to wind excitation. Box girders are now generally accepted as more appropriate for longer spans because of their inherent torsional stiffness. More modest plate girder suspended spans of up to 400 m are not unusual. Cable-stayed composite plate girder bridges have been constructed with spans up to about 500 m. Composite plate girders without additional support are used for many bridge structures over a range of more modest spans, as a competitor to prestressed concrete structures.

Overall girder depths range between one-tenth and one twentieth of the span, with the larger depths used for longer spans. Flange widths will tend to be between one-third and one-fifth of the girder depth. As has been noted previously, flange plates are usually designed to be stocky, to preclude loss of strength by buckling. They would normally be designed as at least semi-compact, a definition for slenderness used in Eurocode 3 (BSI, 1992). In terms of the Eurocode, this means applying a limit of $14t_f\varepsilon$ to the flange outstand width where t_f is the flange thickness and ε is $(235/\sigma_y)^{0.5}$.

The terms *compact*, *semi-compact* and *non-compact* define the moment that a girder can carry prior to buckling and also its ability to redistribute moment along the span prior to ultimate collapse. In modern codes, there are four classes of section. Class 1 sections, generally called plastic, are able to reach their plastic moment, M_p , and to attain sufficient rotation prior to buckling to allow redistribution of moments and hence the use of plastic collapse analysis. Class 2 sections, compact, are also able to reach M_p but have limited rotational capacity. Class 3 sections, semi-compact, are able to reach their first yield moment M_y (a moment that at least achieves σ_y at the extreme fibre of the girder) prior to buckling. Class 4 sections, slender girders (the equivalent of non-compact in BS 5400) will not reach M_y and require design rules that either define a limit stress smaller than the yield stress or that define a reduced effective section, either through an effective width concept or by using an artificially reduced thickness. Figure 10.39 illustrates the response of girders with the four classes of cross-section.

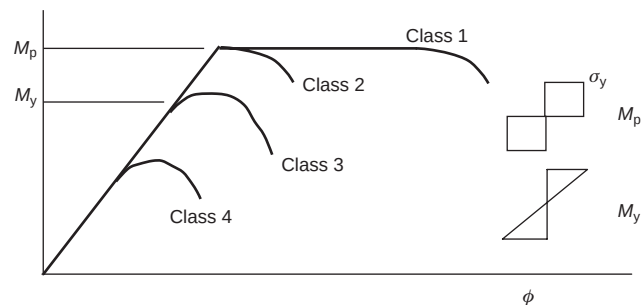


Figure 10.39 Moment–rotation behaviour of girders with different classes of cross-section

In all cases, both the webs and flanges of a girder must adhere to the individual classification limits for a girder to fall within a particular class.

The web thickness cannot be simply defined in terms of a standard range. Because of the range of stiffening options, a web might have a depth-to-thickness ratio ranging between 80 and 500. Longitudinal stiffening can be considered for webs with slenderness ratios larger than about 200.

Plate girder design rules

This section describes the principles behind the rules of Eurocode 3 Part 1.1 (BSI, 1992) for the design of plate girders presenting the main elements of the design process. Alternative rules for girder design are presented in Eurocode 3 Part 1.5, and are specifically aimed at more complex stiffened cross-sections but are applicable to both plate and box girder design. These are presented in the section on box girder design rules.

In all the following sections, where strength is defined, the design must incorporate appropriate safety factors. With the partial safety factor format used in modern limit state design codes, these are applied to both the load and resistance sides of the equation. Reference should be made to the appropriate code to establish appropriate values and use. The equations given here are also only a guide and should not be used out of the context of the code. The descriptions given are intended to outline the key factors in the design process but should not be used in isolation from the full code requirements.

A simple and often effective design basis for a girder is to design the flange to carry all the moment and the web to carry all the shear. Even where a more complex basis is used for the final design, this can be a very effective method for initial sizing. This approach recognises the inherent capabilities of the two elements, and is permitted by Eurocode 3 Part 1.1. The required area of the flange can be readily obtained for a given cross-section by dividing the moment by the distance between the flange centre lines (allowing, of course, for appropriate safety factors). As this involves knowing the flange thickness, a minor iteration might be required. The design of the web will depend on its slenderness, although, if only transversely stiffened, the shear stress can be assumed to be uniform down the depth.

In Eurocode 3 Part 1.1, web buckling is only considered where the web slenderness, d/t_w , exceeds 69ϵ . Prior to this value, the full yield stress in shear may be used. Beyond this value, Eurocode 3 Part 1.1 allows two design methods for plate girder design. These are the simple post-critical method and the tension field approach. The latter has already been mentioned in this chapter, in the context of panels in shear. For the latter design method to be used within Eurocode 3 Part 1.1, transverse stiffeners must be present with spacing between the web depth and three times the web depth. Because the tension field approach allows for the full post-critical behaviour of the girder, higher collapse loads are provided than by the simple post-critical method.

Simple post-critical method within Eurocode 3 Part 1.1.

In this approach, the shear buckling resistance ($V_{b,Rd}$ in Eurocode 3) can be determined simply from three equations, which are applicable depending on the slenderness of the web. These equations provide a design shear stress (τ_{ba}) that can be multiplied by the web area to provide the shear strength. Figure 10.40 shows the design curve for τ_{ba} as a function of $\bar{\lambda}_w = \sqrt{\tau_y/\tau_{cr}}$.

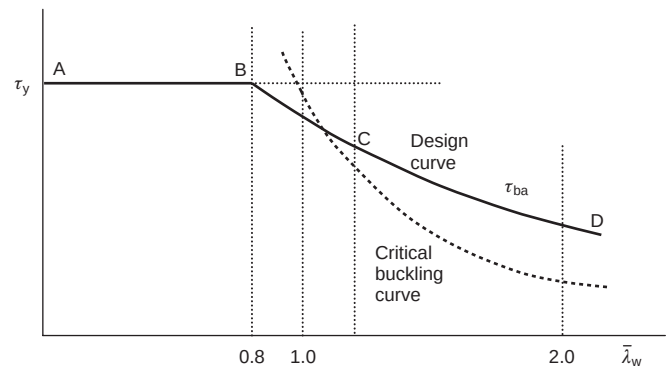


Figure 10.40 Simple post-critical design curve for shear buckling according to Eurocode 3

Equations are provided in the code that, for stiffener spacings larger than the web depth, correspond to that given previously for critical shear buckling.

Equations for zones AB, BC and CD in Figure 10.40 are given in the code, and are termed stocky or thick, intermediate and slender or thin, respectively.

Tension field method within Eurocode 3 Part 1.1

While tension field behaviour has been described in the context of an individual shear panel, the behaviour of a girder web exhibits additional features. The diagonal tension field band, which occurs after the critical buckling stress is reached, anchors off both the top and bottom flanges, and also off the transverse stiffeners on either side of the web panel being considered. The degree of anchorage depends on the transverse stiffness of the flanges, as well as the adequacy of the stiffener design. A third additional component of resistance comes from the bending deformation of the flange (out of plane), which must occur for the girder to fail in shear. This third component is normally evaluated by assuming a plastic collapse mechanism to occur in the flanges for an effective section of the flange plus an effective depth of web.

The equation representing the capacity of the web in shear in the code is

$$V_{bb,Rd} = d t_w \tau_{bb} + 0.9 g t_w \sigma_{bb} \sin \phi \quad (10.15)$$

where τ_{bb} is the shear strength of the panel and the second term provides the tension field contribution.

If the web is stocky, $\lambda_w < 0.8$, it will reach shear yield. If it has a higher slenderness, the web will reach its elastic buckling stress according to the equations reproduced in Figure 10.41.

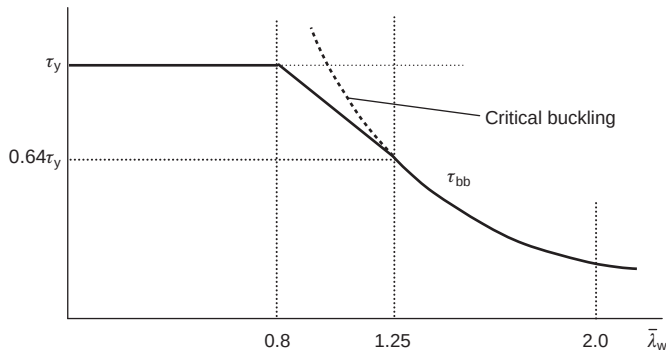


Figure 10.41 Strength corresponding to web buckling not allowing for tension field

Calculation of the tension field effect is relatively complex. The angle of the tension field band ϕ is unknown and, in principle, could be established by an iterative procedure that maximises the resistance. However, studies have shown that if a value of $\phi = \theta/1.5$ is used, where θ is the angle of the web panel diagonal ($\tan^{-1}(d/a)$, where d is the web panel depth and a is the transverse stiffener spacing), a reasonable and conservative value is obtained for the tension field strength contribution. The tension field stress in the tension band, σ_{bb} , is the material yield stress reduced to allow for the shear stress present τ_{bb} . Using the von Mises yield criterion produces

$$\sigma_{bb} = 0.5(f_y^2 - 3\tau_{bb}^2 + (1.5\tau_{bb}\sin 2\phi)^2) - 1.5\tau_{bb}\sin 2\phi \quad (10.16)$$

where f_y is the yield strength of the web material and τ_{bb} is the elastic critical buckling stress described previously.

Finally, the width, g , of the tension field band is evaluated through consideration of the flange plastic mechanism. It is a function of the positioning of the flange hinges, which is itself dependent on the plastic moment of resistance of the flange, reduced to allow for coexistent axial force in each flange, owing to the bending moment in the girder. The full equation is to be found in the code.

Interaction between shear and bending in Eurocode 3 Part 1.1

The presence of bending stresses in the girder web will reduce its shear carrying capacity. These influence both the elastic buckling capacity and the extent of the tension field, as well as the flange plastic resistance, as mentioned above.

For the simple post-critical method, interaction between moment and shear can be considered as limited, as shown in Figure 10.42.

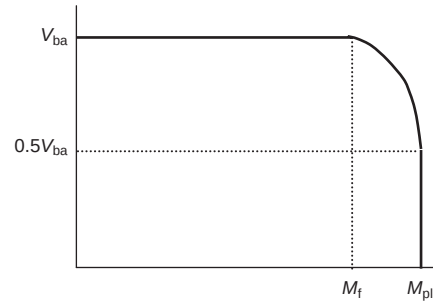


Figure 10.42 Interaction between bending moment and shear force for the simple post-critical method of Eurocode 3 Part 1.1

For moments less than the moment resistance of the flanges alone (the simple design basis mentioned at the start of this section), there is no need to allow for interaction and the full shear resistance calculated from the simple post-critical method can be used. If flanges are not semi-compact, it would be necessary to define an effective width to allow for buckling prior to the calculation of M_f , which is the moment provided by the two flanges alone.

Using the simplified approach of ignoring the web contribution to the moment resistance and designing the flange as semi-compact in combination with the simple post-critical method for shear, therefore, leads to an effective and simple, if slightly conservative, design approach. If the web is designed to carry moment then, when the applied shear is less than half the shear resistance produced by the simple post-critical method, there is no need to reduce the moment capacity of the girder including the web. However, for the region of the interaction curve where the shear is higher, the relationship

$$M_{Sd} \leq M_{f,Rd} + (M_{pl,Rd} - M_{f,Rd}) [1 - (2V_{Sd}/V_{ba,Rd} - 1)^2] \quad (10.17)$$

should be used to define the interaction. This gives the design moment capacity as a function of the design shear value, the simple post-critical shear resistance and the moment capacities of the girder including the web and the flanges.

If the web is more slender than given by $d/t_w \leq 124\epsilon$, the moment capacity needs to be reduced to allow for web buckling owing to the in-plane compression.

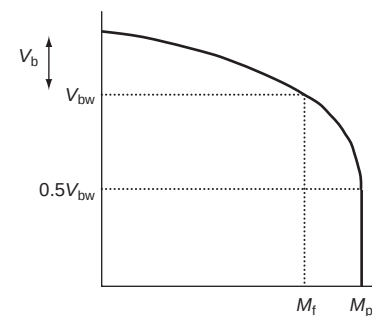


Figure 10.43 Interaction between bending moment and shear force for the tension field method of Eurocode 3 Part 1.1

For the tension field approach, there is a greater interaction between shear and bending in the girder. For moments less than the moment resistance of the flanges alone, a shear capacity including the web tension field, but excluding the plastic moment flange contribution, can be carried. This can be evaluated by giving the flanges a zero plastic moment of resistance in the shear capacity equation. It can be seen from Figure 10.43 that this is conservative.

At the other end of the interaction, shear may be ignored, as long as the shear force is less than half the tension field shear capacity of the girder, ignoring the flange out-of-plane bending contribution. The governing equation for intermediate interaction is

$$M_{sd} \leq M_{f,Rd} + (M_{pl,Rd} - M_{f,Rd}) [1 - (2V_{sd}/V_{bW,Rd} - 1)^2] \quad (10.18)$$

Web crippling

Occasionally, bridge girders are subjected to significant local concentrated loading that can induce high localised compressive stresses in the web. A good example of this occurs if the girder is launched over a rolling support. In this case, the web may be subjected to very severe local patch loading at sections that are not strengthened by transverse stiffeners. In such circumstances, it is necessary to check the web for local web crippling, the resistance to which is a function of the load length, the spacing of the stiffeners and the depth of the panel (in some circumstances). Different buckling behaviour occurs, depending on the relationship of these parameters. The Eurocodes have sections dealing with this form of buckling phenomenon.

Transverse web stiffeners

Intermediate transverse stiffeners regularly spaced along the web of a plate girder are used to increase the buckling capacity of the web. Stiffeners provided at support points require a different design approach and are considered in the next section. The design of intermediate transverse stiffeners in box girders adheres to similar requirements. In general, transverse stiffener cross-sections are made of hot rolled sections and are either flats or bulb flats, angles or T-sections. The latter are used where the depth of the stiffeners needs to be large enough to permit longitudinal stiffeners to pass through cut-outs in them. Longitudinal stiffeners would normally be either bulb flats or angles and would be welded to the transverse stiffeners. It is an important principle that the main longitudinal load-carrying elements, for example, the longitudinal stiffeners, are continuous, to avoid eccentricities in the longitudinal load path.

As mentioned already, the Eurocode requires stiffeners to be spaced at between one and three times the web depth. If the stiffener spacing were any closer, it would be likely that the increased fabrication cost would more than offset the savings in web material achieved through using a thinner web; if more widely spaced, the enhancement to web buckling capacity would be limited.

Transverse stiffeners have two main functions in terms of the web capacity. First, they will increase the out-of-plane buckling resistance of the web by acting as a nodal line, preventing out-of-plane deformation. Second, they provide anchorage for tension field forces thereby enhancing ultimate web shear strength. They can also carry limited direct loading from the deck into the web (not relevant for composite plate girders) and can also help to prevent the flange buckling into the web.

The Eurocode provides an inertia requirement (a stiffness) to prevent buckling of the web distorting the stiffener out of the plane of the web and a column (strut) requirement to ensure that the stiffener can carry the resolved tension field force.

Looking at the Eurocode 3 Part 1.1 design method (BSI, 1992), the equations for the inertia of the stiffener are given in terms of the web depth, web thickness and stiffener spacing and are also dependent on the panel aspect ratio. The strut approach uses the resolved tension field force N_s acting at the centre line of the web. It is, therefore, an eccentric load if the stiffener is only on one side of the web plate, as is the norm for intermediate stiffeners. The stiffener and an associated width of web plate ($15\epsilon t_w$ on either side of the stiffener section) are designed as a strut section using the column rules in the code. The load applied to this section is

$$N_s = V_{sd} - d t_w \tau_{bb} \quad (10.19)$$

where τ_{bb} is the shear buckling resistance without any allowance for tension field and V_{sd} is the design value of the shear force.

Again, in the approach outlined in the Eurocode, the force applied to the stiffener results from the increased web capacity above a certain critical buckling stress. The axial force representing the destabilising action, acting at the centroid of the effective strut, is given by

$$F_{wi} = \frac{l_s^2}{a_{\max}} t_w k_s \sigma_R \quad (10.20)$$

where $a_{\max}$ is the maximum transverse stiffener spacing to satisfy the web design (and can be taken as a), k_s is a numerical parameter that is a function of the strut slenderness, with a maximum value of 0.4 for very slender struts, and σ_R denotes the destabilising stresses present in the web (with an allowance for web longitudinal stiffeners if present):

$$\sigma_R = \tau_R + \left(1 + \frac{\Sigma A_s}{l_s t_w}\right) \left(\sigma_1 + \frac{\sigma_b}{6}\right) \quad (10.21)$$

where ΣA_s is the total area of longitudinal web stiffening, τ_R is the lower of τ or τ_0 , σ_1 is the average value of web longitudinal stress (positive for compression) and σ_b is the maximum stress owing to bending alone.

A simpler design requirement for the effective area of the stiffener is provided in the code if the only load component is the destabilising web action (i.e. the last component in this list).

End post or load-bearing support stiffener design

End posts have to transfer significant shear loads from the web to the support bearings. They also have to anchor any tension

field forces in the absence of an adjacent web panel. Because of the magnitude of the compressive load carried, they would normally be located on both sides of the web to avoid eccentricity. A grouping of four stiffeners, two on either side and close to the bearing on both sides of the web, may be used to provide a strong H-section column over the bearing. Possible arrangements of bearing stiffeners are shown in Figure 10.44.

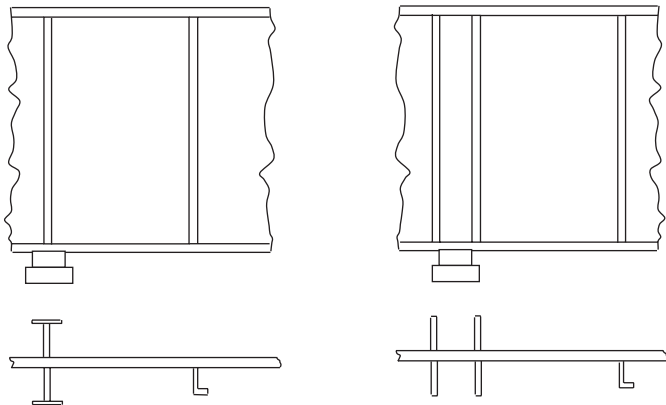


Figure 10.44 Possible arrangements for end posts

The Eurocode outlines a range of forces and moments that the end post section must resist. These include direct loads, the destabilising force from the web buckling and a moment produced by the eccentric tension field force. The code provides strength, stiffness and yield requirements. Again, the requirements are too complex to be presented here in detail.

In addition to providing a similar approach including definition of a tension field behaviour that allows the geometry of the mechanism to be defined, the Eurocode suggests the option of designing the final web panel using the simple post-critical procedure even if the remaining panels are designed using the tension field approach so that the tension field components are not present. In order that the end panel shear strength is the same as the remainder of the adjacent girder, so that it does not present a weak link in the load transfer path, the end panel is closed up with a closer stiffener spacing than elsewhere.

Design of box girders

General design considerations

Box girders, while more expensive to fabricate than plate girders because of their complexity, have a number of significant advantages, particularly for longer spans. First, because of the shape of the box, the top flange itself can act as the decking without the need for a concrete deck. Box girders can also be designed with an aerodynamic optimised shape, again making them ideal for long spans. Their high torsional rigidity again helps in the context of long spans, but also in providing flexibility for bearing arrangements and where a

structure curved in plan is required.

For intermediate spans, a box can be used compositely, with either a full width top flange or two narrow flanges. In all cases, shear connection must be present over the full flange width. Depending on the width of slab needed, the former type can have transverse cross-girders connected to the boxes. Cantilever cross-girders may also be used to increase the bridge width. The cross-girders provide additional support to the concrete flange, allowing it to span in two directions. An example of each type of structure is shown in Figure 10.45.

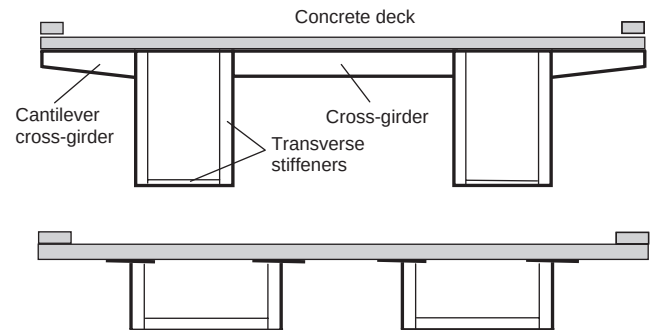


Figure 10.45 Composite box girders with and without a full width top flange

This figure shows bridges with two boxes. It is possible to design bridges with a number of smaller boxes and a composite deck, but the benefits of using boxes instead of plate girders are then reduced.

For the longest spans, a single box, either cable-stayed or with suspension support, is used. There are now many quite significant examples of box girder suspension bridges with spans of 1000m or more. The Humber bridge in the UK, for example, has a span of 1410m, while the Akashi Kaikyo bridge in Japan has a main span of 1991m. All longer-span bridges of this type have an orthotropic steel deck in place of concrete to minimise weight. The decks are very slender, multiply stiffened and aerodynamically shaped to minimise wind-induced drag and possible oscillations. An example of such a cross-section is shown in Figure 10.46.



Figure 10.46 Idealised example of slender box girder cross-section

The very large spans can be constructed by cantilever balanced construction moving out from a pier. New sections are lifted from the water by a temporary gantry sitting on the existing deck and are then bolted to the existing structure, as illustrated in Figure 10.47. The two halves of the bridge are finally joined together at midspan.

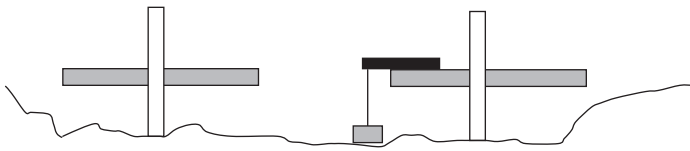


Figure 10.47 Construction process for very large spans

The webs of box girders are stiffened with intermediate transverse stiffeners and, unless stocky, with one or more longitudinal stiffeners. A sensible initial design assumption for one stiffener is to place it at one-third of the depth away from the compression flange, while two stiffeners can be placed at mid-depth and one-sixth of the depth from the compression flange. These locations can be adjusted to optimise the performance of the web plate panels. For a large box, the webs can have a number of longitudinal stiffeners, generally equally spaced down the inclined web. Cross-sections for simple horizontally stiffened girders are shown in Figure 10.48.

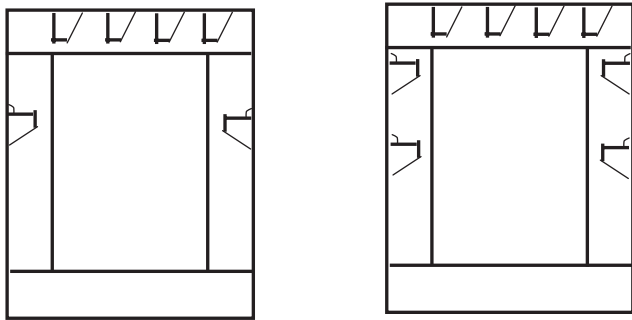


Figure 10.48 Simple longitudinally stiffened box cross-sections

For this type of construction, the breadth-to-depth ratio would tend to be in the range 3:1 to 1:2, depending on the bridge configuration.

While longitudinal stiffeners are not required in the tension flange for the main loading components, they are often provided for robustness, unless the flange is particularly stocky.

If the box has a wide flange and is non-composite, the top flange will be fabricated as an orthotropic steel deck. The deck has to carry wheel loads as well as the main in-plane girder loading and therefore has to have closely spaced longitudinal stiffeners. These are of bulb flat, angle or closed-section form. The latter can be trough or V-shaped stiffeners, which have the advantage of increased support for the flange plate, as well as high torsional rigidity when welded to the deck. Figure 10.49 shows the range of stiffener sections that may be used in box girders.

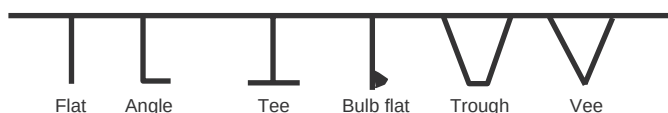


Figure 10.49 Range of stiffener cross-sections used in stiffened plating

All may be used for flange longitudinal stiffeners, although T-sections tend to be used for transverse stiffeners and flats have the disadvantage of very low torsional stiffness. In all cases, longitudinal stiffeners should be threaded through and welded to cut-outs in the transverse stiffeners so as to maintain the integrity of the longitudinal load path. This is particularly important for compressive stress, where eccentricities should be avoided. Figure 10.50 shows the configuration of a typical orthotropic deck, fabricated using open-section stiffeners.

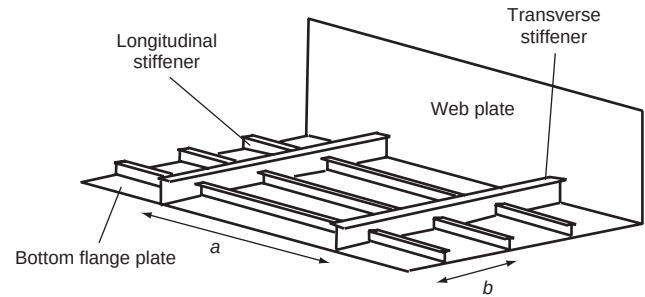
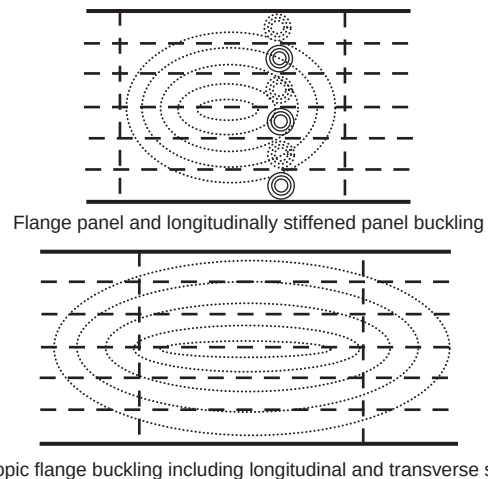


Figure 10.50 Arrangement of stiffeners in a typical orthotropic deck

The buckling modes for such a deck include buckling of the subpanels between the longitudinal stiffeners, buckling of the stiffened flange between cross-girders and buckling of the overall flange with a longitudinal wavelength longer than the transverse stiffener spacing. In the latter case, the cross-girders will deform transverse to the flange. These buckling modes are shown in Figure 10.51.



Orthotropic flange buckling including longitudinal and transverse stiffeners

Figure 10.51 Possible buckling modes for a stiffened flange

All buckling modes can occur either upwards or downwards. In the case of modes involving the stiffeners, this will mean that both the plate and the stiffener outstand could be in local bending compression (in addition to the direct compressive stress), so failure can occur by buckling of either element or by yield in tension. As will be seen later, this leads to both

deformation directions being checked in the design process. It also leads to the possible local buckling of the plates of the stiffener sections themselves, although this is normally eliminated from the design by making these sub-elements stocky. The approach generally taken is where slenderness limits are defined for the elements of the stiffener cross-sections, which provide a factor of safety against elastic critical buckling to allow for non-linear effects.

Figure 10.52 shows the buckling of the panels between longitudinal stiffeners and the buckling of longitudinal stiffeners between cross-frames.

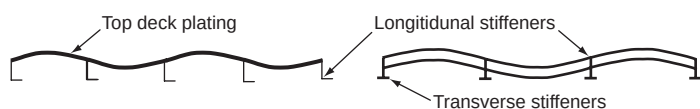


Figure 10.52 Buckling of plating and longitudinal stiffeners

It can be seen that, in both cases, adjacent elements will deform in opposing directions, because of continuity. The weakest mode will determine the failure capacity. The torsional rigidity of the stiffeners will determine whether they remain straight or whether they will twist to remain orthogonal to the plating or longitudinal stiffeners. The former case is shown in the figure.

Buckling of the webs is similar to the case of plate girders, although the increased use of longitudinal stiffeners adds complexity. The buckling restraint provided by the flanges is also significantly smaller, particularly where these are longitudinally stiffened and hence very slender. For any girder where longitudinal stiffeners are present in either web or flange, the tension field approach is normally not allowed, resorting in a more complex approach that considers the strength of each sub-panel in detail. This is described later. The reasons for this are twofold. First, the large shear deformations needed to achieve full tension field action could build in significant eccentricities and destabilise a slender stiffened compression flange. Second, the tension field model for a longitudinally stiffened web is complex, with the diagonal buckles rotating from the subpanel buckles through the longitudinal stiffeners themselves. This form of behaviour was precluded from the codes.

The final main elements that are needed in a box girder are the intermediate cross-frames or diaphragms and the bearing diaphragms. Intermediate cross-frames (Figure 10.53) are needed at regular intervals to maintain the cross-sectional

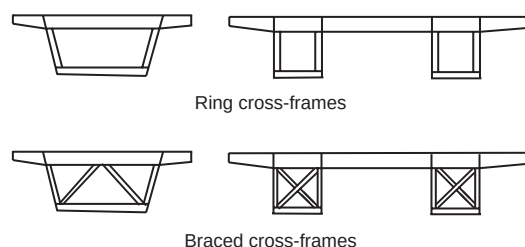


Figure 10.53 Intermediate ring frames and braced cross-frame

shape; they also have a role in supporting the flange longitudinal stiffeners, providing nodal lines to limit the buckling effective length of the longitudinally stiffened flange. The location of these cross-frames or transverse stiffeners coincides with transverse stiffeners in the webs and tension flange, so that a welded ring exists that strengthens the cross-section. As these cross-frames are normally placed around $1.5\text{--}3\times$ the width of the flange apart, additional web transverse stiffeners are normally placed between them to enhance web buckling capacity.

At less regular intervals, the cross-frames may be braced to add additional strength to the cross-section to prevent distortion of the cross-sectional shape. This is also shown in Figure 10.53.

A more complex cross-frame arrangement combined with cross-girders and web stiffeners is shown in Figure 10.54.

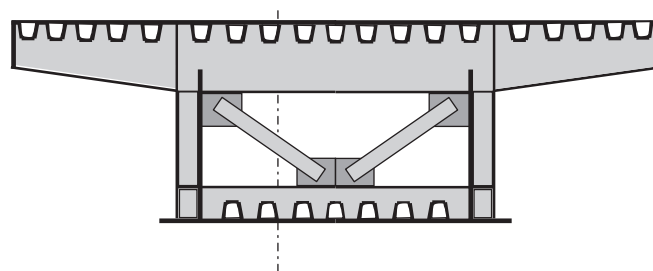


Figure 10.54 Example of a braced cross-section

For larger more slender boxes, plated intermediate diaphragms may be used to fulfil this function. Such diaphragms are not dissimilar to bearing diaphragms but are generally lighter, with some stiffening when slender, but without the load distribution stiffening needed in a bearing diaphragm because of the high compression above the bearings.

There is a substantial range of possible geometries for bearing diaphragms, depending on the size and shape of the cross-section, but also on the location of the bearings relative to the centre line of the box and the webs. A simple bearing diaphragm for a small stocky box is shown in Figure 10.55. The full depth stiffeners prevent the plated diaphragm buckling from the compression above the bearings and also help the shear distribute from the webs into the bearings. Because of the stocky nature of the diaphragm, there is no need for horizontal stiffeners because of the limited horizontal stress resulting from transverse bending.

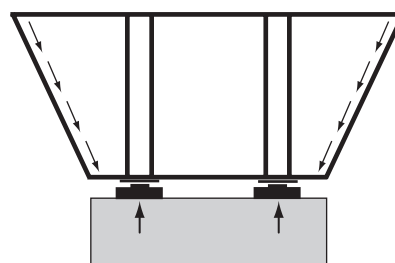


Figure 10.55 Simple plated bearing diaphragm for a stocky box section

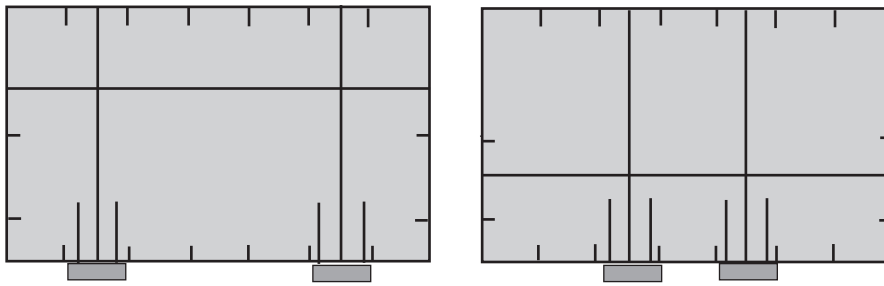


Figure 10.56 Diaphragms illustrating different stiffener arrangements because of bearing location

The two diaphragms shown in Figure 10.56 illustrating the possible effect of bearing location. In the diaphragm on the left, the bearings are located near the webs, introducing transverse bending and putting the top of the diaphragm in horizontal compression. The horizontal stiffener has been added to prevent buckling of the central plate caused by this stress combined with the vertical compression and shear. In the diaphragm on the right, the reverse is true, with the bearings close together and horizontal compression from transverse bending present in the bottom of the diaphragm. The horizontal stiffener is therefore placed in this critical area. Both diaphragms have stub bearing stiffeners to prevent local plate crippling, as well as full depth stiffeners for shear transfer.

Bearing location is often determined by the location and size of the piers, which are themselves influenced by the general environment of the bridge, the nature of the ground, the bridge height, and so on.

The diaphragm of Figure 10.57 shows a complex multistiffened diaphragm, which has widely spaced bearings, a wide slender box and an access hole for maintenance and inspection purposes. Because of the widely spaced bearings, the diaphragm is again horizontally stiffened near the top flange. There is also a complex series of stiffeners around the large opening to prevent the centre of the diaphragm plate from buckling. The design of such a diaphragm is not unlike a multistiffened web as it is essentially a deep beam with high levels of shear, bending and transverse compression. The stress state in each panel must be considered separately to prevent individual panel buckling.

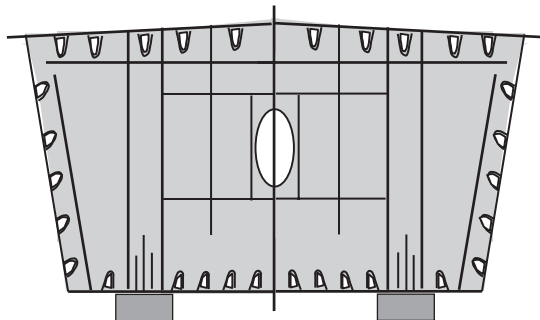


Figure 10.57 Example of a complex multistiffened diaphragm

It makes sense in design to keep diaphragms relatively stocky to avoid complexity. Weight is not a significant issue for a support diaphragm and fabrication costs can be disproportionate if slenderness is not required because of the size of the box.

Box girder design rules

The Eurocode rules for plated structures are published in Eurocode 3 Part 1.5 (BSI, 2019). While having been in existence for a number of years, the rules within BS 5400 Part 3 (BSI, 2000) are probably still the most comprehensive available in the context of complex stiffened plated elements.

This section deals in some detail with the principles behind both the Eurocode Part 1.5 rules and the BS 5400 Part 3 rules, presenting some of the main design equations. However, both codes are complex in this area, particularly the British Standard with many minor rules providing important additional requirements. In some areas, these are important in providing the global conditions under which the main rules apply. In design, it is therefore important to use the source documents in order to ensure the overall integrity of the design process. The totality of the design rules is too complex to present here. Other design codes deal with the area of box girder design, for example, codes issued in the USA, but in general these follow similar principles and often relate to the same source research and development material.

The overall approach in the two codes is somewhat different. Standard BS 5400 Part 3 essentially designs webs and flanges separately (while allowing for contributions to the web shear strength from the flange, the possibility of stress shedding from the web to the flange and the influence of boundary conditions). Eurocode 3 Part 1.5 defines an effective section for the (box or plate) girder for bending, which is then checked against the applied bending moment. Clauses provide effective sections for the compression flange and for stiffeners in the compression zone of the web. A separate check is conducted for web shear capacity, again allowing for a flange contribution. The two governing equations are defined as follows.

For design for bending:

$$\eta_1 = \frac{\sigma_{x,Ed}}{f_{yd}} = \frac{M_{Sd}}{f_{yd} W_{eff}} \leq 1.0 \quad (10.22)$$

For design for shear:

$$\eta_3 = \frac{\tau_{Ed}}{\chi_v \frac{f_{ywd}}{\sqrt{3}}} = \frac{V_{Sd}}{\chi_v \left(\frac{f_{ywd}}{\sqrt{3}} \right) b t} \leq 0 \quad (10.23)$$

Here, W_{eff} is the effective section modulus for bending and χ_v is the reduction factor for shear. These are described later. More complex expressions are given in the Eurocode for use in the presence of axial force and transverse stresses.

These equations then need to be checked in an interaction equation if the value of η_3 exceeds 0.5. The interaction equation is

$$\eta_1 + \left[1 - \frac{M_{f,Rd}}{M_{pl,Rd}} \right] (2\eta_3 - 1)^2 \leq 1.0 \quad (10.24)$$

where $M_{f,Rd}$ is the design plastic moment resistance of the cross-section consisting only of the flanges (basing it on the smaller flange size) and $M_{pl,Rd}$ is the plastic resistance of the whole cross-section not allowing for any reduction due to slenderness. In Equation 10.24, η_1 may be evaluated using gross section properties.

Stiffened compression flanges in BS 5400

The main elements of a stiffened compression flange are shown in Figure 10.50 and the associated buckling modes are shown in Figures 10.51 and 10.52.

As indicated previously, a number of buckling modes must, in principle, be considered:

- buckling of the subpanels between the stiffeners
- buckling of the longitudinally stiffened panel between the cross-frames
- orthotropic panel buckling of the entire stiffened flange, including the cross-frames
- local buckling of the elements of the stiffeners themselves.

Standard BS 5400 Part 3 deals with these in different ways. Buckling modes 3 and 4 are not considered by the designer directly. They are precluded by placing design requirements on the individual stiffeners. In the case of transverse stiffeners, requirements for the stiffness and strength of the stiffeners are introduced in the code, while in the case of stiffener cross-section, local buckling slenderness limits are placed on all the individual components of the outstands.

The panel buckling strength curves of Figure 10.40 are used in the code to define a buckling effective width for incorporation in the behaviour of the longitudinal stiffeners. The main buckling check in the code therefore deals with the buckling of the longitudinally stiffened panels. This is not, however, dealt

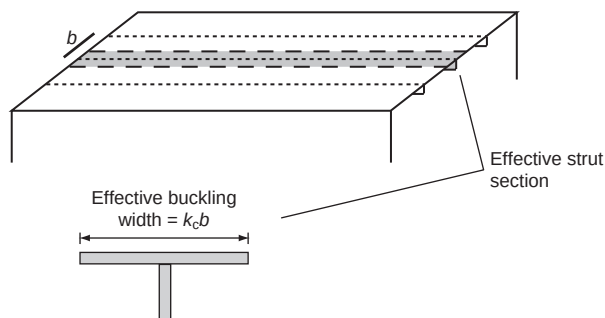


Figure 10.58 Representation of a longitudinally stiffened flange as a series of isolated struts

with as a stiffened plate but as a series of isolated struts spanning between the cross-girders. The struts have a plate effective width of K_c multiplied by the width of the plate panel b . This is illustrated in Figure 10.58.

Representation of the stiffened plate in this way ignores the transverse continuity with the control of flange deformations exerted by the webs. This can be significantly conservative in the case of slender flanges with a significant number of stiffeners but the approach is often used because of its relative simplicity. Other methods are available that consider the buckling of the stiffened orthotropic plate. These tend to be based on an elastic orthotropic plate buckling theory, with ultimate collapse related to simple yield requirements.

The two buckling directions for the effective strut section acting as a column in compression with an effective length equal to the cross girder spacing are dealt with by two column equations in the code.

The first equation deals with the stiffener outstand in compression, as shown in Figure 10.59:

$$\sigma_a + 2.5\tau_1 k_{s1} \leq k_{t1} \sigma_{ys} \quad (10.25)$$

where σ_a is the longitudinal compressive stress at the centroid of the effective section of the strut (positive), τ_1 is the in-plane shear stress in the flange plate owing to torsion (positive) and σ_{ys} is the design yield stress of the stiffener material. Again, it should be emphasised that factors of safety have not been included.



Figure 10.59 Column buckling, producing additional compression in the stiffener outstand

The term K_{s1} is a factor that reduces the effect of shear on the buckling of the strut for low column slenderness and k_{t1} is the column buckling coefficient. Values of k_{s1} and k_{t1} are given in Figure 10.60.

The abscissa in Figure 10.60 is the column slenderness, where r_{se} is the radius of gyration of the effective strut section. The value of λ for buckling in this direction is related to the yield stress of the stiffener. The term η is an effective imperfection factor for the column, which, for buckling with the stiffener outstand in compression, is given by

$$\eta = y_0 \Delta / r_{se}^2 \quad (10.26)$$

The term Δ is made up from a number of components of imperfection. The first is a simple geometric out-of-straightness as a function of the column length, the second allows for the eccentricity in the axial load caused by variation in stress down the outstand caused by global bending of the box and the third relates to any vertical curvature of the bridge over the length $l(e_t)$:

$$\Delta = \frac{l}{625} + \frac{r_{se}^2}{2y_{Bs}} + \frac{e_t}{2} \quad (10.27)$$

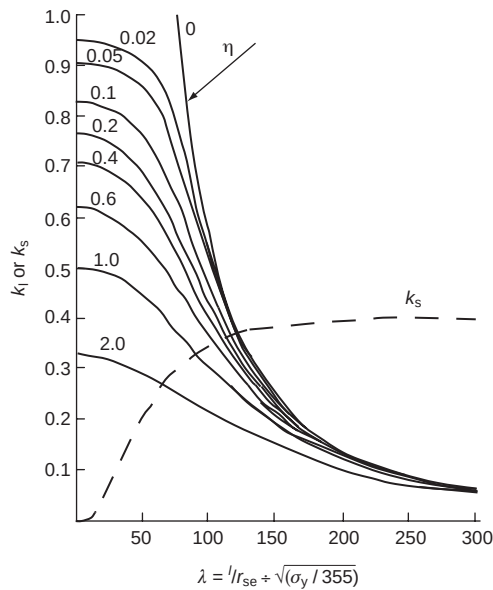


Figure 10.60 Factors for the design of longitudinal flange stiffeners

The distance y_{BS} is taken from the neutral axis of the box to the centroid of the effective strut section and is illustrated in Figure 10.61.

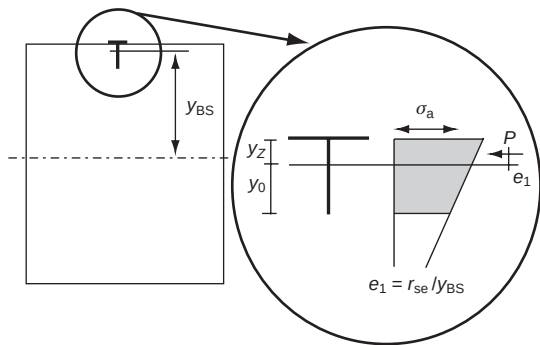


Figure 10.61 Definition of y_{BS}

It is, in fact, illogical to use the value of y_{BS} in the same way for both directions of flange buckling. Equation 10.25 thereby allows this direction of strut buckling to be checked. As a direct equivalent, Equation 10.28 provides the basis for checking buckling of the effective strut section with the plate, rather than the outstand, in additional compression, as shown in Figure 10.62:

$$\sigma_a + 2.5 k_1 k_2 \leq k_{12} \sigma_{ye} \quad (10.28)$$

where σ_{ye} is the design yield stress of the plate material, reduced to allow for coexistent shear stress:

$$\sigma_{ye} = \sqrt{\sigma_{yt}^2 - 3\tau^2} \quad (10.29)$$

where τ is the in-plane torsional shear stress plus half the complementary shear stress at the flange edge due to vertical shear of the girder. An averaging across the flange width is assumed

for the latter. The process is directly equivalent to that defined previously, but with appropriate terms in the equations.

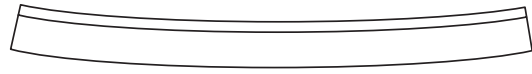


Figure 10.62 Column buckling, producing additional compression in the plate

The imperfection factor of Equation 10.26 for use in Figure 10.60 becomes

$$\eta = y_z \Delta / r_{se}^2 \quad (10.30)$$

where y_z is the distance from the midplane of the flange plate to the centroid of the effective strut section. The term λ in Figure 10.60 for establishing k_{12} is now related to the plate yield stress. In addition to the column buckling requirements, BS 5400 also has clauses relating to yielding of the section.

The design of transverse stiffeners for the stiffened flange, unlike the web, is based on a beam model with the beam spanning between the webs (length B). Stiffness and strength requirements are provided for the transverse members to ensure that they act as nodal lines for the longitudinal stiffener buckling.

The strength requirement is based on an effective section of the stiffener (generally of a T-section) plus an associated width of flange plate in the longitudinal direction of half the transverse stiffener spacing for a non-composite top deck or one-seventh of the main web spacing for the bottom flange in the hogging moment region of a continuous span. The former value allows for transverse in-plane flange compression.

The loading applied to the beam is a uniformly distributed load along the beam span, which equals the total axial compressive load carried by the flange or unit flange width divided by 200, distributed along the length of the beam. This is illustrated in Figure 10.63.

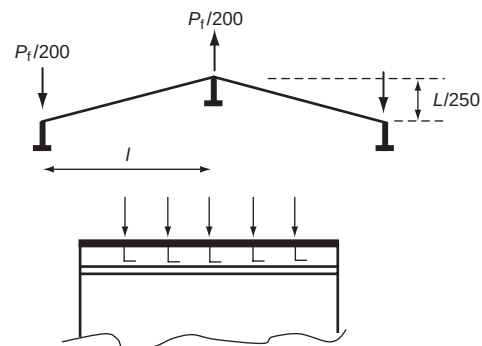


Figure 10.63 Loading applied to transverse flange stiffener

Other loading is added to this, including any directly applied loads, the effect of any bridge vertical curvature and also restraint against distortion and effect of concrete shrinkage (if present). The code gives details of how to evaluate these other components.

The standard beam strength equations, $M_D = Z_{pe}\sigma_{lc}$ and $M_D \leq Z_{xc}\sigma_{lc} \leq Z_{xt}\sigma_{yt}$, depending on whether the stiffener section is compact, should be used. As a hot rolled section will almost certainly be used for the stiffener, it is likely that the compact requirements will be met and it is sensible to design the beam as a plastic member in this context.

The stiffness requirement for the flange transverse stiffeners in BS 5400 is based on satisfying

$$I_{be} \geq \frac{9\sigma_f^2 a B^4 A_f^2}{16 K E^2 I_f} \quad (10.31)$$

where I_{be} is the required effective inertia of the transverse member with an effective flange width equal to one-quarter of the web spacing (not greater than the transverse stiffener spacing), σ_f is the average longitudinal compressive stress in the flange, A_f is the total flange area including longitudinal stiffeners per length B , I_f is the inertia of the stiffened flange about its centroidal axis and K is a factor that varies depending on the location of the cross girder and the type of stiffeners. For example, where the beam spans between two box webs and the flange longitudinal stiffeners are of open cross-section, $K=24$.

Other values exist for the parameters in Equation 10.31; for example, the value of K will be different if the beam cantilevers out beyond the box. Again, the code provision is relatively complex.

As mentioned already, there is no need to check the buckling of individual stiffener plate components because the code limits their slenderness to ensure that they do not buckle. This is because buckling of free edge elements, as discussed previously in terms of plate behaviour, is a relatively unstable phenomenon, with the possibility of significant loss of stiffness. Examples of the slenderness limits applied for flat stiffeners are

$$(h_s/t_s)\sqrt{(\sigma_y/355)} \leq 10 \quad (10.32)$$

where h_s/t_s is the outstand slenderness, and for angle stiffeners are

$$(b_s/t_s)\sqrt{(\sigma_y/355)} \leq 11 \quad (10.33)$$

and

$$(h_s/t_s)\sqrt{(\sigma_y/355)} \leq 7 \quad (10.34)$$

where b_s is the stiffener flange width and h_s is the stiffener depth.

In presenting these design rules, no mention has been made of the phenomenon of shear lag. There is a need to account for it in particular circumstances for the serviceability limit state, notably in the context of cable-supported structures.

Shear lag occurs as a non-linear distribution of direct stress across the flange caused by the in-plane shear flexibility of the flange plating. It is an elastic phenomenon and should not be confused with the non-linear stress distribution caused by out-of-plane buckling. Certain codes assume that, at the collapse limit state, the shear lag induced stress variation redistributes itself through plastic redistribution, although research shows that there is a limit to this redistribution and other codes do

allow for shear lag at collapse in certain circumstances. The appearance of the stress variation is demonstrated in Figure 10.64.

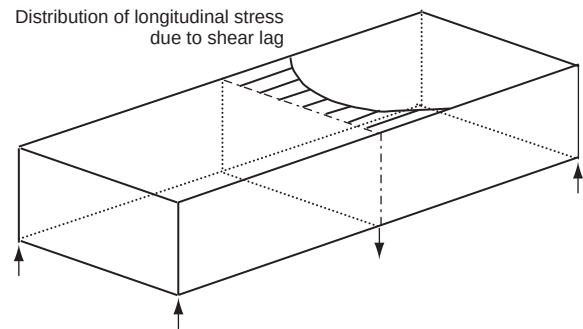


Figure 10.64 Non-linear flange stress distribution caused by shear lag

The phenomenon of shear lag comes about because the direct stresses in a flange are introduced through shear along the web-flange boundary, owing to vertical bending of the web. This stress has to be transferred across the width of the flange through the in-plane shear stiffness of the flange plate. For a typical beam with low width to thickness ratio, there are no significant stress variations across the width but for a flange that is wide compared with the bridge span, a significant variation can occur with central stresses being as low as 20% or less of the edge compression. In effect, plane sections no longer remain plane and simple bending theory no longer applies.

A table of shear lag effective widths for use in serviceability calculations is reproduced here to illustrate the magnitude of the effect and its dependence on a number of parameters. The table presented is for a simply supported girder for a stiffened flange between two webs.

In Table 10.3, α is the degree of longitudinal stiffening equal to the total area of the flange stiffeners divided by the total area of the flange plate.

$B/2L$	Mid-span $\alpha = 0$	Mid-span $\alpha = 1$	Quarter-span $\alpha = 0$	Quarter-span $\alpha = 1$	Support $\alpha = 0$	Support $\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.98	0.97	0.98	0.96	0.84	0.77
0.10	0.95	0.89	0.93	0.86	0.70	0.60
0.20	0.81	0.67	0.77	0.62	0.52	0.38
0.30	0.66	0.47	0.61	0.44	0.40	0.28
0.40	0.50	0.35	0.46	0.32	0.32	0.22
0.50	0.38	0.28	0.36	0.25	0.27	0.18
0.75	0.22	0.17	0.20	0.16	0.17	0.12
1.00	0.16	0.12	0.15	0.11	0.12	0.09

Table 10.3 Shear lag factors

It can be seen that the shear lag effective width Ψ is heavily influenced by the parameter $B/2L$, which is the half width of

the flange divided by the bridge span. For a continuous bridge, L would normally be defined as the distance between points of contraflexure but this is allowed for in the code by providing appropriate values of Ψ in a separate table in terms of L (defined still as the beam span). The effective width of the flange is ΨB , which can also be used to define the ratio between edge and centre flange stresses.

The influence of location along the span is relatively modest for very wide bridges, where the stress has little opportunity to continue to develop across the width, but is very significant for longer spans, where it becomes more uniform away from the supports. The stiffening ratio has a moderate influence, as the lumped stiffener mass reduces the degree to which the shear stiffness can redistribute the stress; hence, large stiffening ratios result in smaller values of effective width.

Stiffened compression flanges in Eurocode 3 Part 1.5

There is a requirement to allow for shear lag at ultimate collapse in Eurocode 3 Part 1.5 (a separate requirement is also specified for serviceability considerations). The shear lag reduction factor β^k is multiplied by an effective section, which allows for buckling to give an effective flange area for compression flanges:

$$A_{\text{eff}} = A_{\text{c,eff}} \beta^k \quad \text{but} \quad A_{\text{eff}} \geq \beta A_{\text{c,eff}} \quad (10.35)$$

where $A_{\text{c,eff}}$ is the plate buckling effective area and β is the shear lag effective width factor. The values of β and κ are given in the code; for a simply supported span and a stiffened compression flange between two webs, κ is given by

$$\kappa = \alpha_0 b_0 / L_e \quad (10.36)$$

where

$$\alpha_0 = \left(1 + \frac{A_{\text{sl}}}{b_0 t}\right)^{0.5} \quad (10.37)$$

where b_0 is half the flange width, A_{sl} is the area of stiffening within this width (hence $A_{\text{sl}}/b_0 t$ is the flange stiffening ratio) and L_e is the girder span.

The shear lag effective breadth b_{eff} is equal to the flange width multiplied by β . For a simply supported span, the values of β are given by Table 10.4.

κ	β
≤ 0.02	1.0
0.02–0.07	$1/(1 + 6.4\kappa^2)$
> 0.07	$1/(5.9\kappa)$

Table 10.4 Values of β in Eurocode 3 Part 1.5 (BSI, 2019) for a simply supported span

The factor ρ for plate buckling has been defined in Equations 10.9 and 10.10. If the flange has no longitudinal stiffeners, the appropriate value $A_{\text{c,eff}}$ can then be defined:

$$A_{\text{c,eff}} = \rho A_c \quad (10.38)$$

This equation is also used to define the subpanel effective width to be used in association with the stiffeners (see later):

$$\rho_{\text{pan}} = \rho \quad (10.39)$$

In the presence of longitudinal flange stiffeners, $A_{\text{c,eff}}$ is given by

$$A_{\text{c,eff}} = \rho_c A_c \quad (10.40)$$

Here, A_c is the reduced area of the stiffened compression flange; A_c is the effective area of the stiffeners together with the total sub-panel area, reduced to allow for plate buckling:

$$A_c = A_{\text{sl,eff}} + \sum_c (\rho_{\text{pan}} b_{\text{c,pan}} t) \quad (10.41)$$

where $A_{\text{sl,eff}}$ is reduced in accordance with Equations 10.8 to 10.10 if local buckling affects the plates of the stiffener cross-section; σ_{cr} in Equation 10.8 is evaluated for the relevant boundary conditions (i.e. a plate with a free edge if an outstand element).

The term ρ_c allows for overall buckling of the longitudinally stiffened flange as a plate with a smeared stiffness, which allows for the stiffeners and buckling of a stiffener and effective width of plate as a strut between cross-frames. This code is hence more sophisticated than BS 5400 in that it allows for the orthotropic behaviour, while BS 5400 Part 3 presumes that the flange strength is based on that of the individual effective strut.

The final equation for ρ_c is

$$\rho_c = (\rho - \chi_c) \zeta (2 - \zeta) + \chi_c \quad (10.42)$$

where ρ is the reduction factor to allow for buckling of the plate of smeared stiffness, χ_c is the reduction factor for strut buckling and

$$\zeta = (\sigma_{\text{cr,p}} / \sigma_{\text{cr,c}}) - 1 \quad (10.43)$$

where $\sigma_{\text{cr,p}}$ is the elastic critical buckling stress of the smeared longitudinally stiffened plate and $\sigma_{\text{cr,c}}$ is the elastic critical buckling stress of the equivalent plate with longitudinal edge support removed.

The value of $\sigma_{\text{cr,p}}$ is evaluated from Equation 10.5, where b is the total flange width, t the flange plate thickness and k a buckling coefficient from orthotropic plate theory:

$$k \text{ is equal to } k_{\sigma,p} = \frac{((1+\alpha^2)^2 + \gamma)}{\alpha^2(1+\delta)} \text{ if } \alpha < (1+\gamma)^{0.25} \quad (10.44)$$

or

$$k_{\sigma,p} = \frac{2(1+\sqrt{1+\gamma})}{1+\delta} \text{ if } \alpha > (1+\gamma)^{0.25} \quad (10.45)$$

where $\gamma = (I_x/I_p) > 50$ is the second moment of area of the stiffened panel divided by the bending stiffness of the plate ($bt^3/(12(1-\nu^2))$ (b is the width of the stiffened plate), $\delta = A_{\text{sl}}/A_p$ is the gross area of all longitudinal stiffeners divided by the gross area of the plate bt and $\alpha = a/b > 1$.

— The term ρ is evaluated from Equation 10.9 or 10.10, with λ_p given by

$$\lambda_p = \sqrt{(\beta_a \sigma_y / \sigma_{\text{cr,p}})} \quad (10.46)$$

where β_a is the effective area of the stiffened plate, allowing for buckling of the subpanels, (A_{eff}) divided by the gross area of the stiffened plate (A).

The stress $\sigma_{\text{cr,c}}$ is evaluated as

$$\sigma_{\text{cr,c}} = (\pi^2 EI_x) / (A a^2) \quad (10.47)$$

where I_x is as defined previously, a is the transverse stiffener spacing and A is the gross area of the stiffened plate.

The reduction factor for strut buckling, x_c , is given by the column buckling expression in Eurocode 3 Part 1.1 with α replaced by $\alpha_c = \alpha_0 + (0.09 / (i e))$, where $i = \sqrt{I_x / A}$ is the radius of gyration of the stiffened plate, e is the larger distance from the centroid of the stiffened plate to the centroid of the stiffener section or the centre of the plating and α_0 is 0.34 for hollow-section stiffeners or 0.49 for open-section stiffeners. This gives an appropriate imperfection factor for an eccentrically stiffened plate, a bow of $a/500$ (in contrast to the values used for columns in Part 1.1). The appropriate column slenderness to use in evaluating the column strength is

$$\bar{\lambda}_c = \sqrt{(\beta_A \sigma_y / \sigma_{\text{cr,c}})} \quad (10.48)$$

$$x_c = 1 / \left[\phi + (\phi^2 - \bar{\lambda}_c^2)^{0.5} \right] \quad \text{but} \leq 1 \quad (10.49)$$

where

$$\phi = 0.5 \left[1 + \alpha (\bar{\lambda}_c - 0.2) + \bar{\lambda}_c^2 \right] \quad (10.50)$$

Stiffened webs in BS 5400

The design of transversely stiffened webs for girders with no longitudinal stiffeners has been dealt with in the context of plate girders and would be applied in the same way for a box. However, BS 5400 (BSI, 2000) requires the use of a different method for the design of webs when either the web or flange has longitudinal stiffeners, for reasons defined previously. For the latter case, the basis of the web design is consideration of the ultimate capacity of the individual web panels in the cross-section (a single pane if longitudinal stiffeners are present in the web). The shear capacity of the web, in the presence of other in-plane stresses, is the summation of the individual panel capacities in a vertical section (Figure 10.65).

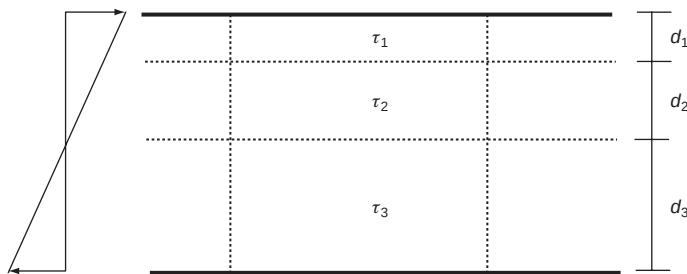


Figure 10.65 Basis for web design for a girder with longitudinal stiffeners

The total web shear strength is given by

$$V_D = (\tau_1 d_1 + \tau_2 d_2 + \tau_3 d_3 + \dots + \tau_n d_n) t_w \quad (10.51)$$

where n is the number of panels through the web depth. The values of the individual shear strengths are evaluated individually as the capacity of each panel in shear, allowing for the other direct stresses present.

The distribution of direct stresses for which the individual panel capacity is to be assessed is based on a simple linear subdivision of the total stresses evaluated for the web, based on simple elastic bending theory. Only the ratio of the individual stresses is initially needed, that is, the ratio between bending stress, uniform direct compression or tension and shear (which is assumed to be uniform down the web depth at this stage) is needed. A representation of these stresses is shown in Figure 10.66.

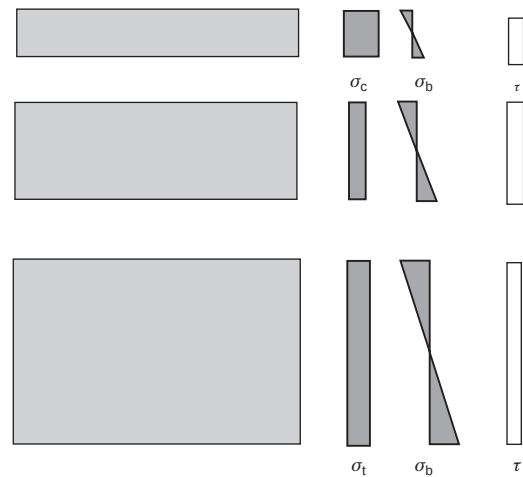


Figure 10.66 Individual stress states for each of the web panels

It can be seen from Figure 10.66 that the direct stresses in the top panel are mainly compressive with a small bending component, in the centre panel they are largely bending with a small compressive component and in the bottom panel they are an intermediate level of tension and bending. The slope of the bending stress in all three panels is identical to the slope of the original bending stress distribution, while the combination of compression or tension and bending at each of the panel boundaries adds up to the direct stress level at that location in the original distribution.

The stresses, of course, must be evaluated on an initial trial of a web thickness and stiffener locations (and a trial flange geometry), as with all designs. Once the web panels have been checked, the thickness of the web or the locations of transverse and lateral stiffeners can be adjusted to optimise the design. There is also a need to iterate between flange and web designs.

To evaluate the collapse stress combination of each panel, the strengths of the panels under the individual stress components are determined using the curves of Figures 10.28 and

10.31 for the cases of compression, shear and bending, respectively. Values of k_{c1} , k_{q1} , k_{b1} , k_{c2} , k_{q2} , k_{b2} , k_{q3} and k_{b3} are therefore obtained. While the code does not make this clear, it is logical to take the value of k_{c3} as 1.0 because the panel is in tension. If a smaller value is taken, it is potentially non-conservative because the effect is beneficial as a negative term in the interaction equation, which is then artificially large.

In evaluating these coefficients, an assumption has to be made about the boundary condition for each panel (the degree of in-plane restraint). The code defines internal panels as being restrained and panels abutting a flange as being unrestrained, unless the latter satisfy conditions relating to the slenderness of the edge panel or the bending restraint provided by the flange to the web (to anchor transverse in-plane boundary stresses). The edge panel either has to have a slenderness λ less than 24 or a value of m_{fw} greater than (including factors of safety)

$$\left[\sigma_{yf}^2 / (\sigma_{yf}^2 - \sigma_f^2) \right] \times (0.00025(\lambda - 24)) \quad (10.52)$$

for $24 \leq \lambda \leq 84$, where $\lambda = b/t_w \sqrt{(\sigma_{yw}/355)}$ and $m_{fw} = (\sigma_{yf} b_{fe} t^2 f / 2 \sigma_{yw} b^2 t_w)$, a measure of the out-of-plane bending stiffness of the flange compared with the thickness of the web. The term b_{fe} is an effective width of flange used as the section that restrains transverse stress $= 10 t_f \sqrt{(355/\sigma_{yf})}$ (but not greater than half the web spacing).

Each panel is then checked in the interaction equation presented previously as Equation 10.14 (which also allows for any vertical compression). The form given in the code is

$$m_c + m_b + 3 m_q \leq 1 \quad (10.53)$$

where

$$m_c = \sigma_1 / (\sigma_{yw} k_1 (1 - \rho)) \quad (10.54)$$

$$m_b = (\sigma_b / (\sigma_{yw} k_b (1 - \rho)))^2 \quad (10.55)$$

$$m_q = (\tau / (\sigma_{yw} k_q))^2 \quad (10.56)$$

and $k_1 = k_c$ for the appropriate curve. The value of 3 in Equation 10.53 comes from the use of σ_{yw} in Equation 6.55 rather than τ_y in Equation 6.14 ($\sigma_{yw} = \sqrt{3}(\tau_y)$ from the von Mises yield criterion).

The function $(1 - \rho)$ in Equations 10.54 and 10.55 allows for the differential benefit of stress redistribution in the context of restrained and unrestrained panels. The code allows shedding of up to 60% of direct stresses from any or all web panels into the flanges (which can carry them more efficiently), as long as the overall axial and moment equilibrium is maintained. The stresses used in Equations 10.54 to 10.56 are the reduced levels of stress; hence, benefit ensues from the interaction equation for the particular panel being considered. The value of $\rho = 0$ for restrained panels therefore has no effect but equals the proportion of stress shed for unrestrained panels, eliminating any benefit in stress shedding for that panel in terms of the buckling criterion (the reduced value of stress is used in the yield criterion within the rules). This limitation was introduced because

of concerns about the strain discontinuities required to introduce this stress differentiation in the context of an unrestrained panel, with its less stable post-peak behaviour. If stress shedding is required, the provision of a narrow edge panel is worth considering, to ensure it is restrained (if not already satisfying the restraint requirements) as this can improve the economy of the design. This stress shedding concept is an interesting one in that it goes some way towards the simple design process allowed in Eurocode 3 Part 1.1 for plate girders, whereby webs carry the shear and the flanges carry all the bending moments. With reduced in-plane stress levels, the web panels can carry greater shear stresses, leading to a higher shear capacity for the girder but the flange, of course, has to be designed for the increased bending moment in order to maintain equilibrium.

Once each of the panels, with or without stress shedding, has been checked in the interaction equation design, iterations will be required. These depend on the outcome of the panel buckling factors. For example, if the value from Equation 10.53 is greater than 1 in all cases, the whole of the web is underdesigned and either the web thickness (or web depth) or the number of web longitudinal stiffeners must be increased. In certain circumstances, if the overstress is modest, the transverse stiffener spacing can be reduced, which can have an effect on the shear buckling resistance. Clearly, if all are more than marginally less than 1, the web is overdesigned and savings can be made using the converse approach. If the result varies between panels (i.e. some factors greater than 1 and others less), it is possible to improve the efficiency of the design by moving the location of the longitudinal stiffeners to improve the balance of the resistance of each of the panels. Panels with factors greater than 1 should have their breadth reduced, and so on. Clearly, as with all design work, there is a practical limit to the value of endless iteration.

Design of web stiffeners in BS 5400 Part 3

The design of transverse web stiffeners has been dealt with previously, in the context of plate girders. The only significant difference is the inclusion of the longitudinal stiffener area (when present), which has been incorporated into Equation 10.21.

The design of longitudinal web stiffeners is based on similar principles. The longitudinal stiffener is designed as a column spanning between transverse web stiffeners with an effective section comprising the stiffener plus an effective width of web in the transverse direction equal to $32(1 - \rho)$ multiplied by the web thickness (as for transverse web stiffeners other than for the inclusion of the stress shedding parameter). The loading on the stiffener relates to the axial load present in the web, together with an allowance for shear. The design equation is

$$\sigma_{se} \leq \sigma_{ls} \quad (10.57)$$

where σ_{ls} equals the column strength obtained from a column design curve in the code.

The slenderness of the column $\lambda = a/r_{se}\sqrt{(\sigma_{ys}/355)}$, where r_{se} is the radius of gyration of the effective stiffener section, is

$$\sigma_{se} = k'_s \sigma_1 + \left(2.5\tau + \frac{a^2}{b^2}\sigma_2\right) \frac{bt_w k_s}{A_{se}} \quad (10.58)$$

where $k'_s = 1$ for continuous longitudinal stiffeners, σ_1 is the longitudinal stress at the stiffener, σ_2 is a transverse compression stress when present, τ is the average web shear stress at the location of the stiffener, K_s is a reduction factor equivalent to that shown in Figure 10.60 and used in Equation 10.25 and A_{se} is the area of the effective stiffener cross-section.

Stiffened webs in Eurocode 3 Part 1.5

The design of webs in Eurocode 3 Part 1.5 (BSI, 2019) can follow the process defined for compression flanges, although it is somewhat more complex because the stress gradient down the web must be allowed for and buckling factors, used to define the effective section, only apply in the compression zone of the web. The modified equations are not presented here but are available in the Eurocode.

A simpler method is available for webs with only one or two longitudinal stiffeners in the compression zone, the common case for all but very large boxes, which avoids calculating $\sigma_{cr,p}$ for the actual geometry of the web. The basis of the method is to define a gross area for a fictitious column, which is the gross area of the stiffener A_{sl} together with associated plating. If a subpanel is fully in compression, one-half of the subpanel width is used; if stresses change to tension in the subpanel, one-third of the compression width should be used. This gross area is then reduced to allow for buckling by evaluating reduction factors for the plate elements using Equation 10.9 or Equation 10.10, with λ_p replaced by

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{(\sigma_{comm,Ed}/\sigma_y)} \quad (10.59)$$

where $\sigma_{comm,Ed}$ is the maximum compressive stress in the gross cross-section of the plate.

The critical buckling stress $\sigma_{cr,p}$ is then given by

$$\sigma_{cr,p} = \frac{1.05E\sqrt{(I_{sl}t^3b)}}{Ab_1b_2} \quad \text{if } a \geq a_c \quad (10.60)$$

or

$$\sigma_{cr,p} = \frac{\pi^2 EI_{sl}}{Aa^2} + \frac{Et^3ba^2}{4\pi^2(1-\nu^2)Ab_1^2b_2^2} \quad \text{if } a \geq a_c, \quad (10.61)$$

where A is the gross area of the effective column, I_{sl} is the second moment of area of the gross column cross-section about its centroidal axis, b_1 and b_2 are the distances from the centre line of the stiffener to the compression and tension flange, respectively, and $b = b_1 + b_2$.

If there are two stiffeners in the compression zone, they should first be considered separately and then a single stiffener with A and I_{sl} located at the position of the resultant axial force in the stiffeners should be used to evaluate $\sigma_{cr,p}$.

Using this elastic buckling stress in Equation 10.46 with A and A_{eff} relating to the compressed part of the web to give λ_p ,

and then Equation 10.9 or Equation 10.10 to give ρ , gives the first reduction factor of Equation 10.42.

The column buckling stress $\sigma_{cr,c}$ may be evaluated allowing for the stress gradient by the use of an effective length factor given in Part 2 of the Eurocode or Equation 10.47 may be conservatively used. Equations 10.49, 10.42 and 10.43 are then used, as for compression flanges, to define ρ_c .

If $\rho_c \sigma_y$ is greater than the average stress in the stiffened column ($\sigma_{c,Ed}$), the reduction factor ρ_c is applied to the area of the fictitious column, with plate widths reduced to allow for plate buckling as defined previously; otherwise, the effective area is further reduced by multiplying the effective area by $\sigma_y/\sigma_{c,Ed}$.

The reduction factor for shear χ_v for use in Equation 10.23 comprises contributions from both web and flanges:

$$\chi_v = \chi_w + \chi_f \leq \eta \quad (10.62)$$

The contribution χ_w depends on whether the end posts are rigid. A rigid end post essentially requires two pairs of transverse stiffeners on either side of the web, a distance at least one-tenth of the web depth apart, with the first stiffener above the centre line of the bearing and the second outside the span of the beam. The section modulus of each stiffener should be at least $4h_w t^2$, where h_w is the web depth for bending out of the plane of the web or if a flat stiffener it should have an area of $4h_w t^2 e$, where e is the stiffener spacing.

The value of χ_w is given in Table 10.5; $\eta = 1.2$ for normal strength steels multiplied by a yield stress factor.

$\bar{\lambda}_w$	Rigid end post	Non-rigid end post
$< 0.83/\eta$	η	η
$0.83/\eta \leq \bar{\lambda}_w$	$0.83/\bar{\lambda}_w$	$0.83/\bar{\lambda}_w$
≥ 1.08	$1.37/(0.7 + \bar{\lambda}_w)$	$0.83/\bar{\lambda}_w$

Table 10.5 Values of χ_w

The value of $\bar{\lambda}_w$ depends on the type of web stiffening present; for a web with intermediate transverse stiffeners or longitudinal stiffeners, the expression for $\bar{\lambda}_w$ is

$$\bar{\lambda}_w = \frac{b_w}{37.4t\epsilon\sqrt{k_t}} \quad (10.63)$$

where

$$k_t = 5.34 + 4.00(h_w/a)^2 + k_{rst} \quad \text{when } a/h_w \geq 1 \quad (10.64)$$

$$k_t = 4.00 + 5.34(a/h_w)^2 + k_{rst} \quad \text{when } a/h_w < 1 \quad (10.65)$$

and

$$k_{rst} = 9\left(\frac{h_w}{a}\right)^2 \left(\frac{I_{sl}}{t^3 h_w}\right)^{0.75} \quad \text{but } \geq \frac{2.1}{t} \left(\frac{I_{sl}}{h_w}\right)^{\frac{1}{3}} \quad (10.66)$$

where I_{sl} is the second moment of area of the longitudinal stiffener about the web (the sum of the web stiffener contributions where there are more than one).

The contribution from the flanges χ_f is reduced to allow for applied bending and is given by

$$\chi_f = \frac{b_f t_f^2 \sigma_{yf}^{\sqrt{3}}}{c t h_w \sigma_{yw}} \left[1 - \left(\frac{M_{Sd}}{M_{f,Rd}} \right)^2 \right] \quad (10.67)$$

where

$$c = \left[0.25 + \frac{1.6 b_f t_f^2 \sigma_{yf}}{t h_w^2 \sigma_{yw}} \right] a \quad (10.68)$$

If an axial force is present in the girder, there is a further reduction.

If the intermediate transverse stiffeners are required to act as rigid supports to internal panels, they should have a stiffness of

$$I_{st} \geq 1.5 h_w^3 t^3 / a^2 \quad \text{if } a/h_w < \sqrt{2} \quad (10.69)$$

$$I_{st} \geq 0.75 h_w t^3 \quad \text{if } a/h_w \geq \sqrt{2} \quad (10.70)$$

They should also be designed for an axial force, which is defined in the code.

Design of stiffeners in Eurocode 3 Part 1.5

In Eurocode 3 Part 1.5 (BSI, 2019), as for BS 5400 (BSI, 2000), transverse stiffeners in either the compression flange or web are designed to provide support for longitudinal stiffeners. Transverse stiffeners must satisfy stiffness and strength requirements. The effective section of the transverse stiffener includes an effective width of plate equal to $30\epsilon t$. This is almost identical to the effective section used by BS 5400, with the exception of the yield stress multiplier. All the following definitions relate to this effective section.

However, unlike the web requirements of BS 5400, the transverse stiffener is treated as a beam, both for stiffness and strength requirements, with a span equal to the web depth between flanges or webs, as appropriate, and an imperfection equal to the distance between transverse stiffeners divided by 300. The transverse stiffener may be loaded by a resolved component of the in-plane compressive stress, which is then checked against yield in the stiffener or a deflection increment limit of $b/300$. Alternatively, these may be satisfied by providing a second moment of area for the effective stiffener cross-section of I_{st} :

$$I_{st} = \frac{\sigma_m}{E} \left(\frac{b}{\pi} \right)^4 \left[1 + w_0 \frac{300}{b} u \right] \quad (10.71)$$

where

$$\sigma_m = \frac{\sigma_{cr,c} N_{Sd}}{\sigma_{cr,p} b} \left[\frac{2}{a} \right] \quad (10.72)$$

$$u = \frac{\pi E e_{\max}}{\sigma_y 300 b} \geq 1.0 \quad (10.73)$$

Here, $e_{\max}$ is the extreme fibre distance from the stiffener centroid and N_{Sd} is the largest design compressive force of an adjacent panel.

The code provides requirements for longitudinal stiffeners to prevent torsional buckling of the outstands.

Diaphragm design

The principles behind the design of diaphragms have been described earlier. In the code, the design is based around the checking of the conditions of the original subpanels, in a similar way to an arrangement of stiffened webs, in association with the design of the individual stiffeners, again in line with the principles applied for web stiffeners already discussed. The details within the code are too complex to present in summary form.

Connections

Introduction

The design of the connections forms an important part of the overall design of a bridge structure. Early in the project, advice should be sought from steelwork fabricators as to which factors are significant when minimising the cost of the connections. However, cost is not always the dominant factor for the bridge designer and careful attention must also be given to the strength and fatigue behaviour of the chosen connections. The bridge designer also has to consider other important factors, such as the optimum location of the joint, which connections are to be welded, which are to be bolted and, following on from this, which connections are to be made in the fabricator's workshop and which connections are to be undertaken on-site. When considering the location of joints, it is desirable to position the joints at points of contraflexure or areas of low stress. Also, consideration must be given to optimising the total number of joints, taking into account transportation to site, and the weight and corresponding ease of erection of a particular section of the structure. It is generally uneconomic to transport very large sections of steelwork, which, because of their size, will require a police escort in the UK, and may also demand the one-off hire of special lifting equipment in order to erect the part. In general, most 'shop' joints effected in the fabricator's workshop are welded. In the workshop, it is easier to obtain the correct 'fit-up' of the various parts for welding and also to make allowances for and, if necessary, remedy any distortions resulting from welding. In bridgework, in situ joints can be welded, bolted or made using a combination of welding and bolting. However, most in situ connections are bolted, with extensive use being made of high-strength friction grip (HSFG) bolts.

Advantages and disadvantages of bolted and welded in situ connections

Owing to restrictions imposed by the need to transport to site individual sections of the structure shorter than approximately 25 m, in situ connections are nearly always required to construct the complete bridge. To assist the bridge designer in making the correct choice of connection, Table 10.6 summarises the advantages and disadvantages of both bolted and welded in situ connections.

Bolted joints

Welded joints

Advantages

- Generally cheaper.
- Connections are almost self-aligning and relatively quick to make.
- Only semi-skilled labour required.
- Allows adjustment to vertical and longitudinal alignment if normal clearance holes are used.
- Easy to inspect.
- Normally does not govern bridge fatigue life.
- Not weather-sensitive during 'bolting up'.
- Good aesthetics forming a visually unobtrusive connection.
- Minimises girder weight with no deductions required for holes in the tension flange.
- Full-strength connection can be achieved by using full-penetration butt welds.

Disadvantages

- May be visually unacceptable.
- Deduction for holes in tension flange may lead to the use of a larger section.
- Can be prone to corrosion if not properly protected.
- Expensive unless in large numbers.
- Weather-sensitive, must be protected from moisture.
- Pre-heat of steelwork is usually necessary requiring a tent/shelter to prevent heat loss and protection against inclement weather.
- Expensive to inspect for defects.
- If defects are present, it is expensive and time-consuming to repair, with subsequent risk of delay to project.
- May govern the fatigue life of the structure.
- Skilled operatives required, working to an approved specification.
- Temporary cleats needed to assist in pre-weld alignment. Cleats normally have to be carefully removed and parent material restored to original condition to prevent fatigue problems.

Table 10.6 Advantages and disadvantages of bolted and welded in situ connections

Manual metal arc process (MMA)

This process is the oldest of all of the arc processes and is widely used by fabricators for attaching stiffeners, and such like. The process is a manual operation, with the quality of the finished weld depending to a large extent on the skill of the welder. The electrode consists of a steel wire coated with a flux formed from cellulose, silicates, iron oxide, and so on. The primary purpose of the flux is to shield the weld pool to prevent the molten metal oxidising and, in addition, taking into solution nitrogen and other unwanted gases. Some of these gases, if present, are released back into the air as the weld cools, in the process causing porosity in the weld metal. The flux on the electrode also helps to stabilise the plasma arc and can be used to control the deposition rate of the molten metal. It is most important that the electrodes chosen are of the correct grade of steel, at least equivalent to that of the parent parts to be joined, and that they are completely dry and corrosion-free before use. Eurocode 3 (BSI, 1992) specifies that the filler metal in the electrodes must have mechanical properties, namely yield strength, ultimate tensile strength, elongation at failure and a minimum Charpy V-notch energy value, equal to or better than the values specified for the steel grade being welded. The electrodes used in the manual metal arc process are relatively short; hence, for long continuous welds there will be numerous stop-start positions, each the potential source of an imperfection and possible fatigue damage.

Submerged arc welding (SAW)

Unlike the manual metal arc process, the submerged arc method is a fully automatic method, employing a travelling gantry or robotic unit. The electrode is a continuous bare steel wire, which is unwound from a storage drum as required, and fed, via the welding nozzle, into a bed of flux, which has been deposited on the surfaces to be welded. The flux, which completely encapsulates the plasma arc, can also be used to enhance the composition of the weld metal. The submerged arc process can only be successfully used in the 'down hand' position and is generally the preferred method for making long welds, such as the web to flange welds required for plate girder fabrication.

Gas-shielded processes

These processes, like the submerged arc process, use a continuous bare wire electrode. However, the weld pool is protected from the atmosphere not by a flux but by a gas, which is also fed via the welding nozzle, around the plasma arc and weld pool. The shielding gas used may be inert argon or non-reactive carbon dioxide and the processes are termed metal inert gas (MIG) and metal active gas (MAG), respectively. These processes are very versatile and can be used in both the 'down hand' and 'overhead' positions but must be sheltered from draughts, which tend to disturb the gas flow.

Welded connections

The welding process

The process of welding requires the formation of a molten metal weld pool, which is used to fuse together the faces of the parts to be joined. Arc welding is the predominant method used for welding structural steelwork; in this process, a weld pool is formed by passing a high current through an electrode, which when placed close to the earthed work piece allows a plasma arc to form. The high current passing through the electrode and subsequently through the arc into the earthed work piece causes the tip of the electrode to melt and allows the transfer of molten metal from the electrode to the work piece to form a weld bead. Several arc welding techniques are available for use by the fabricator and the major methods are briefly outlined here.

Welded connection design

Two types of structural weld are in common use, namely, butt welds and fillet welds. A butt weld is normally made within the cross-section of the abutting plates, whereas a fillet weld is a weld of approximately triangular cross-section applied to the surface of the plates to be joined. When plates greater than about 5 mm thick are to be butt welded together, the plate edges will have to be prepared before welding in order to obtain a full-penetration weld. Figure 10.67 shows typical butt welds, together with the required edge preparation. A full-strength penetration butt weld in structural steel, made correctly with the appropriate electrodes, is considered to be as strong as the parent steel plates; consequently, no strength calculations are necessary, unless fatigue is a consideration.

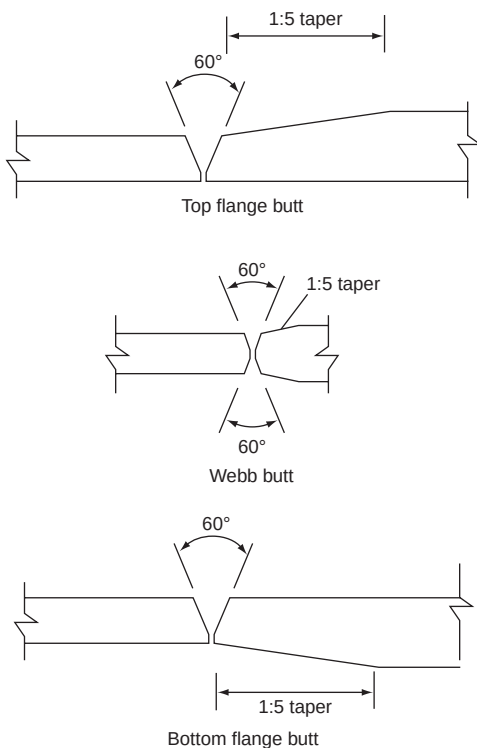


Figure 10.67 Typical butt welds showing the required edge preparation (Needham, 1983)

Fillet welds, however, do not require any edge preparation for the parent plates and consequently are usually cheaper to undertake than butt welds. Fillet welds are usually specified by requiring a minimum leg length for the weld. These welds are considered to carry all of the loads applied to them through shear, which acts on the effective throat of the fillet weld. Figure 10.68, which is similar to Figure 4.3 in EN 1993-1-8 Clause 4.5.2 (BSI, 2005a), shows the method for calculating the effective throat area, which is described as the height of a triangle that can be inscribed within the weld cross-section and measured perpendicular to its outer side.

Eurocode 3 Part 1-8 (BSI, 2005a) suggests two alternative

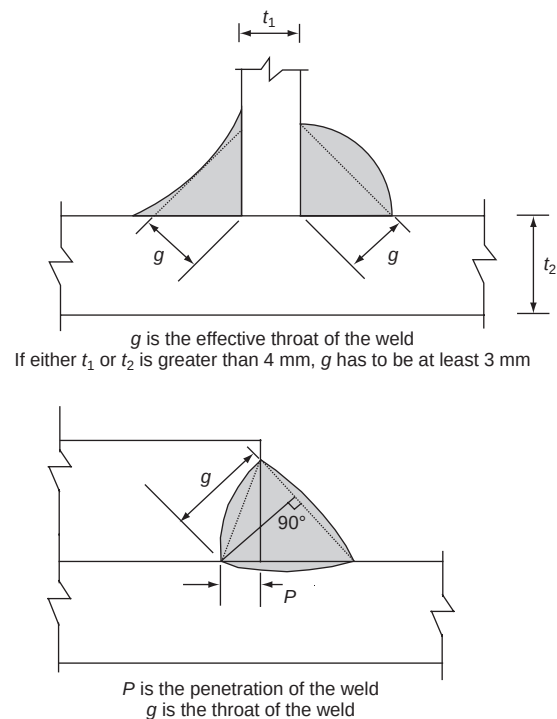


Figure 10.68 Effective throat thickness of fillet welds (after EN 1993-1-8:2005)

procedures for assessing the design resistance of a fillet weld, namely the directional method and the simplified method. In the directional method, the forces transmitted by a unit length of weld are resolved into components transverse and parallel to the longitudinal axis of the weld and transverse and normal to the plane of the weld throat. Figure 10.69, similar to Figure 4.5 in Eurocode 3 Part 1-8, shows the stresses acting on the throat section of a fillet weld, where the following notation is used:

$\sigma_{\perp}$ is the normal stress perpendicular to the throat.

$\sigma_{\parallel}$ is the normal stress parallel to the axis of the weld.

$\tau_{\perp}$ is the shear stress in the plane of the throat perpendicular to the axis of the weld.

$\tau_{\parallel}$ is the shear stress in the plane of the throat parallel to the axis of the weld.

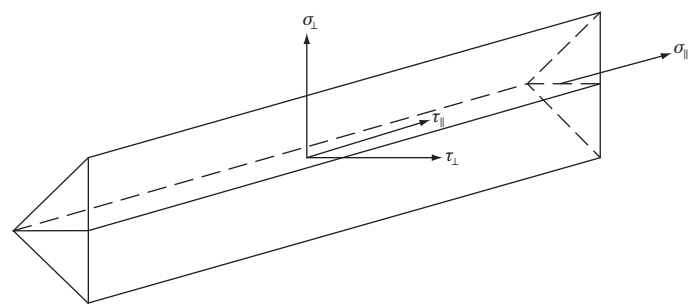


Figure 10.69 Stresses acting on the throat section of a fillet weld

The normal stress $\sigma_{\parallel}$ parallel to the axis is not considered when checking the design resistance of the weld. The design resistance of the fillet weld will be considered to be sufficient if the following expressions are both satisfied (EN 1993-1-8:2005 Clause 4.5.3.2 (6)):

$$\left[\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)\right]^{0.5} \leq f_u / (\beta_w \gamma_{M2}) \quad \text{and} \quad \sigma_{\perp}^2 \leq 0.9 f_u / \gamma_{M2} \quad (10.74)$$

where:

f_u is the nominal ultimate tensile strength of the weaker part joined.

β_w is the appropriate correlation factor taken from Table 10.7.

Standard and steel grade			Correlation factor β_w
EN 10025	EN 10210	EN 10219	
S 235 S 235 W	S 235 H	S 235 H	0.8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0.85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0.9
S 420 N/NL S 420 M/ML	—	S 420 MH/MLH	1.0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1.0

Table 10.7 Correlation factor β_w for fillet welds (Table 4.1 of EN 1993-1-8:2005)

In the simplified method, the fillet weld will be satisfactory if, at every point along its length, the resultant of all the forces per unit length transmitted by the weld satisfies the following criterion:

$$F_{w,Ed} \leq F_{w,Rd} \quad (10.75)$$

where:

$F_{w,Ed}$ is the design value of the weld force per unit length.

$F_{w,Rd}$ is the design weld resistance per unit length.

Independent of the orientation of the weld throat plane to the applied force, the design resistance per unit length, $F_{w,Rd}$, should be determined from

$$F_{w,Rd} = f_{vw,d} a \quad (10.76)$$

where $f_{vw,d}$ is the design shear strength of the weld, given by

$$f_{vw,d} = \frac{f_u \sqrt{3}}{\beta_w \gamma_{M2}} \quad (10.77)$$

Full-penetration butt welds should be taken as equal to the design resistance of the weaker of the parts being joined, provided the weld is made with consumables that will produce all-weld tensile specimens having both a minimum yield strength and a minimum tensile strength not less than those specified for the parts being joined.

Bolted connections

It is generally a requirement that, for bridgework, all bolted structural connections are made using preloaded bolts, generally known as high-strength friction grip (HSFG) bolts. These bolts, which are tightened to achieve a specified shank tension, act by clamping together the plates to be joined, so that, under normal loading conditions, the applied forces are transferred through the connection by friction acting at the plate interfaces. High-strength structural bolts of bolt grade 8.8 and 10.9, conforming to the standards given in EN 1993-1-1, 2.8: Group 4 (BSI, 1992) may be used as preloaded bolts when controlled tightening is carried out in accordance with the reference standards given in EN 1993-1-8, 2.8: Group 7. Table 10.8 gives the nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts (Table 3.1 of EN 1993-1-8:2005).

Bolt class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_{yb} : N/mm ²	240	320	300	400	480	640	900
f_{ub} : N/mm ²	400	400	500	500	600	800	1000

Table 10.8 Nominal values of the yield strength f_{yb} and the ultimate tensile strength f_{ub} for bolts (Table 3.1 of EN 1993-1-8:2005).

Bolt tension

The reliable control of bolt tension is essential for ensuring predictable performance in preloaded bolted joints. The Code of Practice EN 1090-2:2008 (BSI, 2008a) outlines the procedures acceptable for tightening these bolts. The code gives four procedures for tightening preloaded bolts namely, the torque method, the combined method, the high-resistance calibrated (HRC) method and the direct tension indicator method. The torque method requires the use of a manually operated torque wrench or power-driven wrench to achieve the required bolt tension. The procedure is undertaken in two stages, typically with an initial torque followed by a final torque using an accurately calibrated torque wrench. With the combined method, the initial tightening is undertaken using a calibrated torque wrench, which is then followed by a part turn of the nut relative to the bolt shank. After the first step is complete, the position of the nut is marked and then the nut is rotated a certain amount relative to the bolt thread, as specified in EN 1090-2:2008. For the HRC method, bolt tightening is achieved using a special wrench, which uses two coaxial sockets that react one against the other. The nut is held by the outer socket, which rotates clockwise, while the inner socket locates onto the spline top part of the bolt. These tension control bolts have been designed so that the top spline of the bolt shears off at the required torque. Figure 10.70 shows a typical tension control bolt. There are other methods of obtaining the required bolt tension, the most frequently used method employing direct tension indicators (DTIs), previously known as load indicator washers. The direct tension indicator method relies on the plastic compression of nibs, which are incorporated in the surface of

the washer, down to a predetermined gap, giving a reliable and relatively easily inspected bolt assembly. Figure 10.71 also shows a typical splice connection in a plate girder, made using tension control bolts.



Figure 10.70 Tension control bolt

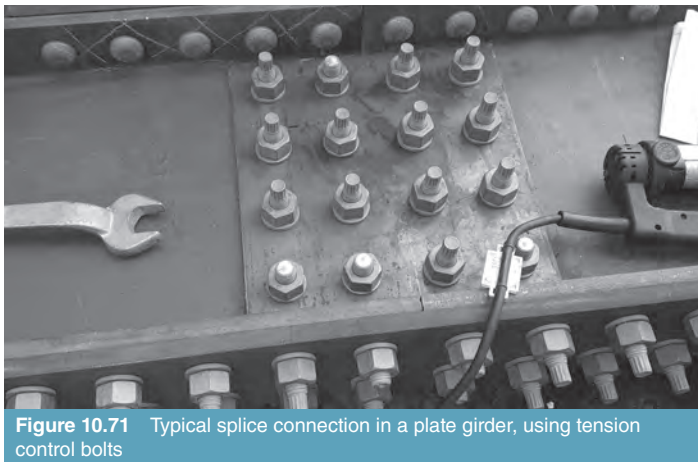


Figure 10.71 Typical splice connection in a plate girder, using tension control bolts

Bolted connection design

The Eurocode EN 1993-1-8:2005 Clause 3.4.1 (BSI, 2005a) classifies bolted connections loaded in shear into three categories.

Category A: Bearing type

This applies for bolts from Class 4.6 up to and including Class 10.9. No preloading is required and the design ultimate shear load should not exceed the design shear resistance obtained from Clause 3.6 of the Eurocode. Also, the ultimate shear load should not exceed the design bearing resistance given in Clauses 3.6 and 3.7 of the Eurocode. Owing to the tendency of these connection types to shake loose under vibration, as well as their poor fatigue resistance, it is unwise to use Category A connections in bridges.

Category B: Slip resistant at serviceability limit state

This category applies to preloaded bolts in compliance with Clause 3.1.2(1). Slip of the connection should not occur at the serviceability limit state but may occur at the ultimate limit state. The serviceability shear force should not exceed the design slip resistance obtained from Clause 3.9 and the ultimate shear force should not exceed the design shear resistance

obtained from Clause 3.6. Also, the ultimate shear force should not exceed the design bearing resistance obtained from Clauses 3.6 and 3.7 of EN 1993-1-8:2005.

Category C: Slip resistant at ultimate limit state

As for Category B connections, this category applies to preloaded bolts in compliance with Clause 3.1.2(1). Slip should not occur at either the serviceability or the ultimate limit state. Consequently, the ultimate shear force should not exceed the design slip resistance obtained from Clause 3.9 or the design bearing resistance obtained from Clauses 3.6 and 3.7 of EN 1993-1-8:2005.

Design resistance of individual bolts

Table 3.4 of EN 1993-1-8:2005 (BSI, 2005a) gives a summary of the design resistance for individual fasteners subjected to shear or tension. Only the shear resistance of individual bolts will be considered here, where the shear resistance per shear plane is given by

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} \quad (10.78)$$

where the shear plane passes through the threaded portion of the bolt and A is the tensile stress area of the bolt. For bolt classes 4.6, 5.6 and 8.8, $\alpha_v = 0.6$. For bolt classes 4.8, 5.8, 6.8 and 10.9, $\alpha_v = 0.5$. Where the shear plane passes through the unthreaded portion of the bolt, A is the gross cross-sectional area of the bolt and $\alpha_v = 0.6$.

The bearing resistance is given by (Table 3.4 of EN 1993-1-8: 2005):

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} \quad (10.79)$$

where α_b is the smallest of α_d , f_{ub}/f_u or 1.0.

In the direction of load transfer:

$$\text{For end bolts: } \alpha_d = \frac{e_1}{3d_0}$$

$$\text{For inner bolts: } \alpha_d = \frac{p_1}{3d_0} - \frac{1}{4} \quad (10.80)$$

Perpendicular to the direction of load transfer:

$$\text{For edge bolts: } k_1 \text{ is the smallest of } 2.8 \frac{e_2}{d_0} - 1.7 \text{ or } 2.5$$

$$\text{For inner bolts: } k_1 \text{ is the smallest of } 1.4 \frac{p_2}{d_0} - 1.7 \text{ or } 2.5 \quad (10.81)$$

The punching shear resistance is given by (Table 3.4 of EN 1993-1-8:2005)

$$B_{p,Rd} = 0.6 \pi d_m t_p f_u / \gamma_{M2} \quad (10.82)$$

Slip resistance connections using Class 8.8 or 10.9 bolts

The design slip resistance of a preloaded Class 8.8 or 10.9 bolt should be taken as (Clause 3.9.1 of EN 1993-1-8:2005)

$$F_{s,Rd} = \frac{k_s n \mu}{\gamma_{M3}} F_{p,C} \quad (10.83)$$

where k_s is given in Table 10.9 (Table 3.6 of EN 1993-1-8:2005), n is the number of the friction surfaces and μ is the slip factor given in Table 10.10 (Table 3.7 of EN 1993-1-8 : 2005).

Description	k_s
Bolts in normal holes	1.0
Bolts in either oversize holes or short slotted holes with the axis of the slot perpendicular to the direction of load transfer	0.85
Bolts in long slotted holes with the axis of the slot perpendicular to the direction of load transfer	0.7
Bolts in short slotted holes with the axis of the slot parallel to the direction of load transfer	0.76
Bolts in long slotted holes with the axis of the slot parallel to the direction of load transfer	0.63

Table 10.9 Values of k_s (Table 3.6 of EN 1993-1-8 : 2005)

Class of friction surfaces (EN 1090)	Slip factor, μ
Class A: Surfaces blasted with shot or grit, with and loose rust removed, no pitting	0.5
Surfaces blasted with shot or grit and spray-metallised with aluminium	
Surfaces blasted with shot or grit and spray-metallised with a zinc-based coating certified to provide a slip factor of not less than 0.5	
Surfaces blasted with shot or grit, painted with an alkali-zinc silicate paint to produce a coating thickness of 50–80 μm	0.4
Surfaces cleaned by wire brushing of flame cleaning, with any loose rust removed	0.3
Surfaces not treated	0.2

Table 10.10 Slip factor, μ , for preloaded bolts (Table 3.7 of EN 1993-1-8 : 2005)

For Class 8.8 and 10.9 bolts with controlled tightening (Clause 3.9.1(2) of EN 1993-1-8:2005), F_{pC} should be taken as

$$F_{pC} = 0.7f_{ub}A_s \quad (10.84)$$

Other considerations

Lack of fit

In an HSFG bolted connection, good ‘fit-up’ of the parent and cover plates is essential if a load-bearing connection is to be achieved. If this is not the case, and the joint faces are not flat or are not properly aligned, some or possibly all of the prestress in the bolts will be used to bring the plates into contact, with little or no bolt tension available to induce friction resistance in the faying surfaces. CIRIA (1981) has published a report on the lack of fit in steel structures, which offers valuable advice on how to circumvent such problems.

Relaxation of bolt tensions

Research has shown that the resistance to slip along the faying surfaces is a result of shearing of the contact interfaces of microscopic protrusions on the surface of the plates. This resistance can be enhanced with surface contamination and roughness. In an HSFG bolted joint, these surface effects, together with a small reduction in joint plate thickness caused by in-plane plate tensile stresses owing to the applied load, can cause a loss of bolt pre-tension. The loss of bolt tension owing to plate thinning has been shown to increase rapidly when the in-plane tensile stress becomes large enough, which when combined with the normal bolt pressure, causes local yielding of the plate material (Cullimore and Eckhart, 1974). The loss of bolt tension from this cause will therefore increase with the ratio of bolt prestress to the yield stress of the parent plates; with other parameters remaining constant, the slip factor will decrease as the bolt tension increases.

Slip factor

The slip factor μ that can be used when assessing the friction capacity of a preloaded (HSFG) bolted connection is given in Table 10.10 (Table 3.7 of EN 1993-1-8:2005). Generally, for UK bridge construction, the faying surfaces are grit-blasted and then masked until the erection of the steelwork, allowing $\mu=0.5$. As mentioned previously, the slip load of a connection will be improved by increasing the friction coefficient of the faying surfaces. If this is achieved by increasing the roughness of the faying surfaces, it will generally result in offsetting the accompanying loss of bolt tension. Where difficulties arise in assessing a suitable value of μ , the characteristic value of the slip factor should be determined in accordance with the procedure given in EN 1090-2:2018:1.2.7. Reference Standards: Group 7 (BSI, 2018).

Fatigue

Introduction

Fatigue is the mechanism caused by cyclic loading that permits crack growth in a member, finally resulting in failure of the element. The process is strongly influenced by the magnitude of the applied stress range to which the element is subject and also by the presence and acuity of stress concentrations occurring within the element.

Fatigue fracture surface

A fatigue fracture surface normally exhibits two distinct regions, typically a smooth flat area and a rougher area, which forms the remaining part of the fracture cross-section as shown in Figure 10.72.

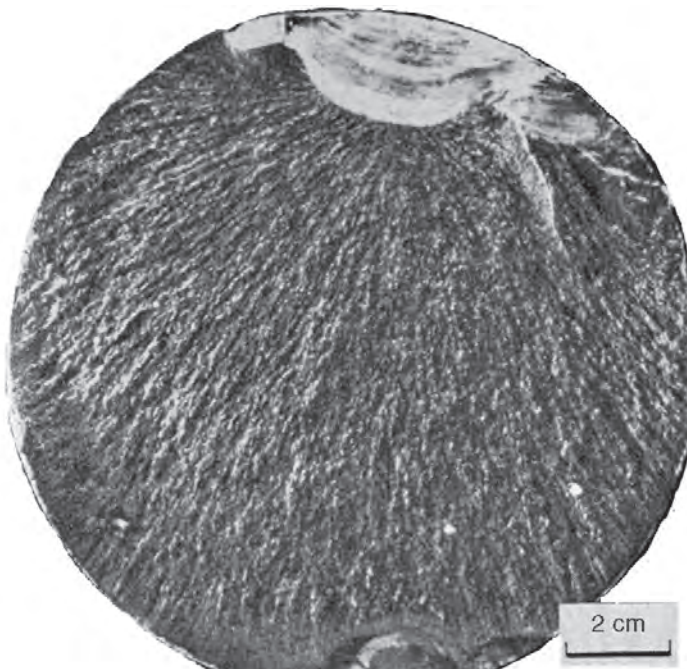


Figure 10.72 Typical fatigue failure in a steel component (ESDEP, 1998)

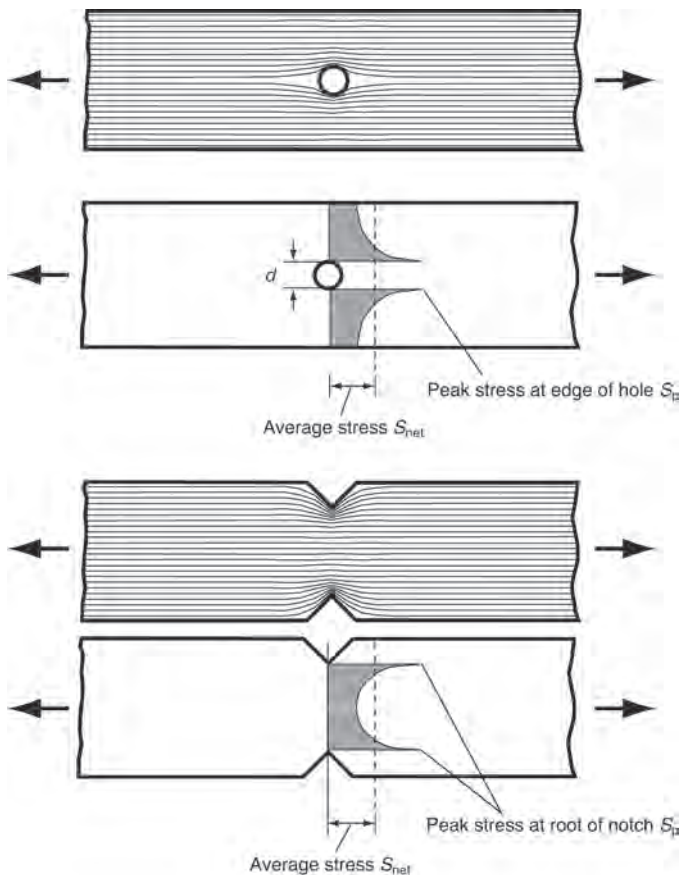


Figure 10.73 Stress concentrations (Gurney, 1979)

The smooth region frequently exhibits concentric rings or 'beach marks', which surround the fracture point, and radial lines, which tend to point towards the fracture nucleus. The smooth area is the region of the fracture surface over which the fatigue crack grew, initially in a slow stable manner. The rougher region is the final rupture area, through which the fatigue crack progressed rapidly in an unstable manner.

The relative size of the two regions gives a good indication of the stress level causing failure. If the rough area is large, compared with the smooth area, the stress level was high, indicating that only a small part of the overall cross-section, represented by the smooth region, could be lost through crack propagation. If the smooth area is large, relative to the rough area, then a large percentage of the cross-section could be lost through propagation of the fatigue crack before failure occurred, indicating a low stress level initially acting on the cross-section.

Stress concentrations

In welded steel bridges, the fatigue performance of the entire structure is usually governed by the fatigue strength of the individual welds. Fatigue cracks will initiate and grow from both load-bearing and non-load-bearing welds. This is because the welding process causes metallurgical discontinuities, together with physical changes in shape, both of which cause local stress concentrations. Stress concentrations are also produced by notches, holes and any abrupt change of shape in a member that interrupts a smooth stress path. The stress concentration factor, which is used to define the magnitude of the stress concentration, is defined by the relationship $K_t = S_p / S_{net}$ where S_p is the peak stress and S_{net} the average stress on the net cross-section, as shown in Figure 10.73.

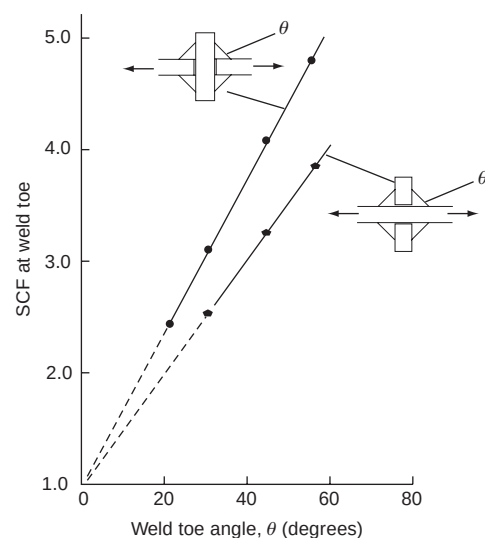


Figure 10.74 Influence of weld toe angle on the stress concentration at the weld toe, obtained using finite-element analysis (Gurney, 1976)

Where stress concentrations arise from a sudden local change in shape, it is not necessary for the change in shape to involve a reduction in the cross-sectional area of the member; an abrupt increase in the cross-sectional area can also induce stress concentrations.

For welded joints, where the weld is transverse to the direction of stress, the weld toe angle has a major influence on the stress concentration factor (SCF), with the SCF increasing as the toe angle increases (Figure 10.74).

The weld size also has an influence on the SCF; in non-load-carrying joints, the SCF tends to increase with increasing weld size, although for load-carrying welds, also transverse to the stress direction, there is a dramatic decrease in SCF with increasing weld size (Figure 10.75).

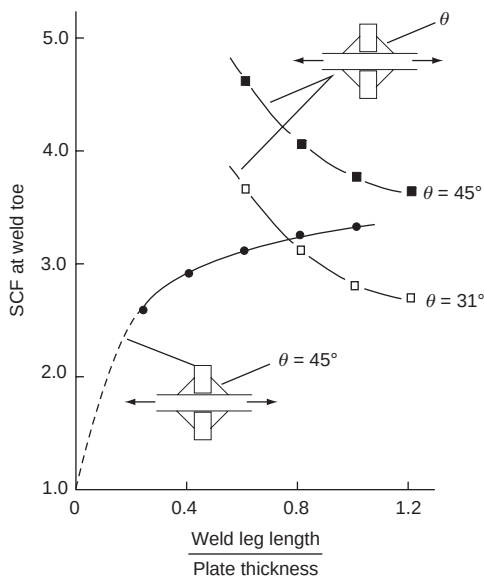


Figure 10.75 Influence of weld size on the concentration at the toe of transverse fillet welds (Gurney, 1976)

It is important to appreciate that fatigue cracks and failure can occur in members that are nominally in compression if they are welded to the structure or contain a welded detail. During the welding process, shrinkage of the weld as it cools induces large tensile residual stresses in the weld regions. The large tensile residual stresses occurring in the weld vicinity must be added to the fluctuating compressive stress acting in the member, resulting in a tensile stress variation in the welded regions and causing crack propagation.

Stress ranges and mean stress

As mentioned previously, poor fatigue performance may be associated with welded joints. The stress range σ_r , developed from cyclic loading, can cause crack propagation in welded connections. The stress range is defined as the difference between the maximum stress $\sigma_{\max}$ and the minimum stress $\sigma_{\min}$, while the mean stress is defined as $\sigma_m = (\sigma_{\max} + \sigma_{\min})/2$. Welded

regions are always susceptible to fatigue, owing to the presence of residual tensile stresses. This can affect the behaviour and rate of propagation of the fatigue crack, owing to the summation of the applied stress and the residual tensile stress. The characteristics of the crack can behave in different ways based on the stress–time variation occurring within the bridge. For example, if the alternating stress cycles equally in tension and compression, the mean stress will be zero, as $\sigma_{\min} = -\sigma_{\max}$.

However, if the stress alternates only in the tension zone, the mean stress can be expressed as $\sigma_{\max}/2$. Thus, the determination of the fatigue loading must be carefully interpreted, in order to choose the correct stress condition acting on the structural element. Generally, a fluctuating stress under constant amplitude (CA) can be represented by a sine wave, as shown in Figure 10.76.

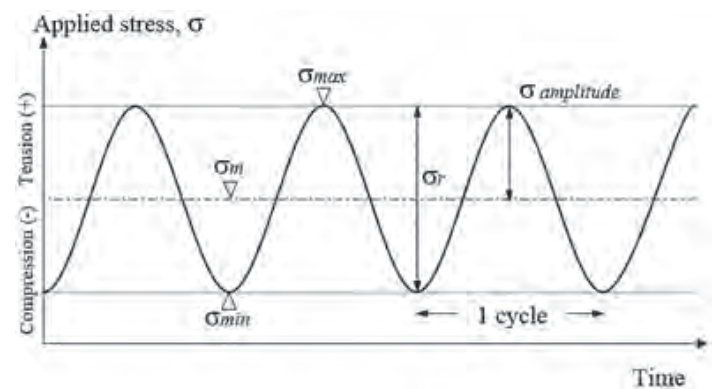


Figure 10.76 Typical constant amplitude fluctuating stress

S–N curve

Generally, the fatigue strength of a bridge connection is based on its local shape, size and metallurgical discontinuities. Therefore, based on actual fatigue tests of specimens under a constant mean stress, σ_m and constant amplitude cyclic (CA) loading for different stress ranges, σ_r , the S–N curve was introduced to permit assessment of the fatigue strength corresponding to typical detail categories. However, a large number of specimens must be tested to produce a S–N curve for each structural detail. This is to ensure that the resulting S–N relationship is sufficient to represent the fatigue strength of the chosen structural detail. In addition, the condition of the structural details should include the residual stress field to ensure the accuracy of the test results (Nussbaumer et al., 2011). This is because the weld quality, for example, can affect the fatigue strength of the structural detail.

The S–N curves are normally plotted on a double-logarithmic scale, as shown in Figure 10.77. This logarithm scale results in a relationship known as Basquin's law, which can be written as

$$\log(N) = \log(C) - m \log(\Delta\sigma) \quad (10.85)$$

where m is the slope of the mean test results line and C is a constant representing the influence of the structural detail.

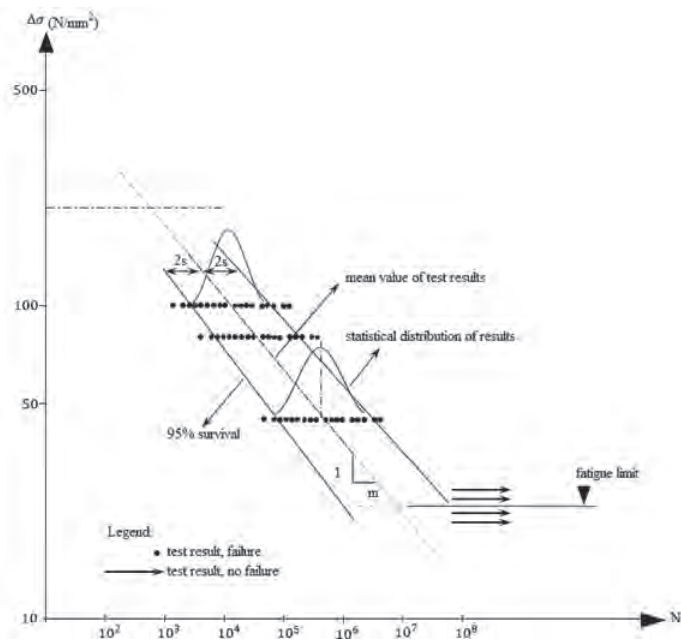


Figure 10.77 Typical set of fatigue test results under constant amplitude loading (Nussbaumer et al., 2011)

Equation 10.85 can be simplified as

$$N(\Delta\sigma)^m = C \quad (10.86)$$

The application of this formula can be seen in Figure 10.77, where the test results for a particular structural detail are given in the range between 10^4 and 10^7 cycles. The S–N relationship reaches a lower limit plateau, which represents a constant value, signifying that at, and below, this limit, the fatigue life is infinite, indicating there is no fatigue failure.

It should be noted that the fatigue limit can only be derived if the test specimens have undergone constant amplitude loading. Moreover, the cycles will be limited up to 2×10^6 cycles as a

consequence of the dominance of fatigue crack propagation (Maddox, 1991). In addition, the survival probability must be set. Most of the codes of practice, including the Eurocodes, chose a 95% survival probability taken from the mean value calculated for a 75% confidence to determine the ‘best-fit mean’ curve. Figures 10.78 and 10.79 show the fatigue strength curves for direct and shear stress range, respectively, for the welded structural details set in BS EN 1993-1-9 (BSI, 2005b).

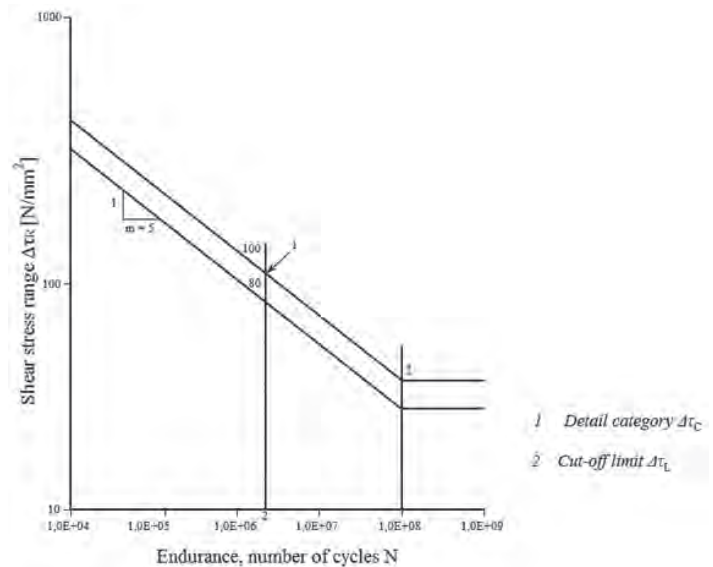


Figure 10.79 Fatigue strength curves for shear stress ranges (BS EN 1993-1-9)

Variable amplitude (VA) loading

As mentioned previously, the S–N curves for each of the structural details were determined under constant amplitude (CA) loading. However, it should be noted that the actual loading on a bridge structure, such as moving heavy vehicles crossing the bridge, induces different stress ranges, producing variable amplitude (VA) loading. Figure 10.80 shows a typical stress history for a particular point on a bridge, produced for a sequence of loads traversing the structure.

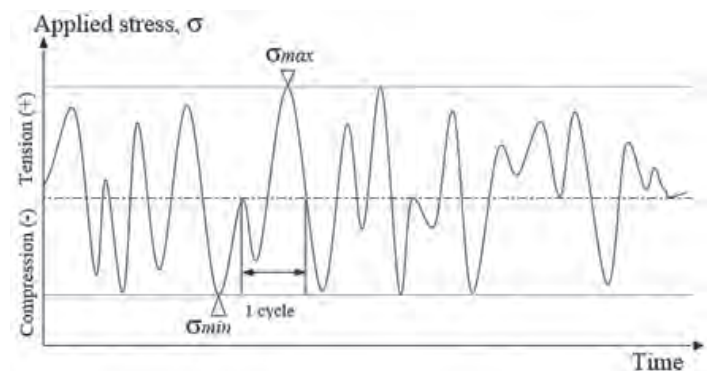


Figure 10.80 Variable amplitude (VA) loading

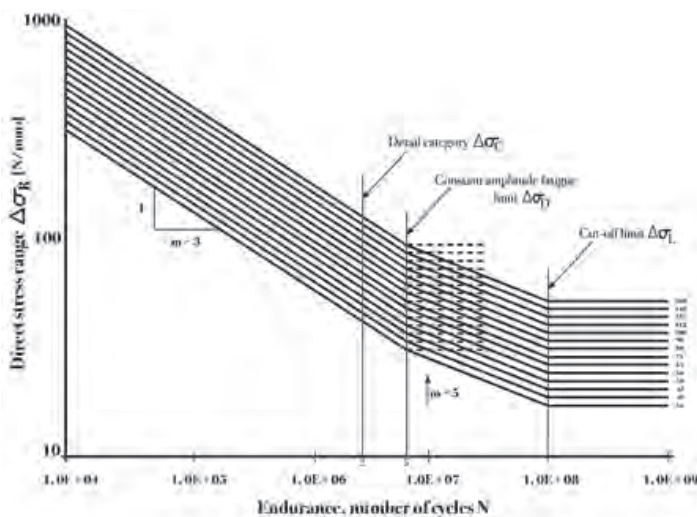


Figure 10.78 Set of fatigue strength curves for direct normal stress ranges (BS EN 1993-1-9)

Therefore, understanding how the S–N curves can be used appropriately in design for a bridge under VA loading is very important. Practically, the endurance limit obtained from constant amplitude loading should not be considered when designing a bridge structure subjected to variable amplitude loading. Langer (1937) was the earliest researcher to declare that the maximum stress arising from variable amplitude loading could exceed the endurance limit of the material recommended by using the S–N diagram obtained from constant amplitude loading.

This is because so many questions arise over how fluctuating stress cycles, shown in Figure 10.80, can be counted and how these different stress magnitudes can affect the fatigue life. To overcome these difficulties, a linear cumulative fatigue damage estimate can be achieved using the well-known Palmgren–Miner rule, expressed as

$$D = \sum \frac{n_i}{N_i} \quad (10.87)$$

where D is the total damage, while n_i is the applied number of cycles and N_i is the corresponding number of repetitions to failure. This rule stipulates that when $D=1.0$, fatigue damage will occur. However, the value of the total damage D is recommended to be limited to 0.5 by IIW (2008) and BS 7608 (BSI, 2014) as an additional safety factor for structural details subjected to multiaxial stresses and imperfections.

In addition, the use of the Palmgren–Miner rule to estimate the damage accumulation should be appropriately associated with the methods used to analyse the stress history. This is because when different stress ranges occur, specific approaches are used to estimate the damage, such as the peak counting method, the level crossing counting method, the rainflow counting method and the reservoir counting method. The rainflow counting method and the reservoir counting method are the preferred approaches, as both of these methods give a good definition of the stress ranges acting on the structure and take into consideration the damaged caused by even small load variations (Nussbaumer *et al.*, 2011).

Damage accumulation

As mentioned previously, owing to the difficulty in obtaining the damaging individual load cycles occurring under variable loads, Miner's rule has been introduced. This rule describes the fatigue damage that will occur at any particular stress range, which is directly proportional to the total number of cycles to failure, also at that particular stress range. The fatigue damage will be calculated by counting the number of applied cycles, n_i , accumulated at stress σ_i and comparing this number with the total number of cycles to failure, N_i , also at stress σ_i , obtained from constant amplitude tests. Therefore, with variable amplitude loading, partial damage owing to individual cycle counting can be summarised as

$$D_d = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \dots = \sum_{i=1}^n \frac{n_i}{N_i} \leq 1.0 \quad (10.88)$$

The failure defined by the summation of all of the partial damages can also be represented graphically, as shown in Figure 10.81, to show the schematic of partial damage corresponding to the variable amplitude loading shown on the S–N curve.

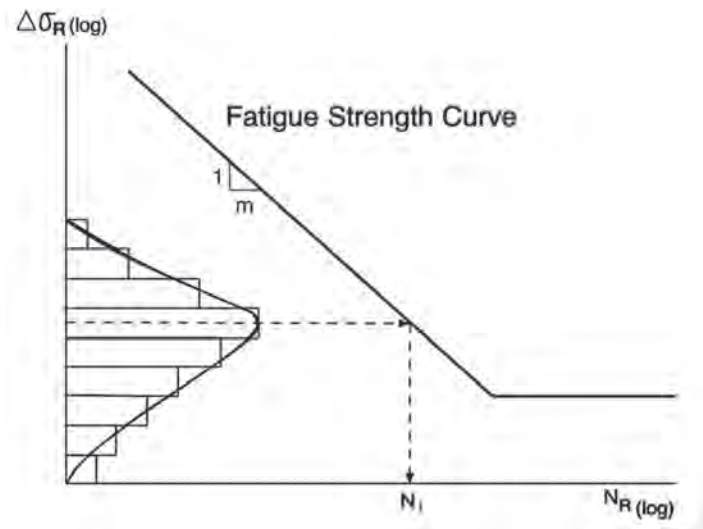


Figure 10.81 Damage accumulation scheme shown on an S–N curve (Nussbaumer *et al.*, 2011)

It should be noted that, with a bridge structure subjected to variable amplitude loading, the sequence of the occurring damage will be ignored, as variable loading can cause random cycles and different values of the stress ranges.

Fatigue assessment to Eurocode

Fatigue assessment of a bridge can be estimated based on its serviceability, which decreases with time, owing to the repetitive loadings that can cause damage to the structure. Two methods for the assessment of fatigue are covered in BS EN 1993-1-9 (BSI, 2005b) in order to adopt the fatigue reliability of members, connections and joints subjected to fatigue loading. These two methods are the damage-tolerant method and the safe-life method. The main difference between these two assessments is the requirement of a fatigue maintenance regime. An explanation of these two recommended methods outlined in the Eurocode is given next.

The damage-tolerant method

This approach ensures acceptable reliability in that a structure will perform satisfactorily for its design life, provided that a prescribed inspection and maintenance regime for detecting and correcting fatigue damage is implemented throughout the design life of the structure. This method is generally used, owing to the insufficient knowledge obtained of the fatigue performance of the structure to sustain its service life. Thus,

by calculating the probability that a fatigue crack will develop during its service life, the remaining structure can still be allowed to sustain its working load until the actual damage is detected. Thereby, this selected method should be accompanied with a high level of monitoring and inspection by the owner and the willingness to undertake frequent repairs and member replacement (Nussbaumer *et al.*, 2011).

Safe-life method

This practice provides an acceptable level of reliability that a structure will perform satisfactorily throughout its design life without the need for regular in-service inspection for fatigue damage. Therefore, no specification is required for a maintenance routine for fatigue that will be needed for this design. This practice may be preferred by the bridge owner for two reasons: (a) poor accessibility to the fatigue-critical details; (b) high-maintenance work and repair costs that the owner might not be willing to commit to (Nussbaumer *et al.*, 2011).

Comparing these two assessment methods, the damage-tolerant method should result in an initial lower cost for the bridge structure but this is accompanied with a lower minimum design service life because the structure will need frequent inspection that will also have an additional cost. The safe-life method will result in a higher initial cost for the structure but, in return, it should not demand regular frequent inspections for fatigue damage. Figure 10.82 shows, schematically, a comparison of the reliability index for the safe-life and damage-tolerant methods in the design service life of a structure.

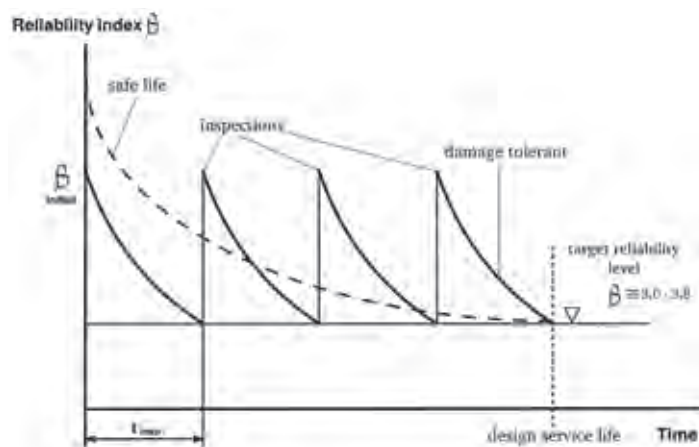


Figure 10.82 Schematic of the fatigue reliability index assuming the damage-tolerant and safe-life appraisal methods (Nussbaumer *et al.*, 2011)

Therefore, to achieve the desired outcome, the safe-life method has higher partial factors for fatigue strength, γ_{Mf} than that used for the damage-tolerant method. Table 10.11 gives the recommended values for the partial factors for fatigue strength, which also depend on the consequence of failure.

Assessment method	Consequence of failure	
	Low consequence	High consequence
Damage-tolerant	1.00	1.15
Safe-life	1.15	1.35

Table 10.11 Recommended values for partial factors for fatigue strength (BS EN 1993-1-9:2005)

Fatigue assessment loading

Five traffic fatigue load models (FLMs) specified in the Eurocode, namely, FLM1 to FLM5, can be applied to design a roadway bridge. Each FLM has its own specification, in order to ensure that the fatigue verification can be designed for a 120-year service life. However, the most suitable FLM for fatigue analysis should be carefully selected to ensure that the fatigue failure will not occur during the bridge service life.

All five FLMs consist of a set of equivalent lorries used to simulate vehicle or traffic loads crossing the bridge. Both FLM1 and FLM2 are used to justify that the fatigue life of the bridge will be infinite. The justification of using these models is that there will be no fatigue crack propagation occurring in any of the structural details. This is based on the assumption that the vehicle or traffic loading on the bridge will generate cycles and amplitude stress values smaller than the constant amplitude fatigue limit (CAFL) by reference to the S–N curve. However, it is difficult to verify these conditions on bridges nowadays because of the increases in traffic loads. In contrast, FLM3 to FLM5 are used to demonstrate that the bridge has a precise fatigue life, obtained by appraisal of the bridge with the traffic flow rates given in these fatigue load models. However, compared with FLM4 and FLM5, FLM3 represents the traffic flow rates in a simplified way.

FLM1

This load model is derived from the primary characteristic load model for road bridges, namely load model 1, (LM1) as outlined in BS EN 1991-2 (BSI, 2003). In terms of the load arrangement, LM1 consists of a uniformly distributed load (UDL), q_{ik} , together with a tandem system (TS) Q_{ik} . The application of FLM1 is very similar to the application of LM1, as the only difference is the superposition used for the UDL and the TS. For example, the value adjustment factors are 0.7 and 0.3, respectively, for UDL and TS in FLM1 for traffic lane i . In contrast, LM1 uses 1.0 as superposition loading for the TS, with 0.61 or 2.2 based on the location of the loading lanes (NA to BS EN 1991-2 (BSI, 2008b)). There is no need to specify the number of cycles, since the use of this load implies that the fatigue life of the bridge is infinite.

FLM2

According to BS EN 1991-2 (2003), FLM2 represents ‘frequent’ idealised lorries for the highway traffic flow rates, as shown in Table 10.12. The aim in using this type model is to determine the maximum stress range resulting from all the lorries crossing the bridge. It should be noted that each lorry should cross the bridge alone in the appropriate slow traffic lane. In addition, the stress range generated will be compared with the constant amplitude fatigue limit. With FLM2, there is also no need to specify the number of load cycles and the use of this load model also will justify infinite fatigue life. The use of FLM2 is more accurate compared with using FLM1.

Lorry silhouette	Axle spacing: m	Frequent axle loads: kN	Wheel type (see EN 1991-2, Table 4.8)
	4.50	90	A
		180	B
	4.20	80	A
		140	B
		140	B
	3.20	90	A
		5.20	B
		180	B
		1.30	C
		120	C
	3.40	90	A
		6.00	B
		190	B
		1.80	B
	4.80	90	A
		3.60	B
		180	B
		4.40	C
		120	C
		1.30	C
		110	C

Table 10.12 Set of ‘frequent’ idealised lorry for FLM2 (BS EN 1991-2)

FLM3

Fatigue load model 3 is a simplified load model developed to encompass the concept of an equivalent stress range at two million cycles together with a ‘damage equivalent factor’, owing to the effect of variable amplitude loading in the traffic flow. The model uses a single fatigue vehicle, as shown in Figure 10.83,

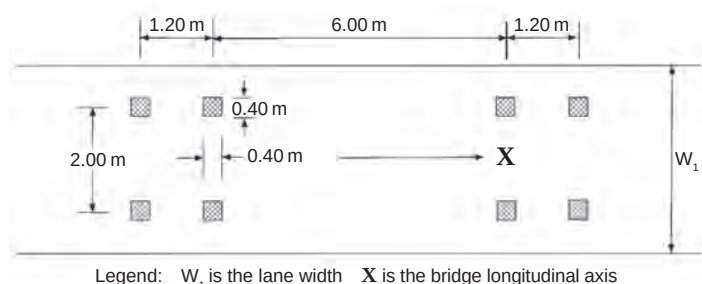


Figure 10.83 Set-up of FLM3 for road bridges (BS EN 1991-2)

which has four single axles, each with a weight $Q_k = 120$ kN. The use of this fatigue load model using a single-vehicle is limited to spans less than 40 m. For a span of more than 40 m, two vehicles should be traversing the bridge, one following the other, along the midline of the slow traffic lane with the second four-axle vehicle with a reduced load of 36 kN per axle. The geometry of these vehicles crossing the bridge is shown in Figure 10.84.

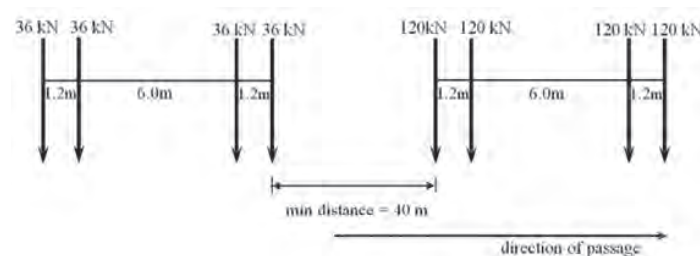


Figure 10.84 Set-up of two vehicles used for FLM3 for a bridge span of more than 40 m (BS EN 1991-2)

This model (FLM3) is associated with the concept of a standard fatigue vehicle (SFV) described in BS 5400-10 (BSI, 2000). Also, it is considered to cause the same fatigue damage (equivalent stress range) as 2 million actual traffic load cycles, corresponding to the number of heavy vehicles expected to load the bridge per year and per slow lane, as presented in Table 10.13. This is one of the reasons why this model is highly preferred by engineers. However, the use of this model might underestimate the actual applied loading, especially for long-span bridges, which, in turn, can jeopardise the fatigue performance of the bridge.

Traffic Category	N_{obs} per year, and per slow lane
1 Roads and motorways with two or more lanes per direction with high flow rates of lorries	2.0×10^6
2 Roads and motorways with medium flow rates of lorries	0.5×10^6
3 Main roads with low flow rates of lorries	0.125×10^6
4 Local roads with low flow rates of lorries	0.05×10^6

Table 10.13 Indicative number of heavy vehicles expected per year and per slow lane (BS EN 1991-2)

FLM4

Fatigue load model 4 consists of a set of five ‘equivalent’ lorries, as shown in Table 10.14. The Eurocode suggests that FLM4 should be used when the application of FLM1 to FLM3 fails to provide sufficient fatigue life (BS, 2008b). This could occur if the structural details susceptible to fatigue or the influence line length used to analyse the bridge does not provide sufficient information, owing to the typical vehicle wheel and axle spacings on the loaded length. Similar to FLM2, it is

expected that each lorry should cross the bridge alone in the appropriate slow lane traffic.

In addition, it is always recommended that the traffic runs in convoy across the bridge, with vehicles at 40m spacing from the centre of the rearmost axle of the front vehicle to the centre of the foremost axle of the vehicle behind, as shown in Figure 10.85.

Since the variation in the stress ranges will be directly due to the variable amplitude (VA) loading, this load model proposes a more accurate fatigue analysis compared with using FLM1 to FLM3. This is because the total damage accumulation D_d owing to the variable amplitude loading can be calculated using Miner's rule. However, the total damage accumulation D_d obtained from this load model should be determined by adding contributions from the traffic-alone cases and the convoy traffic running in lanes 1 and 2, as specified in BS EN 1991-2. Also, the use of this load model can be expected to produce an amount of fatigue damage equivalent to that caused by the actual traffic, as given in Table 10.15.

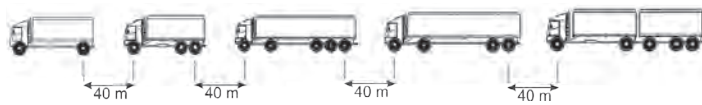


Figure 10.85 Convoy traffic allowable in FLM4

Type	Traffic category		N _{obs} per lane: millions per year	
	Carriageway layout	No of lanes per carriageway	Each slow lane	Each fast lane
Motorway	Dual	3	2.0	1.5
Motorway	Dual	2	1.5	1
All-purpose	Dual	3		
All-purpose	Dual	2		n/a
Slip road	Single	2		
All-purpose	Single	3	1.0	0
All-purpose	Single	2		0
Slip road	Single	1		n/a
All-purpose	Single	2	0.5	0
Local (low lorry flow)	Single	2	0.05	0

Table 10.15 Indicative numbers of heavy good vehicles expected per year and per lane (NA to BS EN 1991-2:2003)

FLM5

The fatigue verification concept used in this load model is similar to FLM4. However, it is based on actual recorded traffic data for a particular or specific location. This model (FLM5)

Vehicle type	Lorry	Axle spacing: m	Equivalent axle loads: kN	Traffic type			Wheel type
				Long distance Lorry %	Medium distance Lorry %	Load traffic Lorry %	
		4.50	70	20.0	40.0	80.0	A
			130				B
		4.20 1.30	70	5.0	10.0	5.0	A
			120				B
			120				B
		3.20 5.20 1.30 1.30	70	50.0	30.0	5.0	A
			150				B
			90				C
			90				C
			90				C
		3.40 6.00 1.80	70	15.0	15.0	5.0	A
			140				B
			90				B
		4.80 3.60 4.40 1.30	70	10.0	5.0	5.0	A
			130				B
			90				C
			80				C
			80				C

Table 10.14 Set of equivalent lorries used in FLM4

is associated with the use of structural monitoring tools, which can be used to record the stress ranges, which can then be counted to determine the total damage accumulation D_d in a similar manner to that used for FLM4. The advantage of using this load model is that the assumption of future traffic increases can be sensibly predicted.

Fatigue load for railway bridges

As defined in BS EN 1991-2 (BSI, 2003), the fatigue load for railway bridges is based on load model 71, as shown in Figure 10.86. This load model can be used together with load models SW/0 and SW/2. Load model SW/0 represents the static effects of vertical loading owing to normal rail traffic on continuous beams and SW/2 represents the static effect of vertical loading owing to heavy rail traffic. The provisions made for these load models consider the train speed, volume and maximum weight of the rail traffic, and also the track quality.

Application of equivalent fatigue damage accumulation in FLM

As mentioned previously, the failure expression is when the total damage accumulation $D_d=1.0$. Recalling that the randomness of actual traffic flow on the bridge generates variable amplitude (VA) loading histories, cycle counting also can be defined on the S–N curve (see Figure 10.81). Moreover, the stress ranges generated below the fatigue limit will not count as partial damage that will contribute to the damage accumulation. This is because, under the constant amplitude fatigue limit

(CAFL), $\Delta\sigma_D$, crack propagation will not occur until the crack reaches a certain size. The fatigue limit is described in Figure 10.87, which shows the stress ranges under CAFL and below the cut-off limit, $\Delta\sigma_L$.

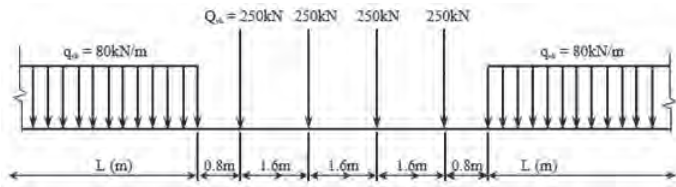


Figure 10.86 Load diagram of load model 71

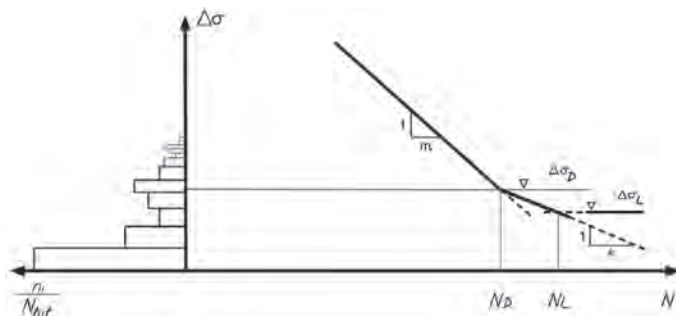


Figure 10.87 Influence of the stress ranges below the constant amplitude fatigue limit, $\Delta\sigma_D$, and the cut-off limit, $\Delta\sigma_L$; the slope $k=2m-1$ (Haibach, 1970) where m is the Wöhler slope (Nussbaumer *et al.*, 2011)

To use the damage accumulation equation (Equation 10.88) in a fatigue assessment, suitable safety factors for the fatigue strength, γ_{Mf} , and a safe-life design, γ_{Ff} , should be applied in the calculation. This is to ensure that an appropriate safety margin can be enforced in a fatigue appraisal, especially for a welded part.

Based on the FLM justification, it is clearly shown that only FLM4 and FLM5 will generate stress histories that have complex stresses ranges. Thus, the rainflow or reservoir counting method is needed to determine individual stress ranges; then, by using an appropriate S-N curve, the fatigue damage can be calculated. However, this approach is a little tedious. Thus, the Eurocode has provided a simplified fatigue design by using traffic simulation through the use of FLM3. The simplified procedure makes use of the concept of the 'fatigue damage equivalent factor'. The concept of the fatigue damage equivalent factor is outlined in Figure 10.88. The left side of the figure shows the fatigue check using real traffic, while on the right side of the figure use of FLM3 is shown. The damage equivalent factor λ is the link between the two approaches, ensuring equivalence between both methods.

Based on Figure 10.88, only the values of maximum and minimum stresses can be obtained. However, the critical detail

requiring fatigue assessment should be identified in order to ensure the accuracy of the determined stresses. Moreover, these stress values should be obtained by using the damage equivalent factor. This is to ensure that the resulting stress range $\Delta\sigma_{FLM}$ obtained has the same value as the equivalent stress range at 2 million cycles (Maddah and Nussbaumer, 2012; Nussbaumer *et al.*, 2011). As mentioned, the use of FLM3 is a relatively simple procedure. Thus, this procedure does not accurately represent the fatigue effect on the bridge owing to real traffic loading (Nussbaumer *et al.*, 2011). Standard BS EN 1993-2 (BSI, 2006) suggests obtaining the damage equivalent factor λ as

$$\lambda = \lambda_1 \times \lambda_2 \times \lambda_3 \times \lambda_4 \leq \lambda_{\max} \quad (10.89)$$

where

λ_1 is the factor for the damage effect of traffic and depends on the length of the critical influence line or area

λ_2 is the factor for the traffic volume

λ_3 is the factor for the design life of the bridge

λ_4 is the factor for the traffic on other lanes

$\lambda_{\max}$ is the maximum value of λ , taking account of the fatigue limit.

Although this simplified approach is preferred by engineers, its use is not suggested for long-span bridges, as the equivalent length proposed is up to 200 m. In addition to this, Chatzis *et al.* (2012) claim that the equivalent factors described in the Eurocodes need to be calibrated based on each region's traffic. This is because the traffic flow is different from one bridge to another.

The hot-spot method in fatigue appraisal

Fatigue damage may be difficult to determine in the vicinity of stress concentrations. With the complexity in geometry or loading in welded structures, a straightforward approach may not obtain an accurate fatigue strength. Thus, the hot-spot method approach has been developed to assist with a complicated stress field.

The hot-spot method is widely used for welded joints formed between plates and other structural members (BS 7608). As welded connections are commonly used in steel structures, this approach should be implemented in conjunction with the finite-element method in order to determine an accurate fatigue life. However, the results of stress magnitudes obtained from a finite-element analysis (FEA) are highly mesh-sensitive. Therefore, IIW (2008) and BS 7608 (BSI, 2014) recommend codes and guidelines to adopt when using this method to avoid any misinterpretations in the fatigue evaluation.

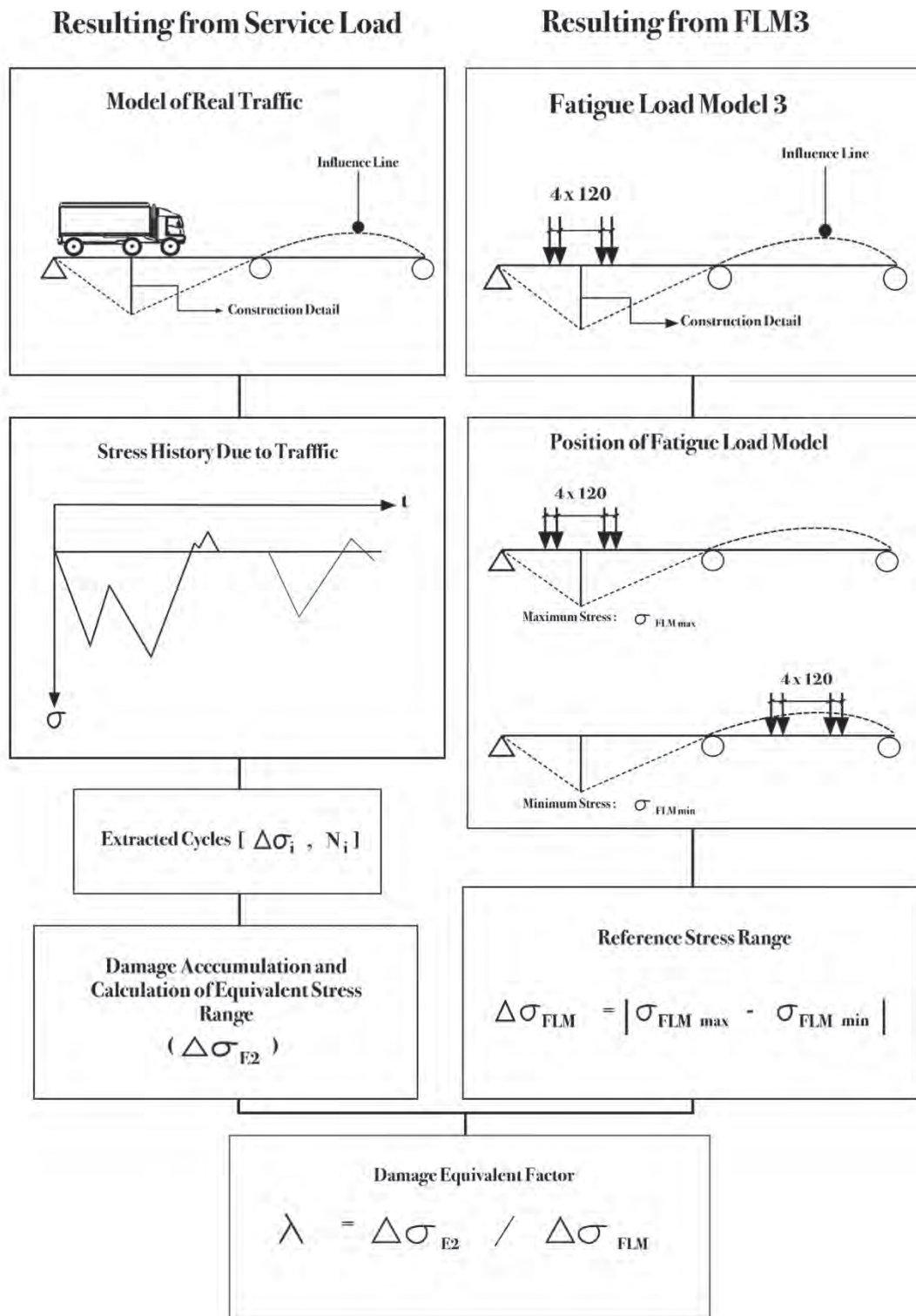


Figure 10.88 Damage equivalent factor concept (Maddah and Nussbaumer, 2012)

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Chapter 11

Composite construction

David Collings Arcadis

This chapter outlines the use of composite construction, the combining of materials such as steel and concrete, in a bridge structure. Composite construction often allows more efficient structures to be conceived than would be achieved by using either steel or concrete on their own. Each material can be used to advantage; concrete in compression areas, avoiding the buckling problems common with steel, and steel in tension areas, minimising cracking issues associated with concrete. The chapter outlines the common forms of composite bridges and reviews key factors governing their design, particularly at the important material interfaces.

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Introduction

Composite bridges are structures that combine materials like steel, concrete, timber or masonry in any combination. In common usage today, composite construction is normally taken to mean either steel and in situ concrete construction (Collings, 2013) or precast concrete and in situ concrete bridges (Hewson, 2012). The term composites is also used to describe modern materials, such as glass or carbon reinforced plastics. These materials are becoming more common but are beyond the scope of this chapter.

Composite structures are a common and economical form of construction used in a wide variety of structural types. This chapter initially reviews the forms of structure in which composite construction is used, then each of the more common forms of composite construction is considered in more detail. Compliance with codes (BSI, 2005a, 2005b, 2006) and regulations (AASHTO, 2017) is necessary in the design of a structure but is not sufficient for the design of an efficient, elegant and economic structure. An understanding of the behaviour, what physically happens and how failure occurs is vital to any designer. Without this understanding, the mathematical equations are a meaningless set of abstract equations. One aim of this chapter is to give an understanding of the behaviour of composite structures.

Future trends

Iron has been used in bridges since 1779, with the construction of the arch at Ironbridge over the River Severn. Concrete has been used for much longer, about 2000 years (O'Connor, 1993), but the modern development of concrete structures started in the 1800s. With the development of concrete structures, there was an era of experimentation, which included the embedment of iron in concrete for the first iron–concrete composite structures. Brunel's bridge near Paddington (Tucker, 2003) is an example of such structures. In more modern times, there was a period of experimentation in the late 1950s and early 1960s

(Svensson, 2005) when composite construction with embedded steel frames (Cracknell, 1963) and prestressed preflex beams, as well as the first precasting of deck units, were first used. One of the first steel–concrete bridges in the UK with a designed shear connection between the welded steel girders and the concrete deck was the 84 m span Thames Bridge near Bray (Allen, 1961), see Figure 11.1.

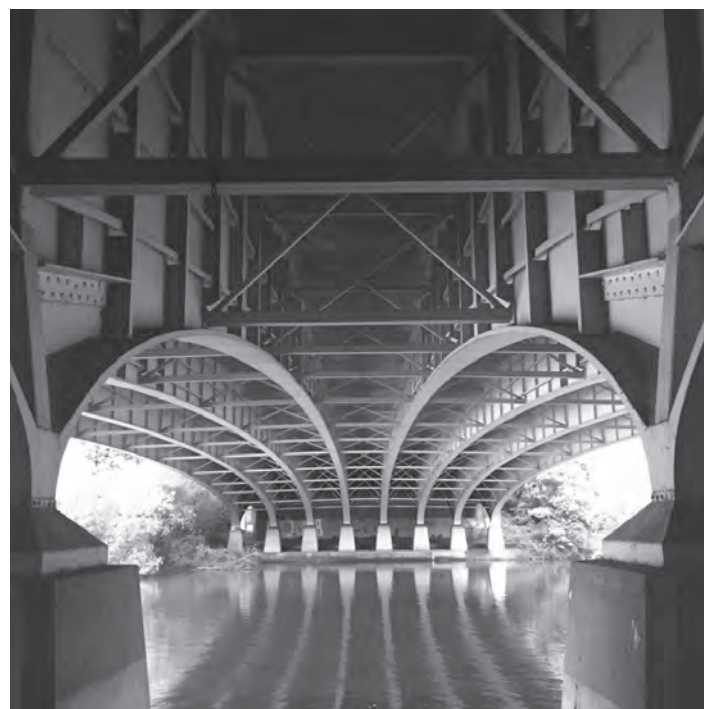


Figure 11.1 An early steel–concrete composite bridge at Bray

For a number of years, composite bridge designs have been relatively consistent, usually consisting of steel or precast concrete girders with an in situ deck slab. These types still form the majority of bridges built. To gain further advantages from prefabricating elements off-site, precast slab elements

(Gordon and May, 2006) for steel beams and precast shell elements (Collings and Brown, 2003) have been developed. Double composite members have efficiency advantages for larger continuous structures (Collings, 2008) and their use is likely to increase. For larger spans, extradosed bridges (Collings and Gonzalez, 2013), using prestressed steel–concrete composites with corrugated webs (Johnson and Cafolla, 1997), are becoming more popular. The environmental footprints of bridges are becoming more important, particularly embodied energy and carbon dioxide emissions during construction and in use. Composite structures can give reasonable energy and carbon dioxide emissions (Collings, 2006), particularly if care is taken in sourcing the materials. Further details on many of these newer structural forms can be found in *Steel–Concrete Composite Bridges* (Collings, 2013).

Structural forms

Most common composite structures are either precast prestressed concrete beams with an in situ concrete slab or steel girders with a concrete slab. Composite structures are very versatile and can be used for a considerable range of types of foundation, substructure and superstructure and forms, such as beams, columns, towers and arches, and for a diverse range of bridge structures, from tunnels and viaducts to elegant footbridges and major cable-stayed bridges. Figure 11.2 outlines the typical span ranges for composite bridges.

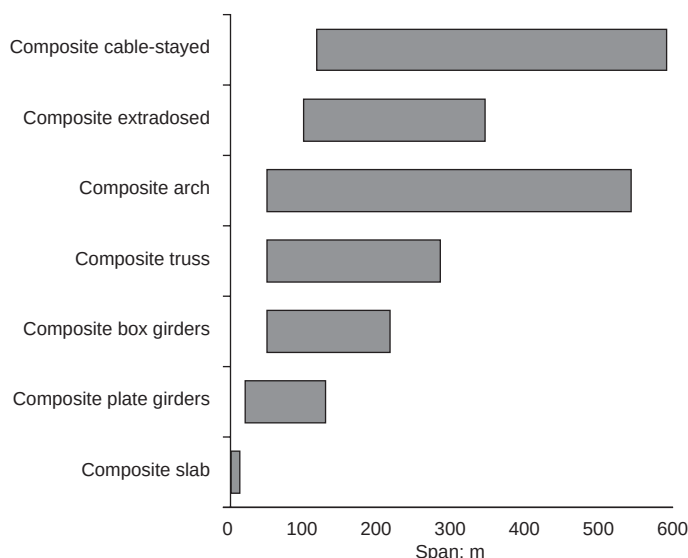


Figure 11.2 Span ranges for composite bridges

Foundations

Piles formed from driven cylindrical steel casings filled with concrete are used to carry loads into the ground. Pile sizes range from micro-piles 200–300 mm in diameter, which use high-strength steel pipe and cement grout, to large 2–3 m diameter caissons, formed from a cylindrical steel shell with a

concrete core (Biddle, 1996). Composite piles are also formed from concrete bored piles with embedded steel beams. This latter form of pile is sometimes used in contiguous or secant pile retaining walls subject to relatively large bending moments from the retained soil (Hubbard *et al.*, 1984), see Figure 11.3.

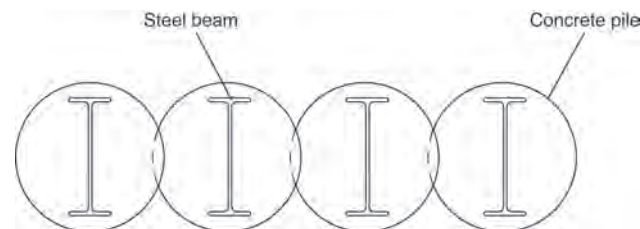


Figure 11.3 Composite steel–concrete secant pile wall

Substructures

Where high compression loads are to be carried, a composite steel–concrete column may be economic (Kerensky and Dillard, 1968), particularly where the size needs to be minimised. For the support of small- or medium-span structures, a concrete-filled rolled hollow section (up to 500 mm diameter) or pipe section (up to 2200 mm diameter) may be used.

Composite substructures are also useful in seismic areas. The use of embedded steel sections in concrete can avoid many of the local reinforcement buckling and bursting issues associated with reinforced concrete. The use of a steel tube section filled with concrete can considerably enhance the containment of the concrete compared with conventional reinforcement, significantly improving ductility.

Towers

For the towers of cable-stay bridges or suspension bridges, an outer fabricated stiffened steel plate structure with a concrete core could be used, or a fabricated steel core surrounded by reinforced concrete (Virlogeux *et al.*, 1994) (see Figure 11.4). The force regime in the towers of cable-stay bridges is extremely complex, with the stay anchorages imposing both horizontal and vertical loads. A fabricated steel core carrying the horizontal tensile component and local anchorage loads, with shear connectors transferring load to an outer concrete skin primarily carrying global compression and bending, provides an economical form and has been used on many of the world's largest structures. Where an outer steel structure is used, it serves the dual function of carrying the loads induced by bending and axial loads, as well as acting as a permanent formwork system.

Single-span beam-and-slab bridges

For modest simply supported spans, a composite beam-and-slab structure can give an optimal structural solution. The structure may be formed from a number of possible material combinations.



Figure 11.4 Steel–concrete transfer structure at tower cable anchorage of a cable-stay structure

Precast–in situ concrete composites

A bridge made of precast prestressed concrete beams forming the web and tensile flange with an in situ top slab constituting the compression flange uses these two types of concrete efficiently. This kind of structure usually avoids the extensive falsework normally associated with in situ concrete construction. For short spans in the 3–15 m range, the beams are often encased in concrete to form a slab. While this form may be relatively heavy, it is comparatively simple to construct and the large contact area between the beams and concrete in-fill ensures low stresses at the interface. Hence, additional reinforcement is not usually required. However, the most common method is to place a thin in situ slab over the beams; this form of construction is useful for spans up to approximately 30 m. Beyond 40 m, the weight of the beams means that craneage costs will be relatively significant and this form of construction is far less economic.

Steel–concrete composites

For small spans, filler joist construction may be used. Here, the beam is encased entirely in concrete (which is a relatively inefficient use of material in bending but is simple to construct); again, the stresses at the interface between steel and concrete are relatively low, such that no shear connectors are usually required for full composite action. This form of construction is popular for high-speed railway bridges (Kang *et al.*, 2018), as it has a high stiffness and reduces deflections and bridge accelerations, increasing passenger comfort.

A series of rolled steel beams forming the webs and tensile flanges, with a concrete top slab forming the compression flange, utilises the properties of the two different materials in a more efficient manner on simply supported spans of 10–25 m. Above 25 m, the symmetrical rolled section is usually replaced

by a purpose-fabricated asymmetric plate girder; at longer spans box girder sections are used. To ensure an adequate connection between the steel and concrete, shear connectors must be used at the interface.

Steel–timber composites

Timber is, in many ways, a more complex material than either steel or concrete. It has similar strengths in compression and tension, and it has a tendency to creep under sustained load. Being a natural material, it can have significant variations in its properties, and it is highly anisotropic, being significantly weaker when loaded across the grain rather than along it. However, it has a high strength-to-weight ratio. Studies in Canada (Bakht and Krisciunas, 1997) indicate that the lightness of a timber deck means that it is more economic than concrete for spans over approximately 50 m. This is particularly true in isolated areas where a good quality structural concrete is difficult to obtain.

Steel–timber bridges are not normally composite. Traditionally, the steel beam spans longitudinally, and the timber deck spans transversely. Because the stiffness of the timber is reduced by about one-sixtieth when loaded across the grain rather than along it, there is no advantage in composite action. Recently, a number of steel–timber composite bridges have been constructed, where the timber deck consists of a laminated construction spanning longitudinally between steel cross-frames. The deck is attached to the steel through shear connectors in pockets in the deck and the pockets are grouted with a fibre-reinforced grout.

Integral bridges

Integral bridges, where the superstructure is connected to the substructure, are preferable to those using joints and bearings, as bearings and joints allow movement and are the major cause of water ingress and deterioration of the bridge. The integral bridge is more robust, particularly with regard accidental impact and requires less maintenance of expansion joints and bearings. Essentially, the design of the composite integral structure is similar to a conventional bridge, but the effects of soil pressures and movements must be considered more carefully (England *et al.*, 2000). In the UK, it is mandatory that structures less than 60 m long with skews less than 30° should be integral. Longer structures are often built (Cooke and Collings, 2018).

Continuous beams

The use of a continuous beam (Figure 11.5) is often preferable to a series of simply supported spans, as the number of bearings and expansion joints is reduced. Both joints and bearings are potential long-term maintenance liabilities. However, in this form of structure, the continuity moments over the supports are of opposite sign to those at midspan and the composite section is not as structurally efficient, as the concrete slab is now in the

tensile zone. For composite precast prestressed beams, heavily reinforced in situ concrete stitch sections are required to create the continuity. For composite steel and concrete construction, cracking of the top slab will occur, causing some redistribution of moment. For similar length spans, it is common to assume that cracking occurs over 15% of the span length. Alternatively, cracked sections are assumed where the concrete stress exceeds twice its tensile strength (f_{ct}).



Figure 11.5 Continuous steel–concrete bridge spanning the M5 motorway, UK

Box girders

The box form is inherently stiffer and more stable than the plate girder form, since closed sections have a considerably greater torsional stiffness. Consequently, the box form is more suitable for slender structures, longer spans or curved bridges. For short spans, the simpler fabrication details of the plate girder make it more economic. For longer spans, where the high slenderness (λ) and low critical stresses require larger sections, box sections become the more economic form.

Composite box girders are generally of three types: a closed steel box of relatively compact proportions (Dickson, 1987), where plate stiffening and diaphragms are relatively minimal; an open-topped steel trough section that is made into a closed box by the concrete top slab (Bernardo et al., 2019) or multi-celled boxes (Curran et al., 2019).

The relative compactness of the small closed box means that the plate thickness is high and requires less stiffening and consequently less welding per tonne fabricated. However, the closed form means that either welding has to be carried out inside the box or that the fillet welds normally associated with plate-to-plate connections have to be substituted by partial penetration welds from outside. The closed-box type is commonly used in place of I-girders on bridges of high curvature or where aesthetics is an important criterion. The design of this form of box is essentially similar to that of the composite I-girder. The depth of the box should not be less than 1.5 m, or access inside it for essential maintenance and inspection would be difficult. The top flange of the box girder will act as permanent formwork for the concrete and must carry the weight of wet concrete. Once hardened, the flange may be assumed to act with the slab for local wheel loads. Additional connectors will be required for this purpose.

The open form of box girder, consisting of steel webs and a bottom flange, has only small top flanges, sufficient for stability during concreting but not closing the section. The advantages of this form are that access to all parts of the section for welding is available and that the web can be inclined, allowing a larger span in the transverse direction of the slab. In some situations, a single box may be sufficient, thereby minimising web material. Another significant advantage of this form in continuous construction is that the section can be made doubly composite (Collings, 2008). In the doubly composite section, concrete is placed in the bottom of the steel box adjacent to the supports (where the hogging moments occur) prior to placing of the top slab, see Figure 11.6. The use of the bottom concrete in compression reduces the size of the steel to only that required for construction.

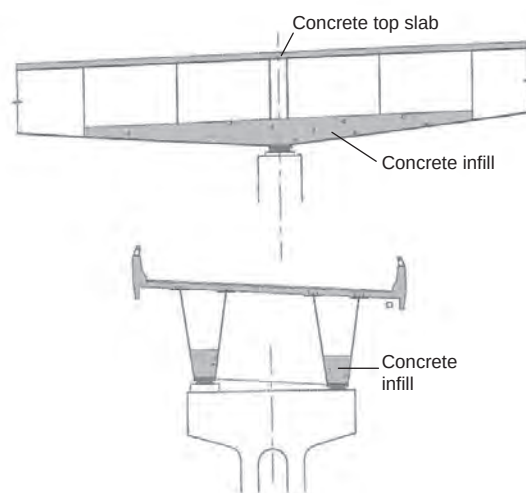


Figure 11.6 Double composite box girder

A disadvantage of the open box is that the high torsional stiffness of a closed section is not present during construction until the concrete slab has gained strength. For bridges with horizontal curvature, the torsional stiffness can be improved by the use of plan bracing. The plan bracing can be considered as an equivalent plate when calculating torsional properties.

Torsional properties

A circular section subject to a torsional load will resist load by a direct shear flow around the section. A square section will carry the load in a similar manner, but there will be a tendency for the section to distort (Figure 11.7). If the section is rectangular or trapezoidal, there will also be a tendency to warp under torsion (Chapman *et al.*, 1971) and both torsional warping and distortional warping will need to be considered.

The amount of distortion depends on the shape of the box, the transverse stiffness of the web and flanges, the spacing and stiffness of diaphragms and whether the torsional loads are point loads or uniformly spread. Typically, diaphragms are required at a spacing of three to four times the box depth.



Figure 11.7 Distortion and warping of a box girder

For larger boxes, closer diaphragms may be required, simply to aid fabrication and transportation.

Trusses

In its simplest form, a composite truss may consist simply of a steel truss with a concrete top slab forming the main compression chord (Mato *et al.*, 2014); see Figure 11.8. However, other forms utilising composite members in the truss are also used. Steel sections embedded in a concrete member or a tubular steel section filled with concrete may be used. For both forms, the behaviour and design of the joints is a key issue in design. Trusses usually have a relatively low span-to-depth ratio (10–16, as opposed to the 20–30 normal for beams) and are relatively stiff. Steel through trusses (Collings and Chiodi, 2011) are common for rail bridges, where the live loads are relatively large.



Figure 11.8 Ulla River composite truss viaduct for high-speed railway

Arches

The arch form is ideal for a material such as masonry or concrete whose strength is primarily compressive. However, the requirement for extensive centring, falsework or other temporary works often makes this form of structure uneconomic. Steel arches reduce the temporary works but require extensive bracing to prevent buckling instability, thereby increasing their material content. A composite structure with a steel framework erected first and the concrete cast around (or inside) utilises the properties of concrete and steel to maximum advantage. Composite arches of over 500m have been constructed using

this technique (Zheng and Wang, 2018; Zuou and Zhu, 1997).

Tied arches have been built using composite construction. Where a steel beam with a concrete slab is used at the tie level, the composite structure must be designed to resist the tensile forces from the tie. Under tension, the steel–concrete composite behaviour is non-linear, the concrete contributing to the initial stiffening effect but not to the ultimate strength of the element. An understanding of this behaviour is particularly critical near the arch–tie interface, where large local stresses and significant cracking can be induced in the concrete (Johnson, 2003).

The steel and concrete elements of the axially loaded members of arches usually have the same centroid (which is not the case in most bridge beams). Initially, the load will be distributed in proportion to the relative stiffness (EA) of the components making up the section. Normally, the primary steel load-bearing components would be fabricated and fitted together before encasing or filling with concrete. If the loads at this stage are significant, a check should be made of the capacity of the non-composite section. However, the section can normally be designed at the ultimate limit state, ignoring the build-up of stresses in the section.

For short members not prone to buckling, the ultimate squash load N_u is the sum of the capacity of the steel, concrete and steel reinforcement:

$$N_u = (A_s f_{ys} / \gamma) + (A_r f_{yr} / \gamma) + (0.85 A_c f_c / \gamma) \quad (11.1)$$

No more than 4% reinforcement by area should be used; reinforcement less than 0.3% should be ignored. The steel contribution factor (δ_c) should be between 0.2 and 0.9, where

$$\delta_c = (A_s f_{ys} / \gamma N_p) \quad (11.2)$$

For $\delta_c < 0.2$, the section should be designed as a concrete column, while for $\delta_c > 0.9$, the section should be designed as a steel strut. For some concrete-filled tubes, the capacity of the concrete is enhanced by the effects of confinement. For sections using very high-strength concrete or structures in seismic areas, this effect may be significant. For most practical design cases, the additional requirements of compactness and low applied moments are not met and the enhancement owing to confinement neglected. For practical design situations, the capacity of the section is influenced by buckling instability and the load that can be carried by the section is less than the squash load

$$N_R = k_1 N_u \quad (11.3)$$

where k_1 is determined from a buckling curve. For most practical applications, the column will be subject to bending effects. For columns with both bending and axial load effects, there will be an interaction between the two effects (Bergmann *et al.*, 1995). Figure 11.9 outlines the interaction by comparing a composite tube with a steel tube.

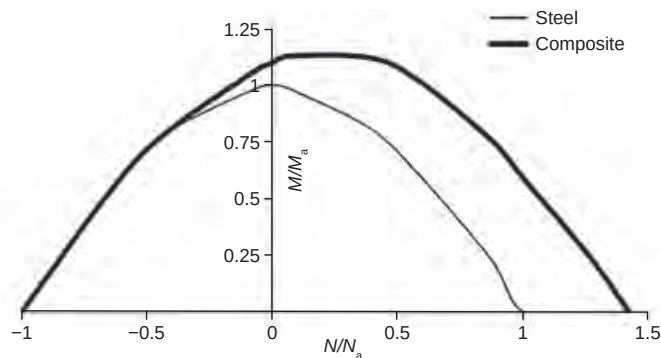


Figure 11.9 Axial load–bending interaction curve for composite arches

Connections in composite arches, columns and trusses are essentially similar to those commonly used in steel structures. The strength of the connection must be adequate to carry the proportion of load on the steel element of the composite section. Bolted connections should use high-strength friction grip (HSFG) bolts designed such that slip does not occur (this would transfer additional loads to the concrete). For concrete-filled tubular sections, the connections are usually made directly to the steel and provision must be made to distribute the loads to the concrete. Where the shear stress at the steel–concrete interface exceeds 0.4 N/mm^2 , shear connectors should be provided to carry the load.

Cable stays

The development of cable-stayed bridges is contemporary with that of composite structures; both arose primarily in the latter part of the twentieth century. Most of the long-span cable-stay bridges combine steel and concrete elements; many have some composite components (Collings, 2013). A cable-stay bridge consists of a deck, stayed by high-tensile wire or strand from towers. The most common form is a single main span with smaller back spans and two towers. Asymmetric layouts hung from a single tower are popular for smaller spans and architectural statements (Collings and Brown, 2003). Multispan cable-stay structures are now also being developed with more rigid frame towers (Virlogeux, 2001). The stay cables are arranged in a harp or semi-fan arrangement; the true fan arrangement, with all stays converging at one point, is difficult to detail and rarely used.

Slabs

In situ slabs are often utilised in composite structures requiring significant formwork or falsework, particularly at the edge, where the slab cantilevers beyond the beams. The use of composite slabs with either steel or concrete permanent formwork often provides a convenient solution, requiring no falsework, with the flexibility of monolithic in situ construction. In the UK, concrete permanent formwork is commonly used on

bridges constructed over or adjacent to major roads, railways or rivers, where access to the soffit is limited.

Precast slabs are economic in major viaducts (Figure 11.10) and cable-stay structures, where otherwise large areas of in situ concrete would be on the critical path of construction. The precast elements are specially fabricated for the individual bridge to suit its geometry. The connection between precast elements and the beams of the bridge needs to be carefully designed and detailed to ensure composite action. This is particularly important in cable-stay bridges, where the slab not only carries traffic loads but also carries a significant proportion of the high longitudinal compression induced by the stay cables. The connection between the precast units must also be considered carefully with regard durability issues, particularly in hogging areas, where the slabs are subject to some tensile loading and all the cracking tends to concentrate at joints.



Figure 11.10 Precast slab on a composite truss bridge

There has been extensive research into the local behaviour of slabs on composite bridges (Collings and Sagaseta, 2015, 2021). This research indicates that, for restrained internal panels, the slab carries a significant proportion of the load by arching action (Figure 11.11), with the slab behaving as a membrane as well as a flexural element. The use of compressive membrane action for internal restrained slab panels can lead to significant reductions in reinforcement quantities.

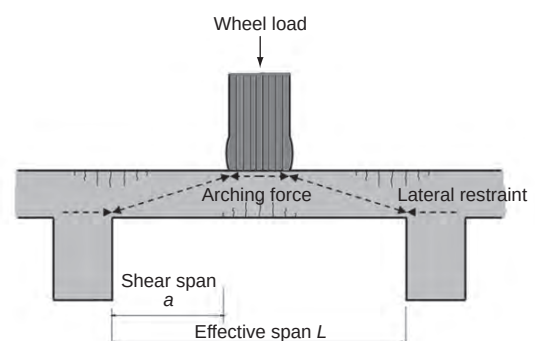


Figure 11.11 Arching action in a concrete slab

Materials

The behaviour of the composite structure is heavily influenced by the properties of its component materials. A brief summary of the key properties particularly relevant to composite action is given in Table 11.1; however, the reader wanting to understand composite bridges should first have a good understanding of the properties and design methods for the individual materials as set out in other chapters of this book (see Chapter 8 on reinforced concrete and Chapter 10 on steel). In particular, the reader should note the differences between materials, as it is the exploitation of these different properties that makes composite construction economic. For example, the use of a concrete slab on a steel girder uses the strength of concrete in compression and the high tensile strength of steel to overall advantage.

Material	Strength: N/mm ²	Elastic modulus: kN/mm ²	Other properties
Steel	$f_y = 250\text{--}500$ similar in tension and compression	$E = 210$	Tensile range may be affected by fatigue and compression range by buckling.
Concrete	$f_c = 30\text{--}60$ $f_{ct} = 3\text{--}5$	$E = 25\text{--}25$	Creep and shrinkage effectively reduce the elastic modulus over time.
Timber	$f_u = 12\text{--}25$ $f_c = 10$ similar in tension and compression	$E_{\text{long}} = 10\text{--}20$ $E_{\text{trans}} = 1$	Creep and shrinkage effectively reduce the elastic modulus over time.

Table 11.1 Comparison of material properties

Basic concepts

There are two primary effects to consider when looking at the basic behaviour of a composite structure:

- the differences between the materials
- the connection of the two materials.

The modular ratio

Differences between the strength and stiffness of the materials acting compositely affect the distribution of load in the structure. Stronger, stiffer materials, like steel, attract proportionally more load than materials such as concrete or timber. To take such differences into account, it is common practice to transform the properties of one material into that of another by the use of the modular ratio (n). Transformation to a single material simplifies calculations of section properties and stresses. Figure 11.12 shows the transformation of a steel girder with a concrete slab to an equivalent all-steel section.

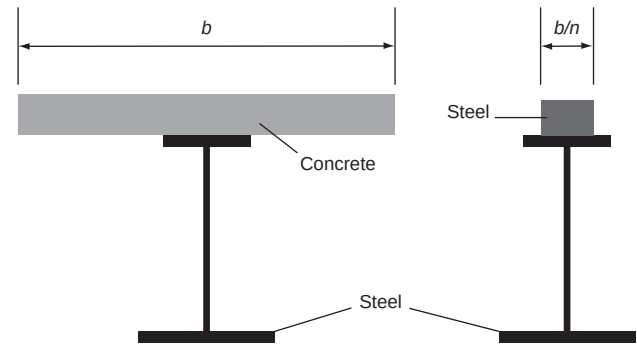


Figure 11.12 Transformed section for analysis

At working or serviceability loads, the structure is likely to be within the elastic limit and the modular ratio (n) is the ratio of the elastic moduli of steel (E_s) and concrete (E_c). For a steel–concrete composite, the modular ratio is

$$n = E_s / E_c \quad (11.4)$$

The value of this ratio varies from 7 to 15, depending on whether the short-term or long-term creep affected properties of concrete are utilised.

At ultimate loads, the modular ratio (n_u) is the ratio of the material strengths. This ratio depends on the grade of steel and concrete utilised. For design, the different material factors need to be considered to ensure a safe structure. For a steel concrete composite

$$n_u = k f_{ys} / f_c \quad (11.5)$$

where f_{ys} is the steel yield strength, f_c is the concrete (cylinder) strength and k depends on the code of practice being used and the values of the partial safety factors (γ_1) prescribed; k is approximately 1.7 to 2.

Interface connection

The connection of the two materials of the composite structure is of vital importance. If there is no connection, the two parts will behave independently. If adequately connected, the two parts act as one whole structure, potentially greatly increasing the structure's efficiency.

Imagine a small bridge consisting of two timber planks placed one on another, spanning a small stream. If the interface between the two were smooth and no connecting devices were provided, the planks would act independently. There would be significant movement at the interface and each plank would, for all practical purposes, carry its own weight and half of the imposed loads. If the planks were subsequently nailed together, such that there could be no movement at the interface between them, then the two parts would act compositely and the structure would have an increased section for resisting the loads and could carry about twice the load of the non-composite planks. The deflections of the composite structure would also

be smaller by a factor of approximately four, the composite whole being substantially stronger and stiffer than the sum of the parts. A large part of the criteria in the following sections is aimed at ensuring that this interface connection between parts is adequate.

The force transfer at the interface for composite sections is related to the rate of change of the force in the slab (usually simply the rate of change in moment). At working loads, or the serviceability limit state, the shear flow q_n at stage n is

$$q_n = (\delta M_n / \delta x) / t y \delta y \quad (11.6)$$

or

$$q_n = V A y / I_n \quad (11.7)$$

where V is the shear force at the cross section equal to the rate of change of the moment ($\delta M_n / \delta x$) in a uniform beam, A is the effective area further from the neutral axis than the level considered, y is the vertical distance from the neutral axis to the centroid of area A and I_n is the second moment of area of the effective cross section of the member. From Equation 11.7, it can be seen that the longitudinal shear is a function of the vertical shear force. For uniform loads, the longitudinal shear will vary linearly with the maximum at the support and a zero requirement at midspan. For concentrated loads, particularly those acting near supports, the local shear flows may be large.

As more load is applied to the structure, the force in the slab increases. At the ultimate limit state, the maximum change in force (F_Δ) in a slab of width b and depth h over a length L from the support to the midspan is

$$F_\Delta = 0.6 f_c b h \quad (11.8)$$

and the shear flow at this stage is

$$q_u = (2 F_\Delta) / L = (1.2 f_c b h) / L \quad (11.9)$$

From Equation 11.9, it can be seen that the maximum longitudinal shear is a function of the compression capacity of the slab. If the section is capable of significant plastic deformation, the shear flow may be considered to be uniform. Eurocode 2 (BSI, 2005a) notes that the change in force will also induce shears in the slab, which will need both longitudinal and transverse reinforcement.

For the composite section to function, two key criteria should be met.

- At working loads, slip should not occur.
- At the ultimate condition, the interface must have sufficient strength.

For design purposes, a simplified approach is adopted by current codes. The section is designed at the ultimate limit state, ignoring creep, shrinkage and other secondary effects. However, the design shear flow used is the elastic value given by Equation 11.7, using the ultimate shear force. Typically, codes allow a 10%–20% variation from the elastic shear distribution. At the ultimate limit state, the longitudinal shear plane (the

interface between the beam and the slab) must have sufficient capacity, through the addition of reinforcement or connectors, to resist the shear flow.

Precast concrete composites

Concrete-to-precast concrete composite construction is a relatively good point to start in looking at composite action as the elastic moduli and strengths of the two composite elements are similar and the complications of modular ratios can often be ignored.

Consider the behaviour of a simply supported precast prestressed beam with an in situ concrete slab cast on top. Initially, the beam will be subject only to its own self-weight and the initial prestress. When the slab is placed on the beam, the stresses in the beam change, with the compression at the bottom of the beam reducing. As other loads, such as surfacing and live loads, are added, the fully composite section resists the load. A compression develops in the slab, with the compression at the bottom of the beam reducing further. At these normal working loads, the concrete is uncracked and the shear stresses at the beam slab interface are small. There is no relative slip between the slab and beam. The stresses in the section are the sum of the stresses induced by each stage of construction (Figure 11.13). The sum of these stresses should be less than the allowable limiting value at the stage considered:

$$f_n > M_1 / W_1 + M_2 / W_2 + M_n / W_n \quad (11.10)$$

where M_n is the moment on the section and W_n is the relevant section modulus of the composite section at stage n .

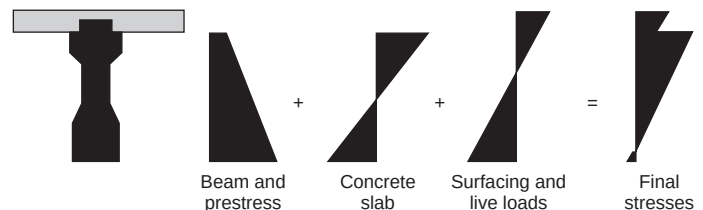


Figure 11.13 Build-up of stresses at various stages of concrete-precaster concrete composite construction

The forces at the slab-beam interface will also increase at each stage of construction:

$$q_n = q_1 + q_2 + q_3 \quad (11.11)$$

where q_1 , q_2 , and so on, are calculated from Equation 11.7 using the properties appropriate to each stage. At the serviceability limit state, there may be some additional longitudinal shear from shrinkage, creep and temperature effects.

The capacity of the shear plane may be estimated as the lesser of

$$q_p = c f_{ctd} L_s + \mu A_s f_{yr} \quad (11.12)$$

and

$$q_p = 0.5 L_s v f_{cd} \quad (11.13)$$

where c is the roughness factor, μ is the friction factor and v is the strength reduction factor for 40 MPa concrete.

To ensure that the interface slip is small, the values of c , μ and v in Equations 11.12 and 11.13 are relatively conservative. The coefficients are dependent on the roughness of the interface (Table 11.2). To ensure a robust structure and that there is not a pulling apart of the components, a minimum area of reinforcement (A_s) of 0.15% is normally recommended to pass through the shear plane.

Type of shear plane surface	Roughness factor, c	Friction factor, μ	Strength reduction factor for 40 MPa concrete, v
Smooth	0.2	0.6	0.5
Rough as-cast surface	0.80	0.7	
Indented surface	0.50	0.9	

Table 11.2 Ultimate interface shear flow for composite members

The vertical shear capacity of the section at the ultimate limit state should be larger than the flexural capacity, to ensure a ductile failure. The shear capacity of the beam may be conservatively assumed to be carried entirely by the precast beam, assuming it to be cracked. If there is significant prestress and the section is uncracked, a method that involves superimposing the principal stresses at the composite beam neutral axis may be used to derive a less conservative shear capacity for the composite section (Clarke, 1983).

Creep and shrinkage

For a simply supported span with a relatively compact section, the effects of creep and shrinkage in the concrete may normally be neglected. In some cases, particularly slender construction or some continuous beams, differential creep and shrinkage between beam and slab has a significant effect and should not be neglected.

Shrinkage

The shrinkage deformation of concrete depends on the environment (particularly the humidity), the composition of the concrete (the water–cement ratio and the cement content), the

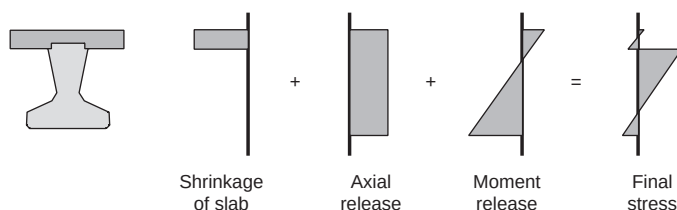


Figure 11.14 Stresses induced in the composite section as a result of shrinkage

size of the member, the amount of reinforcement and the age of the concrete. For concrete-to-concrete composite sections, the amount of differential shrinkage is important. The slab is cast after the beam and has a different rate of shrinkage, imposing a force across the interface (Figure 11.14). Because the slab is not at the neutral axis of the section, a moment is induced. These additional forces and moments may need to be considered in design.

Creep

Creep is a time-dependent deformation of the concrete that, like shrinkage, depends on environmental conditions, the composition of the concrete and the thickness of the section. It also depends on the concrete maturity at loading and the levels of applied stress or strain. The effect of creep is significant, as it can reduce the effect of the shrinkage strains and redistribute load from the beam to the composite section:

$$M = [1/(1+\phi)] M_{\text{beam}} + [1/(1+\phi)] M_{\text{composites}} \quad (11.14)$$

where ϕ is a creep coefficient, which varies from approximately 1 to 3. Where the effects of creep are large, the composite section will eventually carry all of the applied loads.

Continuous construction

Methods of forming a continuous bridge from precast beams vary considerably. Three common types are shown in Figure 11.15. The method in Figure 11.15(b) provides continuity of the slab only, improving durability by removing the expansion joint. Many spans can be joined together, forming a long continuous deck. The design of the beam is essentially as a simply supported span. The only additional complexity is in

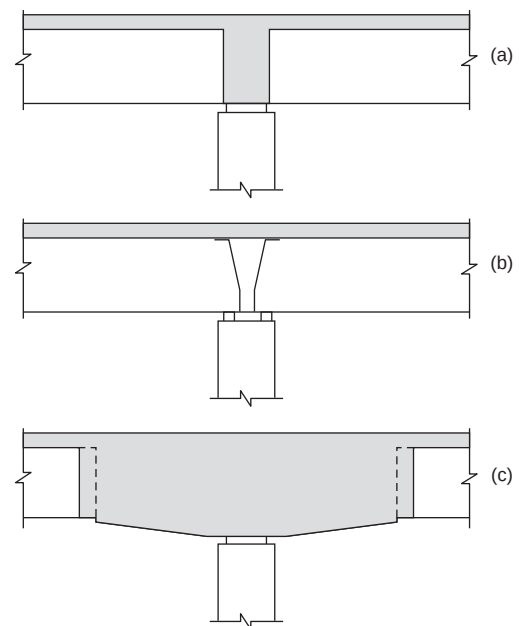


Figure 11.15 Continuity joints in precast beam bridges

the design of the slab. At the joint, the slab must be designed to carry normal live loads, longitudinal tensions or compressions from the bearings and accommodate the rotational movements of the two adjacent spans. This is achieved by ensuring that the slab is debonded from the beam over a sufficiently large length (usually about five times the slab depth).

By placing a reinforced concrete diaphragm at the supports, as shown in Figure 11.15(a), the precast elements can be made continuous, avoiding joints and reducing the number of bearings. This form of structure would normally be constructed by placing the beams on temporary bearings and then casting the top slab on the simply supported beams. The diaphragm would then be cast, forming continuity for live loads. At normal working loads, the diaphragm is subject to a hogging moment from live loads, as is the part of the precast beam adjacent to it. The design and detailing of the precast beam is complicated by the reversals of moment, often leading to the need for extensive debonding or deviation of the prestressing wires, increasing the cost of the unit. The precast beams are prestressed to counteract the main dead and live loads; consequently, the stress diagram is likely to be more compressive on the lower flange. The creep effect will be larger here and the result is a tendency for the beam to curve upwards with time, causing a long-term sagging effect at the diaphragm that requires reinforcement in the bottom of the diaphragm. At the ultimate limit state, there is likely to be significant cracking at the diaphragm and a redistribution of moment from the support to midspan. The bridge capacity might not be significantly greater than that of a simply supported bridge; consequently, this type of structure is rarely economic.

The use of a larger in situ concrete section at the support, with the precast beam starting near the point of contraflexure, as shown in Figure 11.15(c), may aid simplification of beam design, as the reversal of moments will be lower. The potential span of the bridge is also increased. However, the increased in situ concrete means that falsework is required, as is a significant temporary support for the precast beam. Considerable care is required in the design of the connection between in situ and precast elements, as the shear transfer from the edge beam can be particularly problematic.

Steel–concrete composite beams

The properties of steel and concrete are very different; an understanding of the behaviour of the structure and the interface between materials is vital.

Consider the idealised behaviour of a simply supported composite bridge comprising a steel beam with a concrete top slab. Initially, on completion of the slab construction, the unproped steel section only is stressed (ignoring at this stage the effects of shrinkage) and there is no force transfer at the steel–concrete interface. For loads added after this stage, the composite section carries them. Stresses in the beam increase, stresses in the slab occur and there is a force transfer at the steel–concrete

interface. The force at the interface is the rate of change of force along the slab (see Equation 11.7). The development of stresses is shown in Figure 11.16.

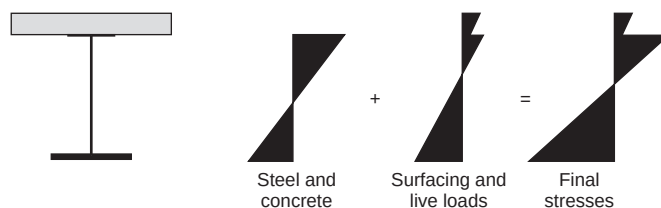


Figure 11.16 Build-up of stresses at various stages of steel–concrete composite construction

The load is increased to cause yielding of the bottom flange (to ensure ductile behaviour, this must occur prior to concrete crushing). Any further increase of load increases the zone of yield in the beam until the section has become fully plastic or fails in a local buckling for class 3 and 4 structures that cannot develop full plasticity. Because of the redistribution of stresses, the composite section will now be carrying more of the total load, including part of the steel and concrete self-weight originally carried entirely on the non-composite steel beam.

For this behaviour, it is implicitly assumed that the interface connection does not fail or deform significantly prior to failure of the composite section. The connectors at the interface have their own load–deflection behaviour. Any significant movement or slip at the interface will reduce the capacity of the structure.

A number of important issues must be noted.

- Below the elastic limit, the force at the steel–concrete interface is proportional to the load applied after the composite connection was made.
- At failure, the force at the steel–concrete interface is proportional to the total load on the bridge.
- The ultimate strength of the connectors at the interface must be greater than the forces applied at the interface.
- The slip at the interface must be small.
- Additional connectors may be required at changes of section.

Section properties

In steel–concrete composite construction, the strengths and stiffnesses of the two materials are very different. In the analysis and design of a structure, it is normal to transform the concrete to an equivalent steel area using the appropriate modular ratio (see Equations 11.4 and 11.5 and Figure 11.12).

The compactness of the section is also important in determining the section properties to be used. The definition of plastic (class 1), compact (class 2), semi-compact (class 3) or slender (class 4) varies slightly with the design code used. The rules shown in Table 11.3 are a reasonable guide.

Section type	Flange outstands in compression	Depth of web in compression	Spacing of shear connectors
Compact	$7t_f$	$28t_w$	$12t_f$
Semi-compact	$12t_f$	$68t_w$	600 mm
Slender	Varies with shear lag; depends on $b:L$ ratio and span configuration Likely to require stiffening if greater than $16t_f$	From $68t_w$ to $228t_w$, the effective web thickness reduces to zero at $228t_w$	600 mm

Table 11.3 Shape limitations for beams

Compact sections can be designed assuming fully plastic section properties. Semi-compact sections use the full elastic section properties. The design of slender sections needs to take into account the effective reductions in web or flange areas that might occur as a result of out-of-plane buckling and shear lag. The differing modes of failure are illustrated by their idealised load–deflection curves, as shown in Figure 11.17.

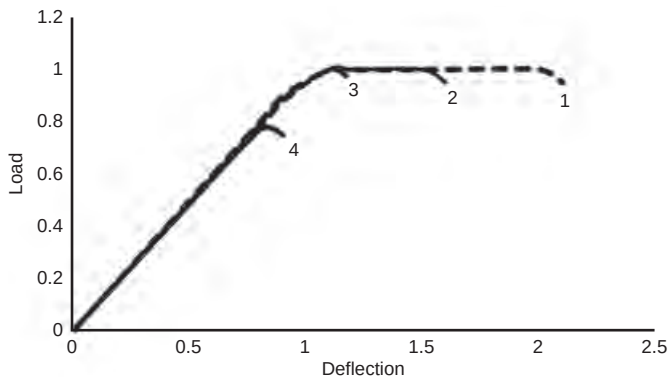


Figure 11.17 Idealised load–deflection curves for various section classes

Shear connectors

Shear connectors are devices for ensuring force transfer at the steel–concrete interface. They carry the shear and any co-existent tension between the materials. Without connectors, slip would occur at low stresses. Connectors are of two basic forms, flexible or rigid (Figure 11.18).

- **Flexible** connectors, such as headed studs, behave in a ductile manner, allowing significant movement or slip at the ultimate limit state (Figure 11.19). At the serviceability limit state, the loads on the connectors should be limited to approximately 70% of the connectors' static strength, to limit slip.
- **Rigid** connectors, such as fabricated steel blocks, behave in a more brittle fashion. Failure is either by fracture of the weld connecting the device to the beam or by local crushing of the concrete.

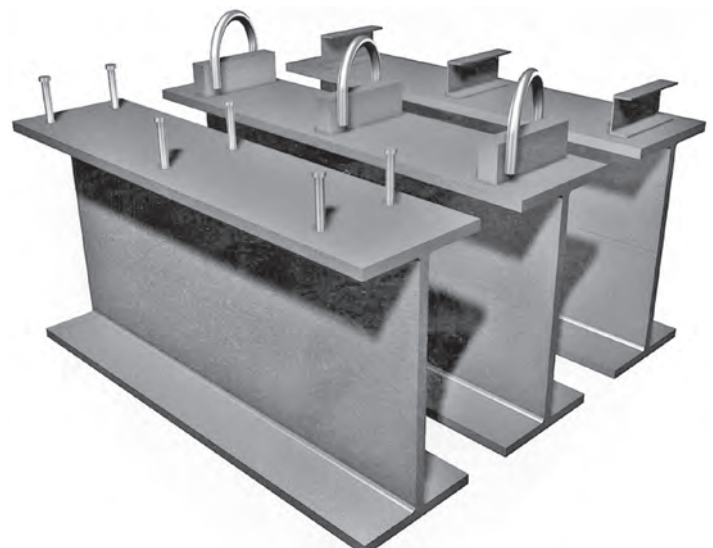


Figure 11.18 Typical shear connector types for steel–concrete composite construction

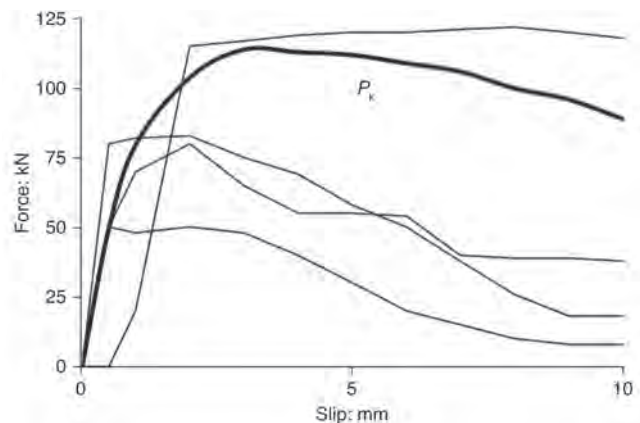


Figure 11.19 Typical load–slip curves for connectors in composite construction

Typical nominal static strengths P_u for various connector types for grade 30 N/mm² concrete are given in Table 11.4. If there is a tension on the connector larger than 10% of its nominal static strength, a more positive purpose-designed tensile connection should be considered.

For headed stud connectors, the capacity can be determined based on a tensile failure of the weld

$$P = 0.8 f_u \pi d^2 / 4 \gamma_v \quad (11.15)$$

or a compression failure of the concrete local to the connector

$$P = 0.29 \alpha d^2 (f_{ck} E_{cm})^{0.5} \gamma_v \quad (11.16)$$

At the serviceability limit state, there should be sufficient shear connectors to prevent excessive slip. The number of connectors required per unit length is

$$N^* = q_n / 0.7 P_u \quad (11.17)$$

Type of connector	Connector material	Static strength per connector for $f_c = 30 \text{ N/mm}^2$: kN
Headed studs, 150mm height	Steel: characteristic strength yield stress, 385 N/mm; minimum elongation, 18%	
Diameter		
19		109
22		139
25		168
50 × 40 × 200mm bar with hoops	Steel: characteristic strength yield stress, 250 N/mm ²	963
Channels	Steel: characteristic strength yield stress, 250 N/mm ²	
127 × 64 × 14.9kg × 150mm		419
102 × 51 × 10.4kg × 150mm		364

Table 11.4 Nominal static strengths of shear connectors

At the ultimate limit state, the number of connectors required is

$$N^* = (\gamma q_n) / 0.8 P_u \quad (11.18)$$

At the ultimate limit state, the failure of shear planes other than at the interface ($x-x$ in Figure 11.20) might need to be investigated. These shear planes will be around the connector ($y-y$) or through the slab ($z-z$). Where haunches are used, a check on shear planes ($h-h$) through the haunch may be required. Equations 11.12 and 11.13 are relevant, where $\mu = 0.9$ and $v = c = 0.5$.

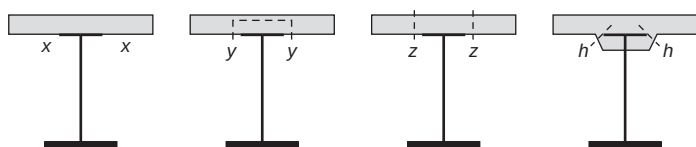


Figure 11.20 Typical shear planes in a steel–concrete composite structure

Construction methods

For steel–concrete composite bridges, the methods and sequences of construction are vitally important. Prior to the concrete being placed, the steelwork is relatively slender and often requires bracing to ensure stability. As the concrete is placed, significant stresses are set into the steelwork; the sequence of placing of the concrete has an influence on the final behaviour of the structure.

For continuous bridges, the sequence of construction will affect the distribution of loads. Concreting spans adjacent to those already composite will induce additional stresses along

the shear interface. It is common practice to construct midspan sections prior to support areas, as this reduces tensile stresses in the slab.

Crane erection

The majority of small- to medium-span composite bridges are built using cranes. For bridges with good access and space to manoeuvre a crane, this method is simple and very economical. There are, however, a number of conditions to be addressed. First, components must be of a size that is transportable to the site. In the UK, this generally means a maximum length of 27.4m and a width or height of 4.3m. Where access roads do not permit this size, or larger units are required, the partial assembly of components on-site may be necessary. Many types of crane are available, from mobile cranes hired in for a specific lift to crawler cranes for smaller but more numerous lifts or for lifts requiring the crane to move under load.

For any lift, the crane's foundation must be adequately considered, and ground improvements or special foundations provided, if necessary. This is particularly important for larger mobile units, where significant loads might be placed on the outriggers.

Lifting points in the form of lugs or cleats will need to be attached to the beam prior to lifting. Normally, these would be designed to be left in place and covered by the concrete slab. During lifting, the beam must be stable. For long girders, a bowstring brace might be required to stabilise the top flange. However, if beams can be lifted in pairs, with the bracing required for concreting in place, the crane erection stage might not be a critical condition.

Joints in the girder will be placed by the designer to occur near points of contraflexure to minimise forces in the joint, with due regard to transportation and lifting. Bolted joints using high-strength friction grip (HSFG) bolts are relatively quick and simple to build on-site. They can be installed while the beam is supported by the crane. Site-welded joints are sometimes specified; however, temporary support towers might be required to hold the girder during welding. The welding process is likely to take longer than bolting. It is also relatively costly, owing to the high initial costs of testing and quality control and the need for cleaning and the application of a full paint system. In general, site welding should be avoided, unless there is a sufficient number of joints to justify it. Aesthetic considerations are sometimes used as a justification for site welds but, in the author's opinion, falsely, as a well-positioned and detailed bolted joint is likely to look more functional.

Launched bridges

Incremental launching (Figure 11.21) involves erecting the steelwork at a single location behind one of the bridge abutments. Once a section of structure is erected, the whole structure is moved forward by the length of that section and the next section is then erected. For a composite bridge, either the steelwork only or the steelwork and the concrete deck are

constructed and launched. Launching of the steelwork only minimises the moments and shears on the girders and means that the rollers and jacking equipment will be comparatively modest. However, the deck slab will have to be cast using conventional suspended falsework after the launch. Launching the entire composite section minimises the construction time (Beavor and Cai, 2006) as no major concreting operations are needed after erection. However, the heavier deck does require larger jacks and rollers (or slide plates), and greater moments and shears during launching will require additional stiffening of the girder webs. Substructures might need bracing to limit movement under launching.

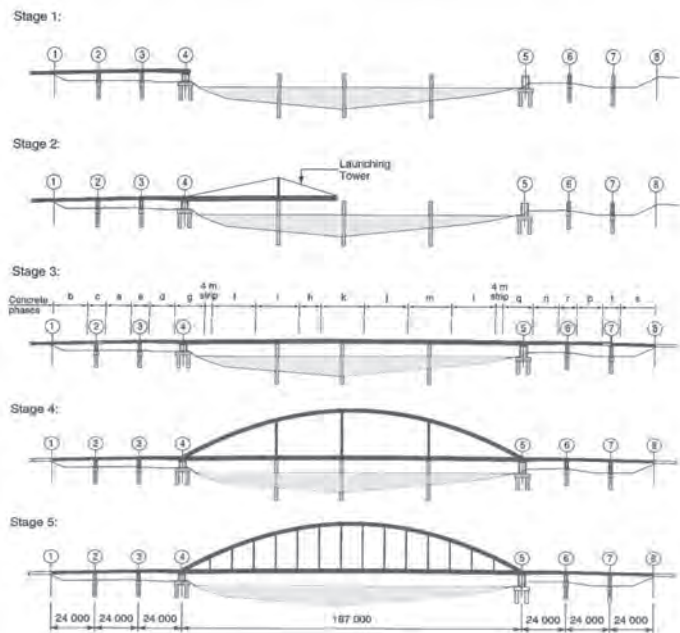


Figure 11.21 Launching of a composite arch bridge

The appropriateness of incremental launching as a method of construction depends primarily on the geometry of the structure. For launching to be economical and practical, the structure is likely to be a viaduct (probably over 200 m long), have relatively consistent spans (of 40–60 m) and be of constant horizontal and vertical curvature. During launching, the girders will be fitted with a launching nose. The purpose of the nose is to reduce the large cantilever moments that would otherwise occur in the deck prior to each pier being reached. The optimum length of the launching nose is usually that at which the cantilever moments induced in the girder adjacent to the nose are of a similar magnitude to those hogging moments that occur elsewhere. The launching nose will be profiled to overcome the tip deflection of the cantilever, such that it can land on the adjacent pier.

Cantilever construction

Cantilever construction is used extensively for the construction of composite cable-stay structures (Curran *et al.*, 2019) and is

appropriate for trusses (Collings and Chiodi, 2011) and arches (Zheng and Wang, 2018). It is not common in the erection of girder bridges because of the significant number of joints required.

Bracing of steelwork

For composite bridges constructed from steel beams or girders with concrete slabs cast on top, the construction stage is a critical area that needs careful consideration. The steelwork is relatively slender and requires bracing to ensure stability during construction. The bracing must be designed to ensure that all likely buckling modes are suppressed. This includes instability of the girders between bracing points and overall instability of the whole bridge.

For an individual beam or girder, the main mode of instability is lateral torsional buckling. The beam undergoes a simultaneous lateral movement and rotation. The tendency for a beam to buckle is influenced by a number of factors, including the nature of the external loads, the beam shape, the bracing type, strength and stiffness, fabrication tolerances, the length between effective bracing, residual stresses (from rolling or welding) and the stress in the compression flange. Bracing, in the form of plan bracing, anchoring to a rigid object, transverse bracing or a combination of methods can be used to suppress lateral torsional buckling. Common forms of bracing are outlined in Figure 11.22.

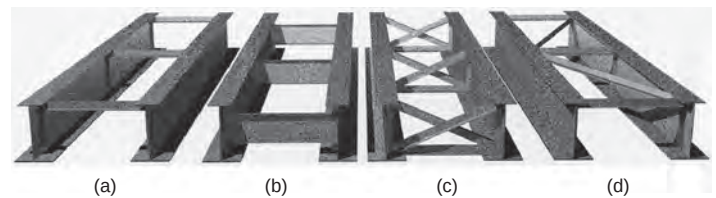


Figure 11.22 Bracing types for steel–concrete composite construction: (a) ineffective bracing; (b) U-frame bracing; (c) cross bracing; (d) plan bracing

For the bracing to function, it must have sufficient strength to resist the applied loads. The destabilising forces generated by the compression flange are generally considered to be approximately 2.5% of the sum of the forces in the flanges being connected, or half of this value when considered with wind or other significant transverse effects. The force can vary significantly; the more flexible the bracing, the larger the destabilising force to be resisted.

Typically, bracing is required at 12–15 times the flange width. The determination of limiting stresses is based on a slenderness parameter λ using an effective length concept.

For torsional cross bracing, shown in Figure 11.22(c), the bracing may be considered as rigid. For U-frame braces (Jeffers 1990), shown in Figure 11.22(b), plan bracing (Figure 11.22(d)) should also be used, unless the U-frame is sufficiently rigid, $l_b \ll l_e$, where

$$l_e = 2.5 (EI_y l_b \delta)^{0.25} \quad (11.19)$$

with δ the deflection caused by a pair of unit forces on the U-frame and l_b the distance between rigid bracing.

There are two primary methods of reducing instability.

- Increase the lateral strength and stiffness, either by the use of plan bracing or by increasing the width of the compression flange (increasing i)
- Increase the torsional strength and stiffness by the use of transverse X- or U-bracing that connects the tension and compression flanges of adjacent beams. It should be noted that the connection of compressive flanges on adjacent beams (Figure 11.22(a)) will not provide stability unless the bracing has sufficient bending strength to act as a U-frame.

For larger bridges or slender footbridges, the use of cross bracing alone might not be sufficient to suppress overall instability of the pair of girders. The effective length of the beam pair is determined using the elastic critical buckling stress approach, as

$$\lambda = \sqrt{(\pi^2 E / \sigma_{ci})} \quad (11.20)$$

For I-girders, the torsional stiffness is generally small and the critical stress can be estimated by considering the warping strength of the girder pair:

$$M_{ci} = (\pi/l)^2 E (IK)^{0.5} \quad (11.21)$$

$$\sigma_{ci} = M_{ci} / 2Z \quad (11.22)$$

where K is the warping constant (Roark and Young, 1985):

$$K = (D^3 B^2 t) / 24 \quad (11.23)$$

Where the overall slenderness of the section is critical, either plan bracing is required or a more stable box section should be used. Alternatively, adjacent pairs of girders should be connected. When more than two girders are connected, the designer should be aware that the bracing will participate in the transverse spreading of load in the final condition. For this participating bracing, the forces in the final condition might govern their design and the detailing of the connections with the slab and main girders must also consider the effects of fatigue, particularly at the area adjacent to web, flange and bracing connections.

In hogging regions of continuous girders, the lower flange is in compression and nominally unrestrained for the full range of dead and live loads, and bracing is often provided in this area to restrain the flange. The deck slab in this area provides restraint in the form of an inverted U-frame; this means that the critical buckling mode involves some distortion and bending. The pure lateral torsional buckling mode given by codes is conservative. Research on buckling in hogging regions indicates that, for many practical beam layouts, the bracing is not required. Where the inverted U-frame method is used, the effects of the additional tensile forces on the connector capacity should be considered.

Fatigue

Fatigue is a phenomenon primarily affecting the steel elements of a composite structure. Fatigue is primarily influenced by the stress fluctuations in an element (the maximum range of stress, as opposed to the maximum stress), the structure geometry, the steel grade and the number of load cycles. The relationship between these variables is shown in Figure 11.23. The stress range prior to significant damage varies with the number of load cycles. At high stress ranges, the number of cycles to damage is low. There is a stress range below which an indefinitely large number of cycles can be sustained. For a bolted detail and a shear connector, the range is about 80 N/mm². For a welded detail, it is nearly half this value. For design, it is critically important to obtain a detail with the maximum fatigue resistance. This is achieved by avoiding sudden changes in stiffness or section thickness, partial penetration welds, intermittent welding or localised attachments.

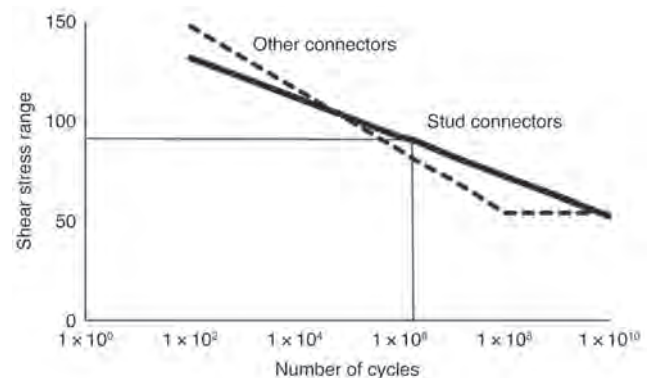


Figure 11.23 Typical stress range – number of cycles to failure for stud shear connectors and welded connectors

Composite highway structures are not particularly sensitive to fatigue problems if properly detailed. The design of shear connectors near the midspan might be governed by fatigue where static strengths dictate only minimum requirements but there is a high stress range. Participating permanent formwork, participating bracing and cruciform joints at crossheads are also areas that are sensitive to poor fatigue details. Railway bridges, with their higher ratio of live to permanent loads, are much more sensitive to fatigue.

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Chapter 12

Aluminium bridges

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Understanding the unique characteristics of aluminium alloys and exploiting these in ways developed by other industries, for example, transportation, architectural applications and consumer goods, will lead to the production of light, durable and whole-life cost-effective aluminium bridges. Adopting the concept of extrusions, placing material where it will be most efficiently used and expanding the use of friction stir welding (FSW) can be described as architecturally liberating, freeing engineers to propose new exciting lightweight bridge structures. Not only do aluminium bridge designers need to consider what are novel and appropriate bridge designs in their own countries, but they should also look at available design possibilities in, say, the USA, Canada, Europe and Australia.

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Introduction

Aluminium and aluminium alloys are not currently widely used in bridge design, partly through unfamiliarity, partly through misconceptions, but largely because designers have rarely been taught how to design in aluminium. Many bridge engineers may be surprised to learn of the 153 m long Arvida aluminium highway bridge (see Figure 12.1), with a main span arch of 91.5 m and a deck width of 10.36 m that was completed in 1950 and is still in service. A number of aluminium bridges are listed later in this chapter. The durability of aluminium can be seen in the aluminium dome of the church of San Gioacchino in Rome, which has been in service since 1897.



Figure 12.1 The Arvida Bridge over the Saguenay (courtesy of Alcan Aluminium Co. Canada)

Aluminium is a material that has exceptional properties that need to be exploited and worked with. Aluminium has excellent corrosion resistance, a high strength-to-weight ratio and can be strengthened by alloying, cold working or strain hardening; further, aluminium and its alloys do not lose ductility, nor do they become brittle at low temperatures. Aluminium and its alloys have a major advantage in that they can be extruded into a very wide range of sizes and cross-section profiles. The use of aluminium in a bridge will reduce the permanent load on the foundations, reduce transportation costs, allow modular prefabrication of sections of the bridge, and reduce construction time and on-site equipment. Aluminium has high corrosion resistance, reduced maintenance costs and a long working life. When used correctly, the results are a light, durable structure that is cost-effective. Aluminium and its alloys must not be designed as a light steel, nor must typical steelwork details be used, as aluminium design has specific non-steel requirements.

This chapter seeks to point the inquisitive person along the path to the successful use of aluminium, to some basic facts, allied to references to more comprehensive information. In this chapter, where the term 'aluminium' is used, in reality the structural materials referred to are aluminium and all alloys of aluminium that have small percentages of other elements added.

What is aluminium and its alloys?

Structural aluminium alloys are those where, by mass, aluminium predominates over each of the other elements. Structural aluminium alloys are divided into nine series.

- The 1xxx series is used for aluminium of at least 99% purity.
- The 2xxx to 8xxx series are grouped by the major aluminium alloying elements:
 - 2xxx copper
 - 3xxx manganese
 - 4xxx silicon
 - 5xxx magnesium
 - 6xxx magnesium and silicon
 - 7xxx zinc
 - 8xxx other elements
- The 9xxx series is unused.



Figure 12.2 Raalte Verkeersbrug (courtesy of Bayards Aluminium Constructions BV, the Netherlands)

Aluminium and its alloys are ductile and easily worked by standard manufacturing and shaping processes in a separate fabrication area away from other metal manufacturing. Aluminium and the majority of its alloys are highly resistant to corrosion, non-combustible, non-toxic and efficient conductors of electricity. Structural aluminium is wrought aluminium, with cast aluminium not usually used structurally. Wrought aluminium alloys that are heat-treatable include the 2xxx, 6xxx, 7xxx and some of the 8xxx alloys.

Why aluminium in bridges?

So why should we all consider aluminium? Predominantly, because it is light, durable, highly corrosion-resistant and has a wide diversity of structural forms and shapes that can be created at low cost.

These properties of aluminium and its alloys have been widely exploited in many structural applications, including motorway sign gantries, prefabricated buildings, roof members, space frame constructions and whole structural sections, such as offshore platform helidecks, aircraft structural members and laboratory structures in space; in the rail industry, for coach panelling, freight wagons, structures and tankers; in the shipping industry, for hulls, superstructures; in the transport industry, for tankers, tippers, traffic signs and truck bodies. The low self-weight, one-third that of steel, is extremely useful for handling during fabrication, construction and transport, as well as reducing weight in the final design. The durability of aluminium alloys is very high and is one of the most underestimated virtues of the material.

Structural aluminium alloys are available in a range of ultimate tensile strengths, generally between 125 and 350 N/mm², which will meet many of the most demanding requirements. Larger ultimate tensile strengths are available for specialist applications. Pure aluminium is a comparatively low-strength material but alloying with small amounts of other elements can significantly increase its strength. Consequently, the tensile strength of the applicable structural alloys is comparable with mild steel, plus aluminium's excellent low-temperature toughness eliminates concerns about brittle fracture in cold climates.

Aluminium's density is only one-third that of steel, giving a strength-to-weight advantage to aluminium, which is particularly beneficial during fabrication, transportation and erection,

as well as for the design of the completed and supporting structure. Aluminium's lower self-weight is especially relevant for bridge refurbishment, for moving structures and for reducing the inertial forces on elevated structures subject to earthquake.

Aluminium can be easily formed and extruded into many bespoke shapes that allow for structural efficiency, as well as ease of fabrication. The tonnages involved in most bridge applications make it feasible to create specific extruded sections and so the designer is free to create innovative and efficient solutions.

A significant reason for using aluminium is its excellent corrosion resistance. Under most atmospheric conditions, no protective coating is necessary, as illustrated by aluminium parapet systems that have been in service for many years (see Figure 12.2). Coatings can be applied for aesthetic purposes or to protect against specific aggressive environments.

Alloys and product forms

Many different structural aluminium alloys are available, and each may have different tempers or heat treatments, such that the total number of possible combinations run into many hundreds. For the bridge engineer, however, there are basically only three families of alloys, the 5xxx, 6xxx and 7xxx series, that need to be considered and a relatively small number of alloys and tempers within each family, as explained next. These alloys are all readily available, have good corrosion resistance and strength characteristics, and are easily fabricated. Alternative alloys are available and used in other industries, some offering much higher strengths at the expense of durability or workability, others offering better formability at the expense of strength. Additional alloys are being developed and gradually brought into use. Of particular note are the 2xxx series alloys, e.g. 2195 T8511, which uses lithium in addition to copper as a major alloying element, giving an ultimate tensile strength of up to 600 N/mm², and is slightly stiffer and less dense than the values given in EN 1999. Plate and extrusions are available and have been proven in aerospace applications.

Aluminium alloys are categorised by the main alloying element, as mentioned, and use an internationally recognised four-digit reference system. Table 12.1 lists all the structural wrought alloys in EN 1999-1-1:2007, while Table 12.2 gives some typical properties for aluminium alloys. The main alloys

3xxx series alloys	5xxx series alloys	6xxx series alloys	7xxx series alloys	8xxx series alloys
EN AW-3004	EN AW-5005	EN AW-6060	EN AW-7020	EN AW-8011A
EN AW-3005	EN AW-5005A	EN AW-6061		
EN AW-3103	EN AW-5049	EN AW-6063		
	EN AW-5052	EN AW-6005A		
	EN AW-5083	EN AW-6106		
	EN AW-5454	EN AW-6082		
	EN AW-5754			

Table 12.1 Structural aluminium alloys listed in EN 1999-1-1

that are of interest to the bridge engineer are described in the following.

The 5xxx series alloys have superior mechanical properties to the 1xxx, 3xxx and 4xxx series alloys, with better corrosion resistance than the 6xxx series alloys, and can be used in marine conditions. The 5xxx series alloys have magnesium as the main alloying element; these alloys have high strength, excellent corrosion resistance and good weldability. These alloys are generally only available in sheet or plate form. One design standard suggests that 5052, 5083, 5086 are suitable for bridge components. The alloys have increased strength from work hardening during the rolling process and are available in several different degrees of hardness (the letter O denotes the base condition; the letter H followed by two numbers denotes work-hardened material). A common alloy is 5083 H12. Proof strength and ultimate strength increase with the work hardening, but formability decreases. The 5xxx series alloys are readily available and are a good choice if forming a structure from plate materials.

The 6xxx series alloys are the most common aluminium alloys, being widely used in architecture, engineering and transport. The good corrosion resistance of these alloys means

that they can be used in marine and industrial environments. The 6xxx series alloys have magnesium and silicon added as the main alloying elements and have moderately high strength. These alloys are readily extrudable, as well as being available in sheet and plate form. They are the most commonly used in structural and architectural applications, principally on account of the forms and shapes that can be created by extrusion. The alloys have their strength increased by heat treatment processes and are available in a range of tempers (indicated by the letter T followed by a number, e.g. 6082-T6). They are readily weldable and give good all-round performance. One design standard suggests that 6005A, 6061, 6063 and 6082 are suitable for bridge components.

The 7xxx series alloys are high-strength alloys but have reduced corrosion resistance. Protection in moderate or severe industrial, marine and urban environments is required to prevent, for example, stress corrosion cracking. The 7xxx series alloys have zinc and magnesium as their main alloying elements. The 7xxx series are stronger than the 5xxx and 6xxx alloys and have their strength increased by heat treatment. They are harder to form and are more expensive than other common alloys. They are readily weldable and have good post-

Mass: kg/m ³	Modulus of elasticity: N/mm ²	Poisson's ratio	Coefficient of thermal expansion: $\times 10^{-6}/^{\circ}\text{C}$	Melting range: $^{\circ}\text{C}$
2650–2850 (2700)	69000–88000 (70000)	0.3–0.35 (0.3)	16–24 (23)	475–770

Alloy	Condition	Product form available	Proof strength: N/mm ²	Ultimate strength: N/mm ²	HAZ strength factor
5083	O	SH, ST, PL	125	275	1.00
5083	H12	SH, ST, PL	250	305	0.90
5754	O	SH, ST, PL	80	190	1.00
6060	T66	ET, EP, ER/B	150	195	0.56
6061	T6	SH, ST, PL	240	290	0.60
6063	T6	ET, EP, ER/B	160	195	0.56
6082	T6	SH, ST, PL	260	310	0.60
7020	T6	SH, ST, PL	280	350	0.80
8011A	H16	SH, ST, PL	130	145	0.59

DT, drawn tube; EP, extruded profile; ER/B, extruded rod or bar; ET, extruded tube; HAZ, heat-affected zone; PL, plate; SH, sheet; ST, strip

Table 12.2 Typical properties for aluminium alloys; values in brackets are the values used in Eurocode 9

weld strength. They are frequently used for military applications, such as bridges and armoured vehicle hulls, and also for mechanical applications, such as cranes, but less so for normal structural applications.

The range of alloys, tempers and structural forms that are available may be confusing to the first-time user of aluminium. However, the very great versatility of alloys allows for highly efficient structures. A new designer in aluminium would benefit from taking time to become familiar with the available alloys and with the processes, delivery lead times and availability of sections, rather than simply choosing a standard section from a table. It can often be helpful and reduce cost to review the possibility of having extrusions made to order if the tonnage is large enough. Using extruded profiles is better than using welded plate girders as optimal cross-section profiles can be designed; this reduces welding, possible welding strength loss, fabrication costs, fatigue susceptibility, and so on. It also facilitates positioning of welded connections in less-stressed regions of the structure, for example, closer to the neutral axis of bending.

Aluminium alloy plates and sheets are usually available from stock, particularly for the common alloys 5083, 5754 and 6082. Simple rectangular and circular hollow sections and equal angles are also available in moderate quantities from stock for relatively small section sizes suitable for small footbridges, gantries and similar structures.

In aluminium design, it is common for design engineers to create their own individual extrusion profiles. These can be complex extrusions or larger sizes and can be specifically ordered. These designs and orders are not as daunting as might be anticipated by designers familiar with lead times of other materials. The dies used to create the extrusion shape can be changed quickly and quantities as small as 1 t can be ordered. This enables engineers to design their own extrusion sections for particular applications. There are also individual extrusions, developed by several suppliers, that are suitable for forming orthotropic deck panels and structural sections.

Aluminium castings are also available and are generally manufactured from different alloys to the wrought alloys used for plates and extrusions. The design of castings is highly specialised and is not covered in detail in current structural design codes. Castings are used, if appropriate, for joints but rigorous joint testing regimes must be established.

Design and details

Aluminium is a metal that can be cut, welded, formed, bolted and riveted using similar equipment to that used for steel. Standard EN 1999 is written to imitate and be a mirror image of the steel design code, EN 1993, with similar but more intricate design calculations are required. The aluminium code EN 1999 often refers to the steel code EN 1993; much care is needed to remember that the requirements for designing in aluminium are different from those for designing in steel.

Aluminium does not behave like steel and the differences must be recognised and exploited to give an efficient and

appropriately designed structure. The most significant difference that affects aluminium structural design is the smaller Young modulus, which increases deflections. Also, the lower mass of aluminium increases the ratio of live load to dead load and requires additional considerations concerning fatigue life. The effect of welding on the parent metal strength and the welding's effect on the strength, fatigue life and buckling characteristics of the structure need to be considered.

For aluminium, the Young modulus used in EN 1999 is taken as $70 \times 10^3 \text{ N/mm}^2$, which is approximately one-third that of steel. The implication of this value is that aluminium deflections will be three times larger than for the same sized steel section, unless the aluminium section modulus is increased by a corresponding factor. This is most easily and efficiently accomplished by increasing the section depth. However, increasing the section depth affects the column, torsional and local buckling strengths, as nearly all forms of allowable buckling stress are expressed by an equation containing the terms $\pi^2 E / \lambda^2$. A designer who simply emulates a conventional steel design using I-beams and channels (in bending or in compression) will soon find that allowable stresses are predominantly low. Here, the designer must be imaginative in developing alternative section shapes and sizes that permit greater allowable stresses. Hollow aluminium sections are particularly advantageous in aluminium bridge structures as they are much less susceptible to torsional and lateral torsional modes of buckling. However, as with all closed-form sections, the facilitation of through life inspection should be considered. Local buckling of aluminium member elements must also be considered. This is an area of design that the majority of steel or concrete designers would not normally consider, as steel and concrete design codes impose shape and size limitations that guarantee avoiding such failure modes. Provided that this buckling aspect is considered in aluminium design, it will not constitute a problem. It should be noted that aluminium open sections are very often extruded with bulbs or returns at the outer edges of the sections, specifically to improve compressive performance of the edges (see Figure 12.3). The versatility of the extrusion

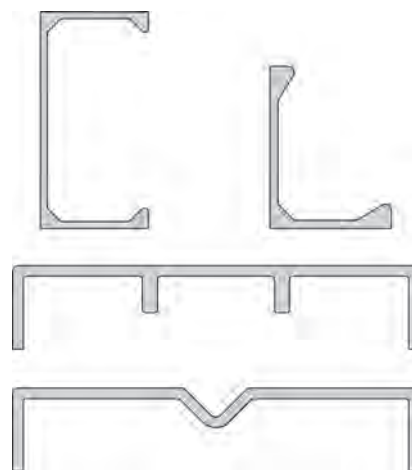


Figure 12.3 Typical bulbs and stiffeners

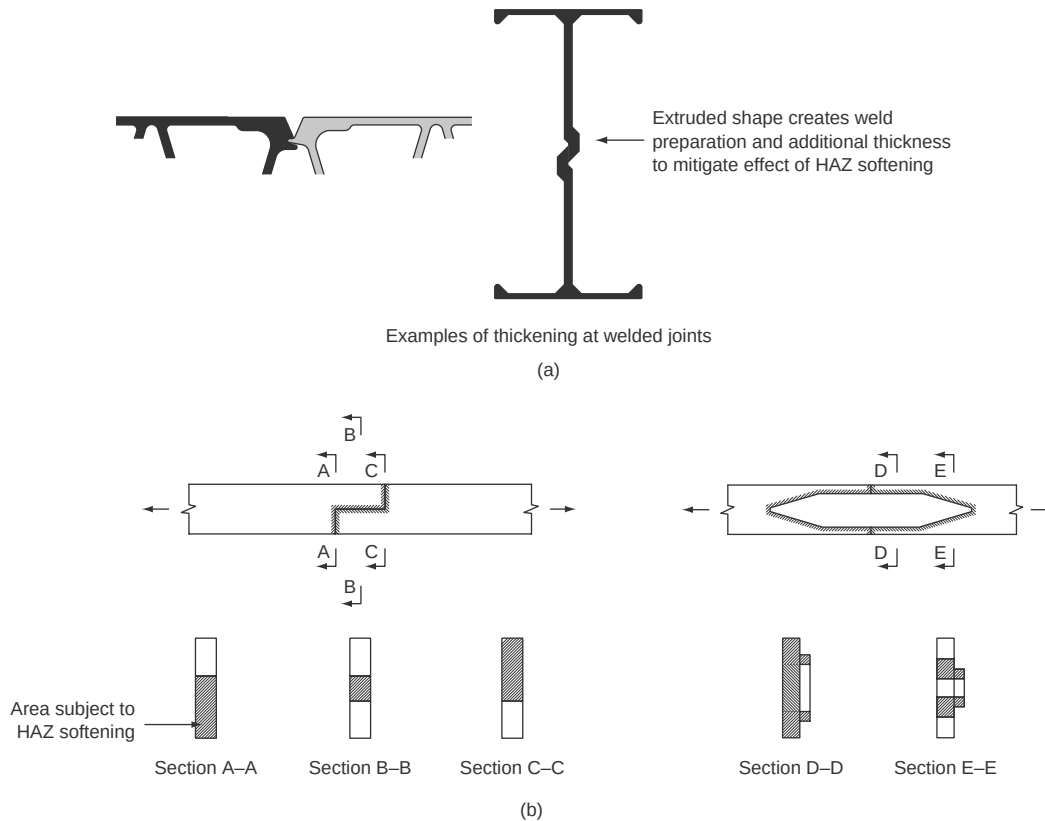


Figure 12.4 Examples of: (a) welded joints (thickening and doublers); (b) staggered and reinforced butt joints to mitigate the effects of HAZ softening

process allows for local thickening ribs to be built into sections, rather than having to increase element thickness or to add ribs.

The majority of structural aluminium alloys in use derive some of their strength from work hardening or heat treatment. This enhancement to the basic strength of the alloy can often be nullified in the welding process, such that the strength of welds and the immediate surrounding material, the heat-affected zone (HAZ), can be reduced by up to 50% of the original value. In steel, there is no significant difference in strength at or around welds; consequently, steel designers often place welds for the welder's convenience, or to suit material sizes. With aluminium alloys, greater thought and planning is required to provide an efficient welded structure. This is also a strong reason for considering bolted or riveted joints for many applications. The extent of the HAZ around a weld, where there is a loss of strength, is governed by the welding process and material thickness. The reduction of the strength in the HAZ varies with both the alloy and the level of heat treatment, or work hardening, applied in producing the alloy.

Current aluminium design codes give comprehensive rules for the extent of the HAZ and how to consider the loss of strength in member capacity. In general, the HAZ effects will usually extend 20–40 mm from any weld, depending on material thickness and weld type. For longitudinal welds, the loss of strength is usually not significant and can often be ameliorated by local material thickening incorporated in extruded profiles.

Transverse welds, however, are of greater significance; careful thought must be given to the weld location, where design details can make large differences, for example, the placement of welded connections in low stress areas rather than in high moment or stress areas, or the use of doublers with longitudinal welds, to supplement transverse welds.

The examples of stiffeners shown in Figure 12.3 and welded joints shown in Figure 12.4 illustrate many of the shapes that can be readily incorporated into extrusions. Other features can be easily accommodated, for example, aids to fit-up, weld preparations, slots for bolt heads and ribs for attachment of other components, incorporating joints, and so on (see Figure 12.5). With careful planning and the correct mindset, the possibilities of jointing and extrusions are immense. These types of feature will lead to significant savings in fabrication costs and time. The ease with which extrusions can be produced makes these types of feature realisable, even for one-off projects. The limits are only governed by the client's and designer's thinking and mindset! The concept of placing material where it can be most efficiently used, rather than simply selecting a section from a catalogue, has to be thought through and practised. This can be enormously enjoyable and is design creativity in its purest form. Figure 12.5 gives a few examples of extrusion features.

Connection design is critical to the efficiency and integrity of any structure and particularly so in aluminium. Mechanical

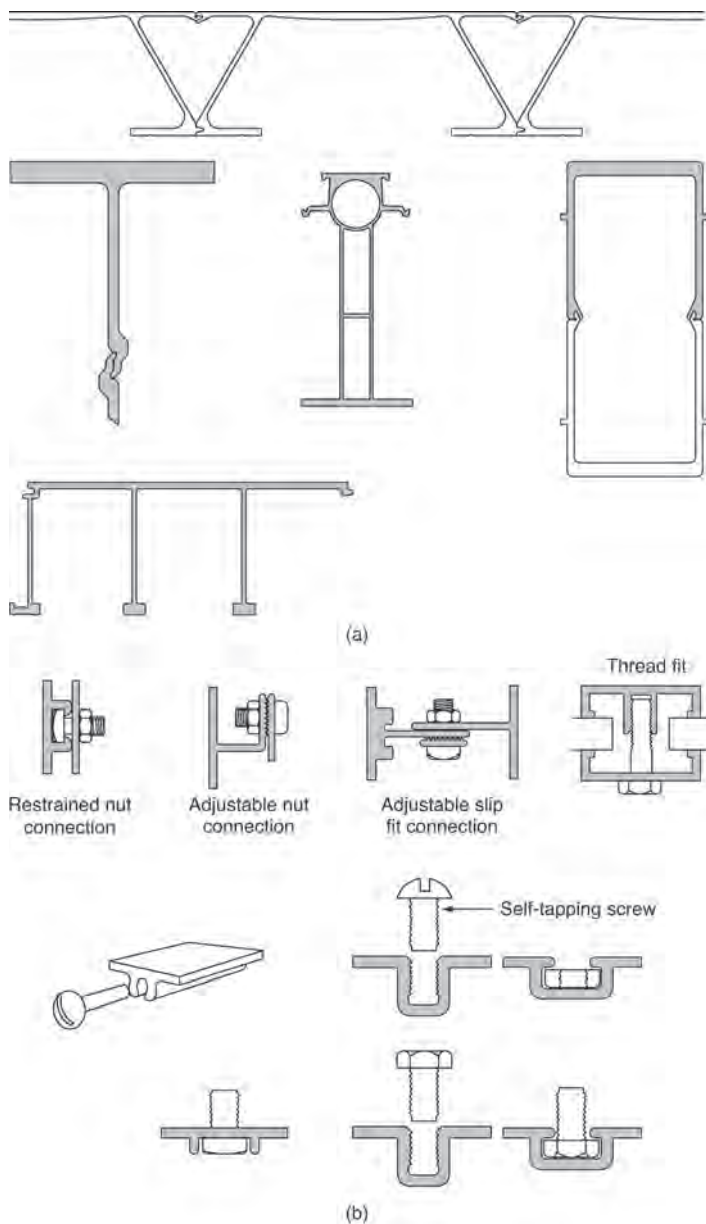


Figure 12.5 Examples of extrusion features

connections are more frequently used in aluminium structures than in steel; this is partly on account of the reduced strength in the HAZ that some alloys exhibit when welded. It is also a reflection of the ease with which the material can be formed and drilled to suit mechanical connections. Rivets, bolts and pins are all commonly used where welding may be inappropriate, but welding may be appropriate in many locations. It should be noted that the selection of the welding process, joint detailing and appropriate filler wire need the input of a skilled and accredited welding engineer as welding parameters affect the structural design. An excellent liaison between the client, designer, welding engineer and fabricator is required. This liaison is even more critical for FSW, where joint design is critical.

Many design codes and other publications give comprehensive details concerning the analysis and usage of aluminium alloys. Some of these publications are listed at the end of this chapter. Certain of the books listed are quite old. A number of publications are produced in the USA and may not use SI units, plus they may often refer to alternative design standards. A few are quite theoretical and probably beyond the immediate requirement for straightforward aluminium bridge design; however, the key aspects of material properties and how best to exploit them are fundamental to the efficient use of aluminium and each reference will add to a bridge designer's understanding. New publications, codes and updated manufacturer's web pages are constantly being added.

Design standards

In 1950, the Institution of Structural Engineers produced the first recognised UK design code for the structural use of aluminium. The British Standard Code of Practice CP 118 was issued in 1969 and was updated and converted to metric units in 1973. For quick and simple assessment of member sizes for static applications, CP 118 remains a useful tool, using permissible stress rules similar to those used in BS 449 (BSI, 1969a).

The relevant UK standard, BS 8118, was published in 1991 and was based on limit state principles and allowed higher loads in some elements than did CP 118. Standard BS 8118 permits notable increases in allowable loads for welded joints. Rules for fatigue design changed considerably and those included in CP 118 are no longer considered valid. Standard BS 8118 has two parts: Part 1 considers design matters, and Part 2 gives specifications and requirements for fabrication.

The relevant European code is Eurocode 9; BS EN 1999: Parts 1-1 to 1-5 were first published in 2007 and have been updated through annexes. It is necessary to ensure that the designer is using the latest edition of EN 1999 and its annexes. Additionally, there are national annexes to each of the five parts of EN 1999, which allows country-specific factors and choices. The national annex contains all nationally determined parameters to be used for the design of aluminium structures constructed in the relevant country. In the UK, the UK national annexes are a prerequisite for use of EN 1999 for building control approval in the UK. The majority of BS 8118 has been superseded by EN 1999. Standard EN 1999, in common with other Eurocodes, makes considerable cross-reference to other European standards. In particular, EN 1999 needs to be read in conjunction with EN 1990, *Basis of structural design*, for general principles (BSI, 2002a), EN 1991, *Action on structures*, for loading (BSI, 2002b), and EN 1090-3, *Technical rules for the execution of aluminium structures*, for execution (fabrication and erection) rules (BSI, 2019). Many of the EN 1999 rules are similar to those in BS 8118, although they are more extensive and allow greater design refinement. Comparisons between EN 1999 and BS 8118 have shown little difference in allowable loads for the static design of typical members and details. However, the BSI committee responsible for the UK inputs to EN 1999 considered that the fatigue rules given in EN

1999-1-3, *Structures susceptible to fatigue* (BSI, 2007b), were potentially unsafe in some instances and prone to misinterpretation for some details. The national annex for EN 1999-1-3 refers to published documents PD 6702-1, *Recommendations for the design of aluminium structures to BS EN 1999* (BSI, 2009c), and PD 6705-3, *Recommendations for the execution of aluminium structures to BS EN 1090-3* (BSI, 2009d). Standard EN 1999 includes several parts that had no previous UK equivalent: Part 1-2, *Structural fire design*, which covers design for fire resistance and Parts 1-4, *Cold-formed structural sheeting*, and 1-5, *Shell structures*. Standard EN 1999 Parts, 1-2, 1-4 and 1-5 will have limited relevance to the bridge designer. It is anticipated that, in the early part of the 2020s, an update to EN 1999 will be published. Currently, the Canadian Highways Bridge Code (CSE, 2016) and the code for welding aluminium structures (CSE, 2015) are undergoing major reforms. In the USA, Canada and Europe, the aluminium design codes are being reorganised to resemble as closely as possible their associated steel design codes, which will help designers appreciate the different requirements of aluminium design.

The analysis of aluminium bridges should be based on the limit state being considered. First-order analysis, which considers only the initial geometry of the structure, is usually the appropriate method of analysis for determining internal forces and moments. Second-order analysis, which considers the deformed state of the structure, may also be suitable in some cases where deformation is significant, particularly in military bridge structures, which are optimised for minimum weight and therefore flexible. Elastic global analysis can be used in all cases, assuming that the stress–strain behaviour of the material is linear. For a very detailed analysis, EN 1999 has available several stress–strain options. Plastic global analysis can only be used in certain circumstances. Aluminium bridges should be designed and detailed to allow for inspections and maintenance. A fatigue assessment should be conducted for all critical areas, in accordance with EN 1999-1-3 (BSI, 2007b). Weld improvement techniques can be used to increase the fatigue life of connections. Deflections and resonance of the bridge often govern the design of the structural elements and need to be considered. An aluminium I-section can achieve the same flexural stiffness as a steel I-section if all dimensions of the steel I-section, except the flange width, are increased by approximately 1.4 and will also reduce weight by approximately 50%. The designer may then find that the highest-strength aluminium alloys are not required.

The design service life of permanent and temporary bridges should be clearly stated; for reusable structural elements, the types of record of actual use should also be stated. The bridge should be designed so that access is achievable for all in-service maintenance and inspections. Further, for pedestrian bridges, the designer must consider that service vehicles may use the bridge. Usually, welded connections are avoided where an element could be subject to impact load. A major consideration is to ensure a reliable load model is used, which is especially essential for aluminium pedestrian bridges. Aluminium has a thermal expansion coefficient approximately double that

of concrete or steel. This means having expansion and contraction joints that can accommodate the appropriate ambient temperature changes.

There is also the possibility that special design conditions may apply; for example, the fatigue assessment of railway bridges and other relevant authorities may need to be consulted.

Fabrication

Structures in aluminium and its alloys can be easily shaped using the usual industrial processes, including, casting, extrusion, forging and rolling, similar to those used for steel fabrication. The aluminium fabrication workshop must be completely separated from any other fabrication workshop. Aluminium components are, however, cleaner, lighter and easier to handle. Appropriate design considerations and the incorporation of suitable features can simplify many operations and give efficiencies.

Aluminium sections and plates are cut using the same types of tool as used for steel, for example, machining, notching, routing, sawing and shearing. The sawing of aluminium is much faster than for steel but uses a different size and shape of teeth and different cutting speeds. Flame cutting is not appropriate, as it alters the mechanical properties of the alloy and produces an uneven edge. To cut shaped pieces, the high-pressure water jet process is particularly suitable and gives a clean edge that can generally be left without the need for further machining or treatment.

Equipment for the bending and forming of aluminium is as used for steel. However, spring back for aluminium is greater than for steel. Aluminium components can be readily shaped, bent and formed, while press brake forming is ideal for creating angles and corrugated shapes from sheet and thin plate. However, consideration should be given to the bend radius residual stress, increasing the potential resulting susceptibility to stress corrosion cracking, particularly for the 7xxx series alloys.

The common structural aluminium alloys are readily welded using either the gas-shielded metal inert gas (MIG) or the tungsten inert gas (TIG) processes. The techniques are similar to the welding of steel but there are differences; interestingly, it is often easier to train a new aluminium welder than it is for an experienced steel welder to convert to aluminium. Care needs to be taken to remove the aluminium oxide film and any contaminants from the fusion faces before welding. It is also important that the welding is carried out in a dry and draught-free environment to ensure the integrity of the gas shielding to the molten weld pool and prevent contamination. Care is also needed, as aluminium does not change colour when heated and there is a strong reaction if molten aluminium comes into contact with water. Recent developments in the patented FSW process are showing significant advantages over the gas-shielded processes, particularly for straightforward, long seam welds, such as would be used in joining extrusions to make orthotropic decks or deep girders. The process has been used extensively in railway carriage fabrication, giving high-quality welds and having less detrimental effects on material properties than from

conventional aluminium welding, owing in part to the absence of a filler material.

Connections made by riveting or bolting are straightforward. Generally, the difference between bolt size and hole size is less than for steel; typically, holes are normally 1 mm larger than the bolts. Friction grip bolted connections are frequently used for bridging applications. Bolted connections should preferably be made with aluminium or stainless steel bolts. Steel bolts with a protective layer can be used, but have a limited corrosion protection life.

In commercial applications, many alloys are successfully welded using laser or electron beam processes. The advantages of this type of welding are higher welding speed, increased productivity, increased dimensional control and less distortion. The number of welding imperfections decreases and no consumables are required. Friction stir welding is also available, which requires no melting of the parent metal and negates many of the usual problems of the fusion welding of certain aluminium alloys. The potential advantages of these processes should be explored at the concept design stage. It is suggested that companies with the relevant expertise and equipment be consulted at an early design stage. Friction stir welding has been used together with 6061-T6 extrusions for pedestrian footbridge decks.

Explosion bonding has been successfully employed to join aluminium alloys to steel plates. The resulting composite can be welded to an aluminium structure on one face and a steel structure on the other face. This technique has been used successfully in the offshore industry to join aluminium modules and topsides to steel base structures.

Developments in the aerospace and automotive industries include the use of high-precision castings for connections and nodes, with extruded members between the castings. Such developments will doubtless soon progress to aluminium bridge structures.

Fatigue

Aluminium structures subjected to fluctuating service loads can fail by fatigue, in a similar manner to steel structures. The light weight of aluminium, with its low inherent damping, can result in resonance, which needs to be avoided. This is often achieved in one of two ways: by avoiding natural frequencies and harmonics of the structure that might coincide with the normal loading frequency range, or by restricting the predicted vertical and lateral vibrational acceleration to be within specific acceleration levels. The methodology and principles given for fatigue calculations are familiar to most bridge designers. Fatigue is more likely in aluminium structures than in steel structures, as the ratio of live load to dead load is some three times higher for aluminium structures. Vibration is considered as a possible fatigue issue and appropriate design considerations need to be taken to reduce vibrations and resonance. For example, it can happen that dynamic load design may predominate over static load design and damping devices may be necessary.

Fatigue failure usually initiates at a point of high stress concentration, often associated with abrupt changes in geometry or at welds. Careful attention to detailing and locating welds in low stress areas, using bolted connections, FSW or reducing welds by using extrusions can give significant increases in fatigue life. For bridge design, it is recommended that bridge structures are designed on a 'safe-life' basis, that is, the members are proportioned such that the predicted cyclic stress levels do not result in any fatigue cracks.

The BSI committee that mirrors the Eurocode committee noted that the EN 1999-1-3 (BSI, 2007b) uses a similar methodology to BS 8118 (BSI, 1992a) for calculating fatigue 'safe life'. However, the BSI committee deliberated that some of the detail categories in EN 1999-1-3 could be subject to misinterpretation, or possibly give fatigue lives that might only be achieved with unrealistic expectations of minimal internal defects. The UK EN 1999-1-3 National Annex (BSI, 2008), therefore, gives references to different detail categories (PD 6702-1 (BSI, 2009c), PD 6705-3 (BSI, 2009d)) from those in the informative annex of EN 1999-1-3. Standard EN 1999-1-3 also allows a damage-tolerant approach to fatigue design, that is, allowing some cracking to occur in service, provided that there is a stable, predictable crack growth and an inspection regime in place, aligned with crack growth rate, to avoid reaching a critical crack condition in service. In addition, material supply must include fracture toughness certification. The damage-tolerant approach should only be used in conjunction with the agreement of the owner of the structure. In general, damage-tolerant design is not likely to be suitable for bridges when in service, although this design principle is used extensively in military bridging, where use monitoring and inspection are more readily implemented. Design assisted by testing is also available.

Standard EN 1999-1-3 is used with EN 1090-3, *Technical requirements for the execution of aluminium structures* (BSI, 2019), as it contains the requirements necessary for the design assumptions to be met during execution of components and structures. In general terms, the 'allowable' fatigue stress is independent of the alloy being used. However, the environment can influence fatigue life and EN 1999 recommends small reductions in fatigue strength for certain alloys when used in a marine environment.

Fire safety

The theory of the fire safety of aluminium alloys structures is governed by the same principles and methods as those used for steel structures. However, most aluminium alloys have reduced strength at temperatures above 100°C, and will have lost a significant proportion of their strength by 350°C. The upper practical maximum working temperature limits range from 200°C to 250°C. However, aluminium alloys do not lose ductility or become brittle at low temperatures; this is a distinct advantage in cold climates.

Contrary to popular opinion, aluminium does not support combustion and does not burn. Aluminium's high coefficient

of thermal conductivity, which is approximately four times that of steel, is advantageous in situations with local fires, in that the heat is rapidly conducted away from the fire location, reducing temperature hot spots. Further aluminium's specific heat is twice that of steel and consequently aluminium needs a greater heat input, compared with steel. Applications that need a specific fire resistance should have insulation applied in a similar manner to steel structures. Standard EN 1999-1-2 (BSI, 2007a) gives comprehensive rules for determining the fire resistance of aluminium structures, to ensure that there is adequate load-bearing resistance and to limit fire spread.

Historic and recent bridges

Highway bridges

One of the first recorded uses of aluminium for bridgework was for the reconstruction of the bridge deck of Smithfield Street Bridge in Pittsburgh in 1933 using a 2014-T6 alloy and later both 5456 and 6016-T6 alloys. The saving in weight allowed a significant increase in bridge capacity and was a good example of the weight advantage of the material. The bridge continued in service with an aluminium deck until the bridge's complete renovation in 1994.

The Massena Bridge built in New York in 1946 incorporated a 30 m span formed of two aluminium alloy riveted plate girders, in 2014-T6 and 2117 alloys. Four years later, in 1950, the construction of the Arvida Bridge in Canada (see Figure 12.1) was completed. With an arch span of 91.5 m and total length of 153 m, this was the first example of an all-aluminium civilian bridge and is still one of the largest. It is probably fair to say that these bridges were rather experimental in nature. They were fabricated from a high-strength alloy containing copper (2014-T6 and 2117 alloys), which would not be recommended for structural use today, as the corrosion characteristics of this alloy are significantly inferior to other available alloys. Despite that, the Arvida Bridge is still in service and maintenance work has been relatively low.

Two opening bascule bridges were built of aluminium in

the UK, the first being the Hendon Dock Junction Bridge in Sunderland, in 1948 (Figure 12.6), and the second in Victoria Dock, Aberdeen, which opened in 1953. These bridges used a 2014-T6 alloy for the deck plate and a 6151-T6 alloy for the riveted truss girders. Additional aluminium bascule bridges have been built in Banbury in 1959 and in Gloucester in 1962 using a 6351-T6 alloy. The weight saving led to economy in the bearings, machinery and counterweights and it seems surprising that aluminium has not been used more widely in opening bridges.

Germany's first aluminium road bridge, the 44.2 m span Schwansbell Bridge, was opened in 1956. The design was a trussed structure made from extruded profiles and sheets using a 6351-T6 alloy. A comparable steel bridge would have been 240% heavier than the aluminium bridge. The aluminium was given no additional corrosion protection and later inspections showed that the bridge was still in an excellent condition, despite being exposed to an industrial atmosphere.

The increases in steel prices during the 1950s resulted in aluminium bridge systems and extrusions being developed in the USA, including the Fairchild system, which used the 5083 alloy and the Baron system. These systems used girders, formed from aluminium sheeting, acting compositely with concrete decks. A number of these were built, as were several bridges using riveted aluminium plate girders acting compositely with concrete decks. These bridges, unfortunately, tended to follow steel techniques and fell from favour as uneconomic when aluminium prices started increasing in the late 1960s. These aluminium bridges have continued to give good service.

The development in Europe of large extrusions from 6xxx alloys in the 1960s extended the potential application of aluminium in bridges but, unfortunately, coincided with the regression in use of aluminium through price increases. A fall in price in the 1990s brought renewed interest and development of extrusion systems. A range of box girders made by welding aluminium extrusions together, complemented by orthotropic decks formed from multivoiced hollow extrusions were then available. Several bridges have been built in Europe between 1998 and 2008 using this technology. Specific examples include the Forsmo Bridge in Norway, a 39 m long bridge to replace a badly deteriorated steel and concrete structure. The old bridge deck was demolished and the new bridge was lifted in place and opened to traffic in a 5 day period. The aluminium alloys used were 5083 plate and 6xxx alloys for the extrusion. Several opening bridges have been built in the Netherlands, including the Helmond bridge in 1999 and the Riekerhavenburg bridge in 2003, made entirely of aluminium and using extruded trapezoidal profiles for the deck (Figure 12.7). The stiff lightweight deck allows overall economy to the structure's weight, which is particularly beneficial for opening structures and for erection considerations. Common alloys used before 1980 included 6082, 6061-T6, 6351-T6, 5083-H113, 5456-H321, 5052, 6151-T6 and 2014-T6, 2117-T4 (for rivets). The alloys given in this section are believed to be correct and the best available information at the time of writing. The information is taken from Das and Kaufman (2007) and Siwowski (2006).



Figure 12.6 Hendon Dock Junction Bridge in Sunderland (courtesy of the Stafford Linsley collection)



Figure 12.7 (a) The Riekerhaven Bridge; (b) the Westerdoksluis Bridge (courtesy of Bayards Aluminium Constructions BV, the Netherlands)

Footbridges

While relatively few highway bridges have been built from aluminium, the use of aluminium is more widespread in long-span footbridges and walkways, especially in corrosive environments, (e.g. industrial, port and marine areas), where high corrosion resistance is required. There is also the use of link-spans for marine applications (see Figures 12.8 and 12.9). Aluminium pedestrian bridges are also used to widen or add footpaths and cycleways to existing bridges. The low self-weight and excellent corrosion resistance of aluminium are particularly beneficial for these applications. Early examples include the 42m welded plate girder Letten Bridge in Sweden, the 52m span arched truss footbridge at Pitlochry in Scotland, built in 1948, and the pedestrian bridge at Rogerstone, south Wales, built in 1957, which uses the 6351 alloy. The Lockmeadow footbridge in Maidstone opened in 1999 and uses an ingenious arrangement of interlocking extrusions that are held together by stressing bars. Modular aluminium footbridges, using extruded sections and, in some cases, cast aluminium joints are now readily available.

Military bridges

Aluminium is particularly suitable for military bridges, where the necessity for portability and speed of erection requires materials with a high strength-to-weight ratio. Most military bridges built since 1960 have been produced from aluminium. Early examples were made using the same high-strength copper-based alloy as the Arvida Bridge in Canada but, since the 1970s, materials of choice have been from the 7xxx alloys, often the 7020 alloy. The 7xxx specialist alloys are similar to those generally available for civilian use but can have tensile strengths in excess of 500 N/mm². Some alloys have been specifically developed to minimise stress corrosion cracking while maintaining good fracture toughness and relatively high HAZ strengths. The fabrications are usually manufactured using extrusions and plates welded together, although some examples make extensive use of riveting, thereby avoiding HAZ softening.

For example, Figure 12.10 shows high-strength weldable aluminium alloy bridge panels designed for use with a modular military bridge. The panels are an 8 m ramp panel together with



Figure 12.8 Typical marine link bridge (photo courtesy of P Tindall)



Figure 12.9 Stokkenbrug (courtesy of Bayards Aluminium Constructions BV, the Netherlands)



Figure 12.11 56t Chieftain tank crossing a 56m link reinforced aluminium alloy military bridge



Figure 12.12 Loaded heavy equipment transporter crossing a 46m two-span aluminium alloy military bridge supported at midspan by a floating aluminium alloy pontoon pier

2 m, 4 m and 8 m panels. Figure 12.11 shows a 56t Chieftain tank traversing a 56m link reinforced aluminium alloy military bridge, while Figure 12.12 shows a loaded heavy equipment transporter crossing a 46m two-span aluminium alloy military bridge supported at midspan by a floating aluminium alloy pontoon pier. These three figures show the ingenuity and versatility of military bridge designers in solving intricate bridge problems. From this, it can be concluded that the considerable research that has been carried out in the military aluminium bridge design, construction and erection areas could be transferred with considerable effect to the civilian arena.



Figure 12.10 High-strength weldable aluminium alloy bridge panels for a modular military bridge (8m ramp panel and 2m, 4m and 8m panels)

typical concrete deck weight. The aluminium decks have good corrosion resistance, which, allied to the low weight, makes them ideal for bridge upgrades, opening bridges and long-span bridges. Many of the Swedish bridge decks have been surfaced with a 6mm thick acrylic surfacing system to minimise total weight. Others have a 40mm thick layer of a mastic asphalt type of surfacing. The close tolerances inherent in the extruded decks allow for good ride quality, even with these thin surfacing systems.

Bridge decks and furniture

Several manufacturers have developed large multivoided hollow extrusions specifically for forming orthotropic bridge decks. These have been used extensively in Sweden for bridge rehabilitation and many bridges have had deteriorated concrete decks replaced by the bridge deck systems. Similar systems developed in other countries, including the USA, Germany and Norway, have been used for the decks of new or replacement bridges. The members made using extrusions are typically formed from alloy 6063-T6 (See Figure 12.13). The weight of these deck systems is often only about one-tenth of that of a

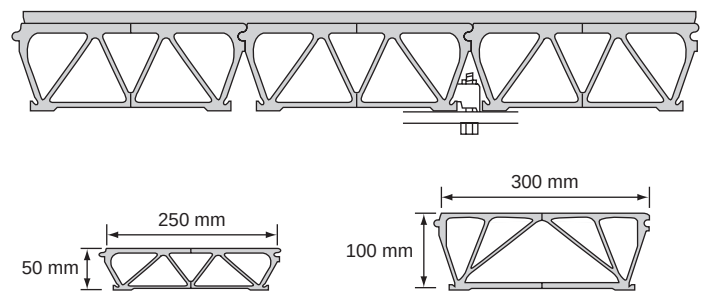


Figure 12.13 Example profiles designed for 1.2m and 3m support girder spacing

Aluminium bridge parapets have been used for many years, generally with no paint system or other surface treatment. The good corrosion resistance of aluminium is highlighted by its excellent performance, despite being subject to road pollutants, salt spray, and so on. Several manufacturers have had their systems successfully tested to meet the requirements of EN 1317 (BSI, 2010). Aluminium is also appropriate for other bridge furniture, including sign gantries, access walkways and maintenance gantries.

Corrosion behaviour

Standard EN 1999-1-1 (BSI, 2014) gives aluminium alloys durability ratings, A, B and C, in descending order of durability. These ratings define the need and degree of required protection; further protection details are given in Annex D of EN 1999-1-1. Under normal atmospheric conditions, the alloys listed in EN 1999-1-1 do not require protective treatment.

If protective treatment is required, it should be described in the project specification, which will consider the corrosion mechanism, exposure condition, material thickness and alloy durability. The treatments range from 'no protection' and 'protection normally required except in special cases' to 'protection that depends on the special conditions of the structure'. Details of the corrosion protection procedures are given in EN 1090-3 (BSI, 2019) and EN 508-2 (BSI, 2009a). Critical areas, such as HAZ regions or regions that are susceptible to pitting or stress corrosion, are often protected using metal spray (aluminium or aluminium–zinc mix), either by hot plasma application or, now being developed, cold spray supersonic powder deposition methods. In terms of paint-based protection, the industry is currently researching new systems to replace the strontium chromate epoxy primer processes used as a frontline defence to corrosion of these susceptible areas prior to its use being banned by the European Chemical Agency, owing to toxicity issues.

The aluminium alloys in general structural use have excellent corrosion resistance; this is attributable to the self-sealing protective oxide film that forms naturally on aluminium when it is exposed to oxygen. The film is usually invisible, relatively inert and adheres strongly to the aluminium surface. Once formed, it prevents further oxidation and, if damaged, reforms naturally. It is thus self-healing and stable within a pH range of about 3 to 9.

The 5xxx and 6xxx series alloys will develop small pits when exposed to industrial pollution. A layer of corrosion product, which inhibits further corrosion, seals the pits and results in a dull grey weathered appearance for untreated aluminium. The 7xxx series alloys exhibit a layered form of corrosion (exfoliation) rather than pitting and have slightly lower durability than the 5xxx and 6xxx alloys. However, pitting corrosion can occur in the HAZ, particularly in the 7xxx alloys, where the alloy elements on the surface with different electrochemical potentials cause a galvanic cell to initiate in the presence of water unless a protective barrier is present, such as a metal spray or an appropriate paint system.

Bimetallic corrosion is often perceived as a problem, although in reality it is only of concern in certain circumstances. The oxide film on the aluminium alloy acts as a natural barrier; it is only if this barrier is broken down from acidic or alkaline moisture, or through abrasion, that the bimetallic cell will activate. Thus, corrosion between stainless steel and aluminium, which have a large potential difference, is non-existent in most environments as the contact is between two inert respective oxide layers. Stainless steel fittings are used on aluminium masts and superstructures of yachts with no separating layers and give many years of excellent service. Connections between dissimilar metals that are subject to fretting or relative movement can sometimes corrode rapidly, as the protective oxide film is repeatedly worn away. Insulation between the dissimilar metals with an inert non-absorbent barrier can prevent reaction. In some cases, sacrificial anodes, such as zinc plates, with higher positive electrochemical potential than the other two dissimilar metals, are attached to the structure to prevent galvanic corrosion of the structure.

Areas that act as moisture traps are susceptible to deterioration in most materials and aluminium is no exception. Good detailing should avoid areas that may pond or hold moisture. Contact with cement and certain wood preservatives can cause corrosion of the aluminium. Foundation base plates and interfaces with concrete decks, abutments, and so on, should therefore be protected by painting the aluminium contact surface. Annex D of EN 1999-1-1 (BSI, 2014) gives guidance on circumstances when protection measures are appropriate.

Coatings and finishes

A variety of finishes and coatings are available for use on aluminium. While not generally necessary for corrosion resistance of the whole structure, these may be applied for aesthetic purposes or to provide protection to specific areas in contact with other materials. Typical finishes include anodising, painting, baked organic coatings and mechanical treatment.

Anodising is a process that thickens the layer of natural oxide protecting the aluminium. The anodised surface is extremely hard, provides good abrasion resistance and maintains its appearance for many years. It also improves the adhesion of paint primers. Anodised finishes are used extensively for architectural applications, such as glazing systems, and for domestic products. Different anodising solutions and dies can give a variety of colours and surface hardness characteristics. The main restriction on anodising is that it is limited to pieces that will fit into the treatment tanks at the anodiser's facility. Generally, this will give a limitation of approximately 10m on length and 2m on width or depth. The finished colour of an anodised piece will vary with alloy composition. If a uniform appearance is required, it is necessary to use the same alloy throughout and to select a welding wire that behaves similarly. (A variety of welding wires can be used on some alloys and some will anodise to a darker colour than the parent metal.) However, processes that modify the surface layer of the material, as opposed to coating it, may reduce fatigue life; this

should be considered for structures under cyclic loading, as cracking of the potentially less ductile modified surface could be initiated by overloads,

If components are to be assembled and need to be protected or prepainted, extra sealing of the contact surfaces will be defined in the project specification, considering the expected life of the structure, the exposure and the required protection quality of the components.

The majority of standard decorative paint topcoats can be used on aluminium, provided that suitable preparatory work has been carried out. The preparation essential to ensure good adhesion of the paint involves an initial degreasing and then either etching with an acid solution or jet cleaning with an abrasive, such as corundum. Steel- or copper-based abrasive must not be used. All acid must be washed off with clean water prior to subsequent treatment. The initial etch or jet cleaning is followed by the application of a thin wash primer, a vinyl-based resin containing a small amount of phosphoric acid. This primer provides the key for standard paint systems to adhere to the aluminium. Areas that will be in contact with concrete or treated timber should be coated with a bituminous paint or a coating providing the same protection after the pretreatment and primer coating. In industrial or marine environments, at least two coats of plasticised coal tar pitch or heavy-duty bituminous paint must be used. Paints containing lead, copper or tin must be avoided, as they are detrimental to aluminium. Fortunately, these compounds are not now common in modern paints.

Baked organic or powder coatings are in widespread use for architectural applications. These give a thick, hard finish with excellent weathering properties and are in a wider range of colours than those available for the anodising process. There are a range of proprietary systems. The systems are usually applied by spraying using an electrostatic process prior to oven baking. The section size is limited by the available oven size at the applicator's premises, and is likely to be similar to that for anodising. The organic coatings and anodised systems may be appropriate for pedestrian parapets and fascias on architecturally influenced bridges.

Mechanical treatment may be used to change the appearance of mill finish aluminium, such as brushing, grinding, polishing and surface blasting. Sheet and plate can have an embossed finish, such as stucco, applied by passing through textured rollers. Light blasting with corundum or beads can be used to give a uniform matt finish and avoid initial high reflectivity.

Sustainability

Bauxite is the principal ore source for new aluminium. However, aluminium is the third most abundant element in the earth's crust, occurring mostly as aluminosilicates. It is extracted by electrolysis of alumina from bauxite fused with cryolite. Extraction rates are excellent, with 4t of bauxite producing 1t of aluminium. The abundance of raw material and ease of recycling are such that resource depletion is not an issue.

Many studies have been conducted to consider the ecological impacts by applying life-cycle analysis techniques and

comparing aluminium with other materials, such as carbon steel and stainless steel, for structural components in bridges and buildings. Comparing the cumulative energy demand, aluminium structures with an anodised or painted surface have slightly less of an impact than painted or galvanised steel, once recycling is considered. If the aluminium is left with no surface treatment, the difference becomes greater. Studies have generally assumed that the structural elements of the building will not be repainted in their lifetime. Similar studies for bridge-work should therefore show an even greater benefit from using aluminium. For example, aluminium has lower maintenance costs, fewer out-of-service times, lower transportation costs to get to site and rapid installation times. Overall, it may be concluded that, on a whole-life basis, aluminium alloys may offer a more competitive solution than carbon steel for bridges and exposed areas of building structures.

The electrolytic process used in a primary smelter is energy-intensive; it is unsurprising that the majority of primary aluminium is produced using hydroelectric power. It can be shown that aluminium used in the building and construction industry has a recycling rate of between 92% to 98%. Energy consumption in the recycling process is only about 5% of that used in producing new aluminium. Other advantages accrue from savings in fabrication, transport and handling, owing to the low self-weight and from labour-saving features that can be incorporated in aluminium extrusion design.

Future trends

Research into the structural application of aluminium alloys in civil engineering structures is disappointingly sparse; most future developments are likely to arise from advances in other applications, such as the aerospace and transport industries, which have different criteria to address. The development of new alloys is one area.

Traditionally, the use of castings has not been encouraged for structural applications, as previously the quality and brittle characteristics of many castings had been unsuitable. However, the automotive and aerospace industries have developed the use of high-precision castings that have good ductility and are increasingly using these in combination with purpose-designed extrusions. Such methods are particularly appropriate for volume production. The technology is being developed for the use of castings in civil engineering, especially for jointing aluminium extrusions, where sufficient applications and quantities can justify the development and tooling costs for quantity production. The development of standardised systems is helping in this development. Further, it is necessary to develop more imaginative extruded profile, which allow the increased use of fabrication using FSW to make larger panels and other structural units for bridges.

Higher-strength aluminium alloys are being developed that have increased durability and corrosion resistance, using new manufacturing processes and materials engineering; for example, billets manufactured using compact powder aluminium, an innovative aluminium powder, mechanically alloyed

and reinforced with fibres can give new structural properties. Development of material technology continues. New processes using fibre-reinforced aluminium and powder metallurgy promise materials that are stronger and stiffer than the aluminium alloys currently in structural use. The increase in stiffness and deflection reduction will be particularly relevant.

Technology for joining materials is an area of research and development that connects different industries. Developments in FSW, electron beam welding and electron beam welding plus adhesive bonding are being added to the jointing techniques available to the civil engineering industry but are unfortunately not yet covered in any detail by current design codes. Some are, however, well established in the manufacture of railway carriages and in military bridging. Additionally, work is being carried out to develop post-weld treatments (e.g. grinding, peening) for improving fatigue performance.

Further thought shows that replacing steel or concrete bridge decks with aluminium decks allows the original piers and foundations to be reused without their replacement, increasing live load capacity, avoiding unnecessary strengthening of the substructure and foundations, and reducing inertial forces of elevated aluminium decks in an earthquake situation. Additionally, new architectural changes to the designs would bring additional aesthetics of the structures into the designer's remit.

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Chapter 13

Arch bridges

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This chapter considers the full range of arch bridge types and a range of materials, presenting several case studies and describing the design decisions that were made. A general treatment of the analysis of arches is presented, including the derivation of the basic equations that can be used to undertake hand calculations, which may be used to validate computer analysis output. Detailed arch bridge design is outside the scope of this chapter, so only general issues are discussed. Most of the chapter is devoted to masonry arch bridges. Masonry arch bridge construction is discussed in its historical context; the importance for engineers to take a holistic approach to bridge assessment and design is emphasised. There is a significant section on bridge assessment, which includes guidance on the application of current and emerging assessment methods. This is underpinned with background information regarding the material properties of masonry. The chapter concludes with a treatment of repair and maintenance strategies, including a table, listing common remedial and strengthening measures.

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Introduction

The origins of the use of arches as a structural form in buildings can be traced back to antiquity (Van Beek, 1987). In trying to arrive at a suitable definition for an arch, we may look no further than Hooke's anagram of 1675, which, when solved, reads, '*Ut pendet continuum flexile, sic stabat continuum rigidum inversum*' – 'As hangs the flexible line, so but inverted will stand the rigid arch.' This suggests that any given loading to a flexible cable, if frozen and inverted, will provide a purely compressive structure in equilibrium with the applied load. Clearly, any slight variation in the loading will result in a moment being induced in the arch. It is arriving at appropriate proportions of arch thickness to accommodate the range of eccentricities of the thrust line that is the challenge to the bridge engineer. Even in the Middle Ages, it was appreciated that masonry arches behaved essentially as gravity structures, for which geometry and proportion dictated aesthetic appeal and stability. Compressive strength could be relied on, while tensile strength could not. Based on experience, many empirical relationships between the span and arch thickness were developed and applied successfully to produce many elegant structures throughout Europe.

The expansion of the railway and canal systems led to an explosion of bridge building. Brickwork arches became increasingly popular. With the construction of the Iron Bridge in Coalbrookdale (in 1779), a new era of arch bridge construction began. By the end of the nineteenth century, cast iron, wrought iron and finally steel became increasingly popular; only to be challenged by ferrocement (reinforced concrete) at the turn of the last century.

During the nineteenth century, analytical techniques developed apace. In particular, Castigliano (1879) developed strain energy theorems that could be applied to arches, provided they remained elastic. This condition could be satisfied, provided the line of thrust lay within the middle third, thus ensuring that no tensile stresses were induced. The requirement to avoid tensile stresses only applied to masonry and cast iron; it did not apply to steel or reinforced concrete (or timber, for that matter), as these materials were capable of resisting tensile stresses.

Twentieth-century arch bridges have become increasingly sophisticated structures, combining modern materials to create exciting functional urban sculptures.

Types of arch bridge

The relevant terms used to describe the various parts of an arch bridge are shown in Figure 13.1. Arches may be grouped according to the following parameters:

- the materials of construction
- the structural articulation
- the shape of the arch.

Historically, arch bridges are associated with stone masonry. This gave way to brickwork in the nineteenth century. Because these were proportioned to minimise the possibility of tensile stress, they tend to be fairly massive structures.

By comparison, the use of reinforced concrete and modern structural steel gives the opportunity for slender, elegant arches.

Nowadays, timber is restricted to small bridges, occasionally

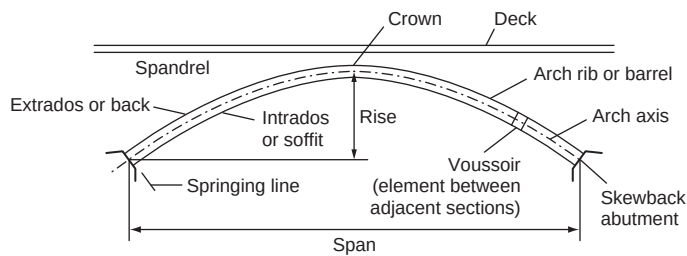


Figure 13.1 Arch bridge terms (O'Connor, 1971)

in a truss form but more usually as laminated curved arches. Although timber has a high strength-to-density ratio parallel to the grain, it is anisotropic and strength properties perpendicular to the grain are relatively weak. This requires careful detailing of connections to ensure economic use of the material.

With regard to structural articulation, the arch can be fixed or hinged. In the latter case, either one, two or three hinges can be incorporated in the arch rib. While the fixed arch has three redundancies, the introduction of each hinge reduces

the indeterminacy by one until, with three hinges, the arch is statically determinate and hence, theoretically, free of the problems of secondary stresses. Figure 13.2 shows a range of possible arrangements. The articulation of the arch is not only dependent on the number of hinges but is also fundamentally influenced by the position of the deck and the nature of the load transfer from the deck to the arch.

The traditional filled spandrel, where the vehicular loading is transferred through the backfill material to the extrados of the arch, represents, at first glance, the simplest structural condition. As will be seen later, this is not the case and has led to much research for the specific case of the masonry arch bridge in an attempt to improve our understanding of such structures.

The spandrel may be open, with columns or hinges used to transfer the deck loads to the arch. In an attempt to minimise the horizontal thrust on the abutments, the deck may be used to 'tie' the arch. Tied arches are particularly appropriate when deck construction depths are limited and large clear spans are needed (particularly if ground conditions are also difficult and would require extensive piling to resist the horizontal thrusts).

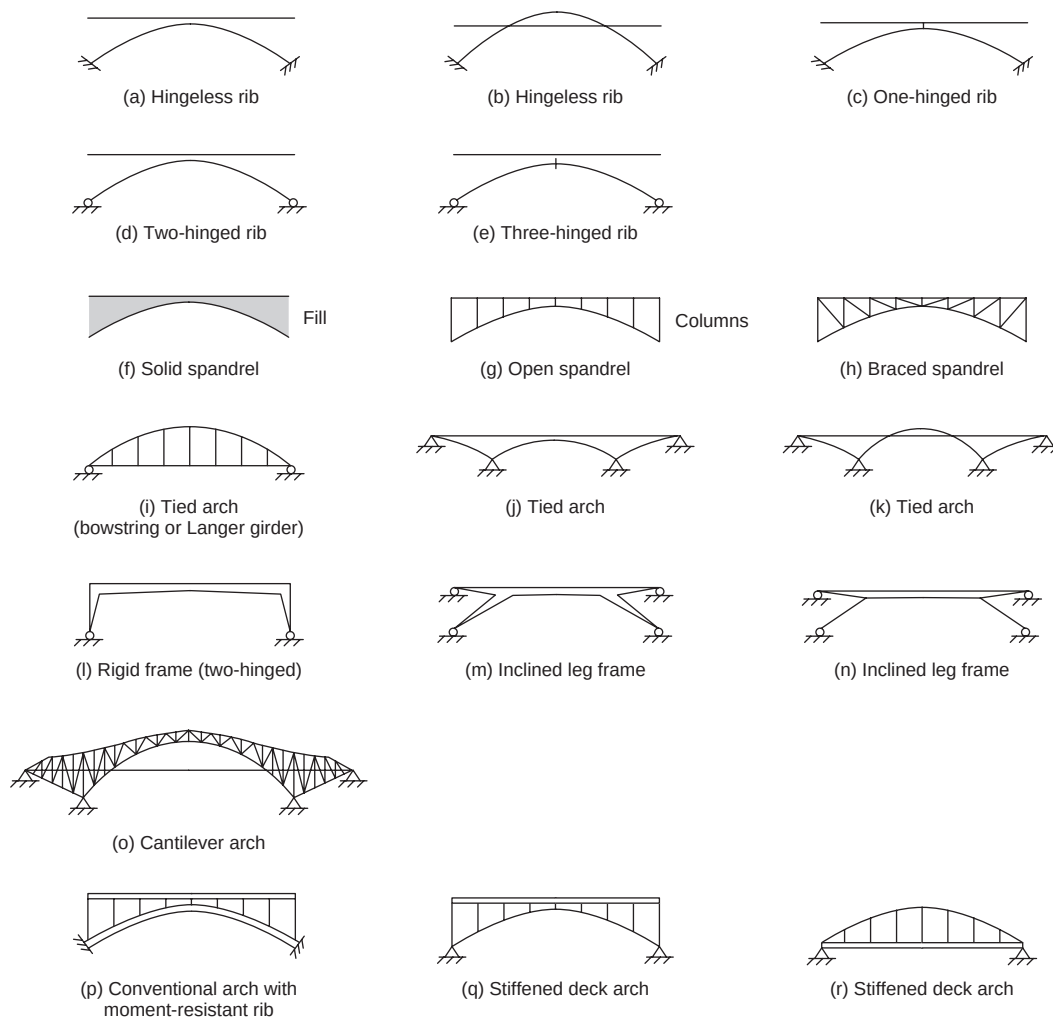


Figure 13.2 Types of arch bridge (O'Connor, 1971)

Returning to Hooke's anagram, the perfect shape for an arch would be an inverted catenary – this would only be the case for a bridge carrying its own self-weight. Vehicle loading and varying superincumbent dead load both induce bending moments. Consequently, the arch must have sufficient thickness to accommodate the 'wandering' thrust line.

For ease of setting out and construction, simpler shapes are adopted nowadays – segmental or parabolic shapes are used, although in situations where maximum widths or headroom have to be provided (say over a railway, road or canal) an elliptical shape may be required, or its nearest 'easy' equivalent, the three-centred arch.

It is worth commenting at this stage on the idealisation of arch structures. Traditionally, arches are perceived as two-dimensional structures; this, of course, is not true – but the extent to which it is not true should be of concern to the designer or assessor. Even in the case of a three-hinged arch, which ostensibly is statically determinate, the 'pins' are capable of transmitting shear even though they theoretically cannot transmit moments. In the case of non-uniform transverse distribution of loading, the hinges will transmit a varying shear, which will produce torsion in the arch. Moreover, in the case of skew arches or non-vertical ribs, the structure has a much higher redundancy and hence will require greater attention to detail in regard to the releases that are engineered into the structure.

From an aesthetic point of view, arches have a universal appeal. In spite of this, care must be taken, as the impact of even modest-sized bridges is significant. Filled arches are invariably masonry (or widening of masonry) bridges. Cleanliness of line, honesty of conception and attention to detail are vital ingredients to a successful bridge. Certainly, simple string courses and copings are preferable to elaborate details, which would be expensive and inappropriate for most modern bridges. Where stone is used, it is important to be sensitive to the nature of the material. Modern quarrying techniques should be employed (laser or water-jet cutting, diamond sawing, flame texturing and sandblasting), reserving traditional dressing to conservation schemes. If brickwork is used, different textured or coloured bricks and mortar can be specified. Here, string courses can be particularly useful to mask changes in bedding angle.

Historically, abutments either comprised rock or else were massive masonry supports, relying on their weight to resist the thrust of the arch. In terms of structural honesty, this is necessary, as it is instinctive to expect such support.

Reinforced concrete and steel arches are altogether much lighter structures. 'The structure consists basically of the arch, the deck and usually some supports from the arch to the deck – in that order of importance. These elements should be expressed in both form and detail, and with due regard for their hierarchy' (Highways Agency, 1996).

It is important that the deck, if it rests on the crown of the arch, should not mask it in any way. Any support, whether spandrel columns or hinges (in the case of the tied arch), should not be allowed to dominate. Preferably, supports should be recessed relative to the parapet and string course.

Concrete arches can be either a full-width curved slab or a series of ribs. Steel is almost always a series of ribs. Ribs formed from concrete-filled steel tubes have also proved popular in recent years. Where ribs are used, thought should be given (if they are going to be seen from underneath) to the chiaroscuro of the soffit.

The ratio of span to rise should generally be in the range 2:1 to 10:1. The flatter the arch, the greater the horizontal thrust; this might affect the structural form selected, that is whether or not a tie should be introduced, or the stiffness of the deck relative to the arch.

Typical structures

Monk New Bridge, Lancashire, UK

Monk New Bridge carries a realigned trunk road over a small river (Figure 13.3). The new bridge was constructed to replace an existing masonry arch bridge, which was demolished as the road was realigned. The bridge was constructed using a mass concrete arch 725 mm thick. The square span is 9.9 m with a skew angle of 29°. The ground conditions were not conducive for an arch bridge but the original bridge on the site had been an arch so, to reduce the environmental impact, the new bridge took the same form. Consequently, piled abutments were used; these were tied together to resist the horizontal thrust from the arch barrel. A novel feature of the bridge was the use of through-thickness inclusions in the mass concrete arch, the purpose of which was to act as crack inducers to convert the mass concrete continuum into voussoir blocks to replicate the articulation of the old stone arch bridge. Detailing the inclusion positions required care. The bridge was subjected to test loading, which demonstrated the low stress levels in the arch, its composite action with the backfill and that forces were induced in the ties under live loading.



Figure 13.3 Monk New Bridge

Chippingham Street Footbridge and Cycleway, Sheffield, UK

The Chippingham Street Bridge carries a footway and a cycleway over a canal, which passes through a steep-sided cutting 10.5 m deep and 52 m wide at the top (Figure 13.4). The ground conditions were good, with 1.5 m of fill overlying sandstone and mudstone beds with a recommended safe bearing capacity



Figure 13.4 Chippingham Street footbridge (Taylor, 1995)

of 600 kN/m^2 . The choice of steel rolled sections, together with the simplicity of line and form, produced a structure that integrates well with its environment, offering an interesting focal point while reflecting the traditional aesthetics of canal structures.

To facilitate ease of construction, a three-hinged approach was used, comprising twin castellated universal column sections 458×305 bent to a circular radius of 20.5 m and braced transversely with angle sections. Vertical universal beam members 356×171 were used to support twin longitudinal beam sections of similar size. The deck comprised an in situ reinforced concrete slab. Lateral stability was achieved using diagonal bracing. Bolted connections were used throughout the steelwork. The hinges were achieved using 'pot' bearings at each of the springings and a bolted connection at the crown.

Mur River Bridge, Austria

The Mur River Bridge is a three-hinged parabolic timber arch bridge (Figure 13.5). The angled supports between the abutment and the arch springing point interact with the main girders to offer longitudinal stability. The vertical posts are rigidly connected to the main girders and arches using steel plates.

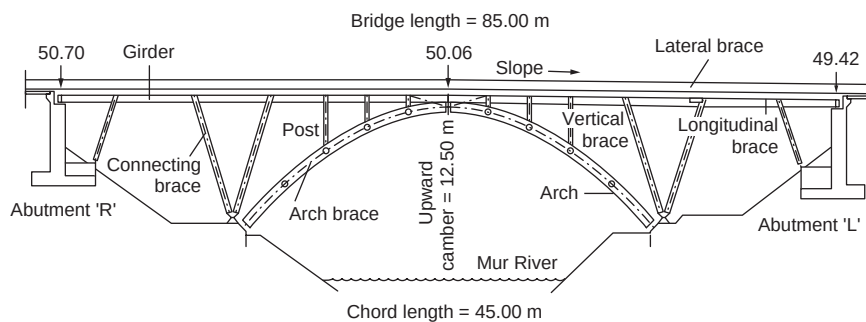


Figure 13.5 Mur Bridge (Pischl and Schickhofer, 1993)

The cross-section (Figure 13.6) and construction photograph (Figure 13.7) clearly show the attention to detail and structural complexity of the construction. The precast concrete deck units were placed on neoprene strips and fixed using glued threaded steel rods 30 mm in diameter. Longitudinal and transverse bracing was also provided (Figure 13.8).

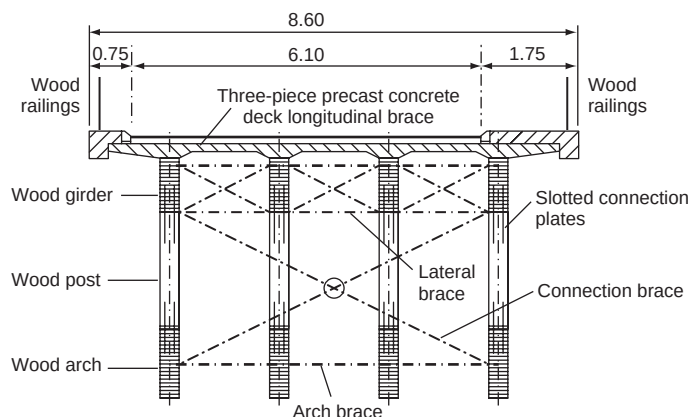


Figure 13.6 Mur River Bridge – cross-section (Pischl and Schickhofer, 1993)



Figure 13.7 Mur River Bridge – construction (Pischl and Schickhofer, 1993)

Wisconsin Avenue Viaduct, USA

The Wisconsin Avenue Viaduct in Milwaukee, Wisconsin, is a 444 m long, 11-span precast, prestressed concrete arch structure (Figure 13.9). The arches are provided by curved trough-shaped, precast post-tensioned arch segments that function as both load-carrying structural members and self-supporting permanent forms. The deck comprises pretensioned precast concrete beams that were framed into in



Figure 13.8 Mur River Bridge – bracing (Pischl and Schickhofer, 1993)

situ concrete cross-beams. The use of precast concrete units mitigates the construction constraints expected during severe weather conditions.

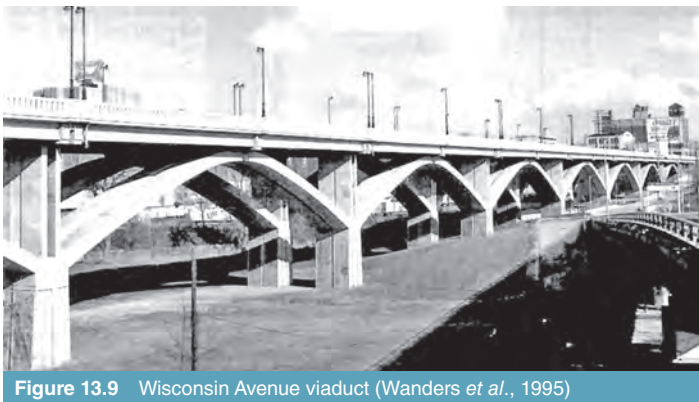


Figure 13.9 Wisconsin Avenue viaduct (Wanders *et al.*, 1995)

This structure is of particular interest as the alternative arch solutions were presented by the designers (Wanders *et al.*, 1995). Figure 13.10 shows the four alternative arch solutions that were considered. Alternative 1 comprised a deeply arched box girder. Although the temporary works would be reusable, it was thought that this might prove difficult to construct.

The second alternative proposed was for a classical concrete arch with an open spandrel. The client had specified that deck replacement had to be possible without total demolition. This resulted in the provision of arch ribs and spandrel columns with longitudinal support girders below the deck. The designers were not convinced that a replacement deck could be installed economically.

A solid or filled spandrel arch design (alternative 3) offered a viable solution to the deck replacement issue. The spandrel walls would stiffen the arch rib and also provide support for the transverse deck ribs. The deck supports comprised equally spaced transverse beams and were thus fully replaceable. The forming system would be reusable. This proved to be the most expensive solution.

The adopted solution was alternative 4, which has already

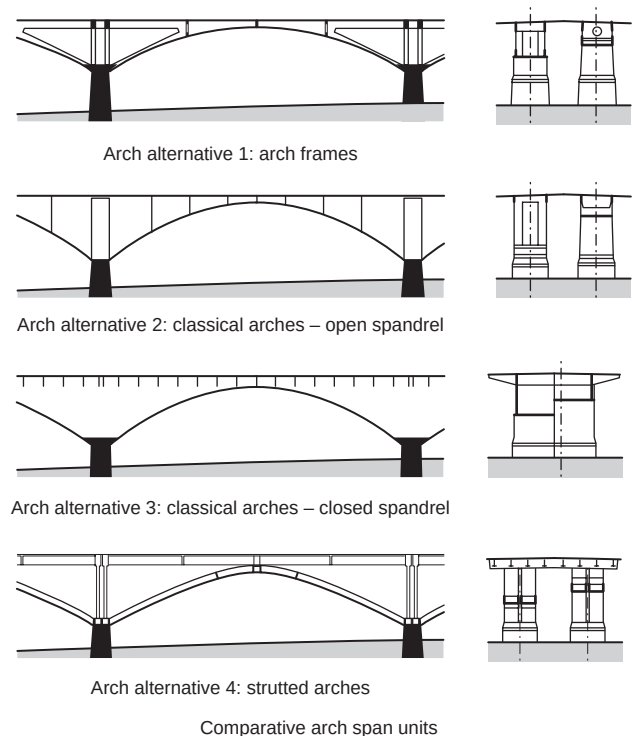


Figure 13.10 Wisconsin Avenue schemes (Wanders *et al.*, 1995)

been described. Because the arch ribs were precast, minimal support from ground level was required. The segments were connected and prestressed at the crown, then filled with concrete. The precast prestressed concrete deck beams were then installed and the deck, diaphragms and cross-beams finally cast using temporary supports of the extrados of the arch ribs.

Commercial Street Bridge, Sheffield, UK

The bridge carries an arterial tramway close to Sheffield's city centre (Figure 13.11). There were many constraints to the site, which led to a tied arch solution that offered minimum depth of construction and an enhancing visual impact. All the main members are in steelwork. The segmental arch members are pinned at each end and are braced together over the central section. Each of the 1.6 m deep steel-edged beams ties its respective arch and is suspended by a series of 60 mm diameter solid high-tensile steel bar hangers. The planes of the two arches are inclined towards each other at the crown, with the result that the hangers appear to criss-cross from any viewpoint. The deck is formed using transverse rolled steel beams spanning between the steel edge beams and acting compositely with a 250 mm thick reinforced concrete deck slab.

There was a conscious effort by the designers to honestly express all the main features, including the pins, hanger connections, bracing, and so on, to produce a very satisfactory solution (Figure 13.12).



Figure 13.11 Commercial Street Bridge (Wilson and Jones, 1995)



Figure 13.13 Hulme Arch Bridge



Figure 13.12 Commercial Street Bridge

Hulme Arch Bridge, Manchester, UK

The bridge has a 52 m span, supported by 22 diagonal spiral strand cables 51 mm in diameter and hung from a single parabolic arch (Figure 13.13). The arch is made from a tapering trapezoidal steel box section coated in aluminium paint. The trapezoidal section varies from 3 m wide and 0.7 m deep at the crown to 1.6 m wide and 1.5 m deep at the springings. The arch rises 25 m above the bridge deck and is connected to each of the springing foundations using 32 high-tensile stainless steel 40 mm diameter bars. The bridge deck comprises a composite concrete slab cast on permanent formwork supporting 17 transverse girders, spanning between steel edge beams. The support cables are attached to outrigger brackets.

Because the arch spans diagonally across the deck, the arch is subjected to not only the vertical arch forces but also significant out-of-plane bending moments and shears. This necessitated extensive stiffening, and the crown section was filled with lightweight concrete. The lower sections of the

arch were designed for sacrificial allowance of 0.6 mm per exposed surface and additionally, were shop-and-site-coated with a vapour corrosion inhibitor (VCI) using a micromist wet-fogging process.

Tyne Bridge, UK

The Tyne Bridge has served as a main route connection to the Newcastle conurbation across the River Tyne since its construction in the 1920s. It comprises a two-pinned steel truss arch structure, spanning the entire width of the river (Figure 13.14). Most of the bridge deck is suspended from hangers attached to the lower members of the arch ribs, while the remaining deck is supported by spandrel columns off the arch ribs below. The main arches comprise two parallel steel arch ribs. Each rib spans 162.76 m between pinned skewbacks (the steel pins are 305 mm in diameter) and rises 51.82 m to the highest point. The ribs have a box section made up of riveted steel angles and plates. The hangers are formed from two 305 × 102 mm channels connected using 254 × 12 mm plates riveted to their flanges.



Figure 13.14 Tyne Bridge

Runcorn–Widnes Bridge, UK

This bridge is a fine example of a truss cantilever arch (Figure 13.15). Its main span is 330 m (1082 ft), while the side spans are only 76 m (250 ft), a little smaller than is usual.

The concrete deck is supported on stringers between the cross-girders, which are supported by 70 mm (2.75 in) diameter locked coil hangers from the arches. Lateral bracing is provided in the deck and at the levels of the upper and lower chords of the arch. There is no lateral bracing between the upper chords of the side spans as they are in tension, owing to the cantilever action of the side spans, which are held down by two pairs of vertical links.

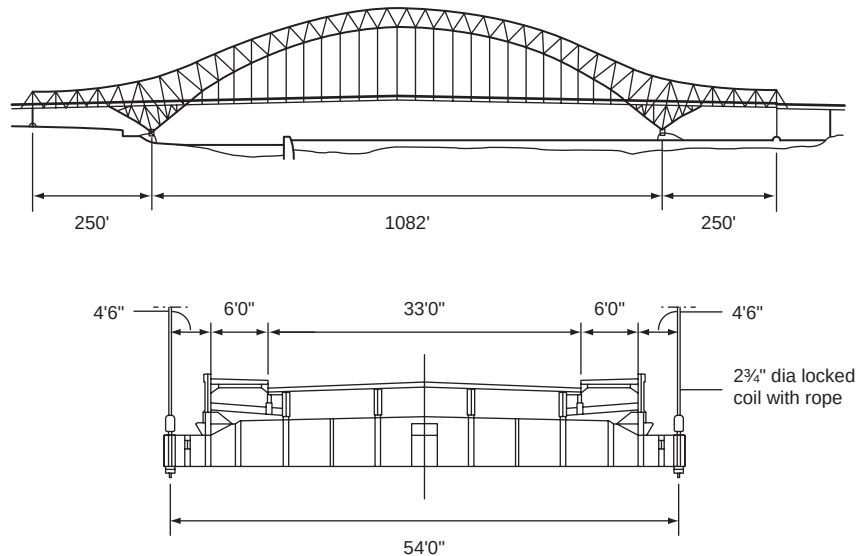


Figure 13.15 Runcorn–Widnes Bridge (O'Connor, 1971)

Port Mann Bridge, Canada

The Port Mann Bridge across the Fraser River near Vancouver is an interesting variation of the tied arch (Figure 13.16). The central part of the main span is supported by rigid hangers, while the remaining parts are supported on spandrel columns secured to the extrados of the arch, which descends below the deck to intermediate piers. This offers some cantilever support to the orthotropic steel deck, reducing its deflections.

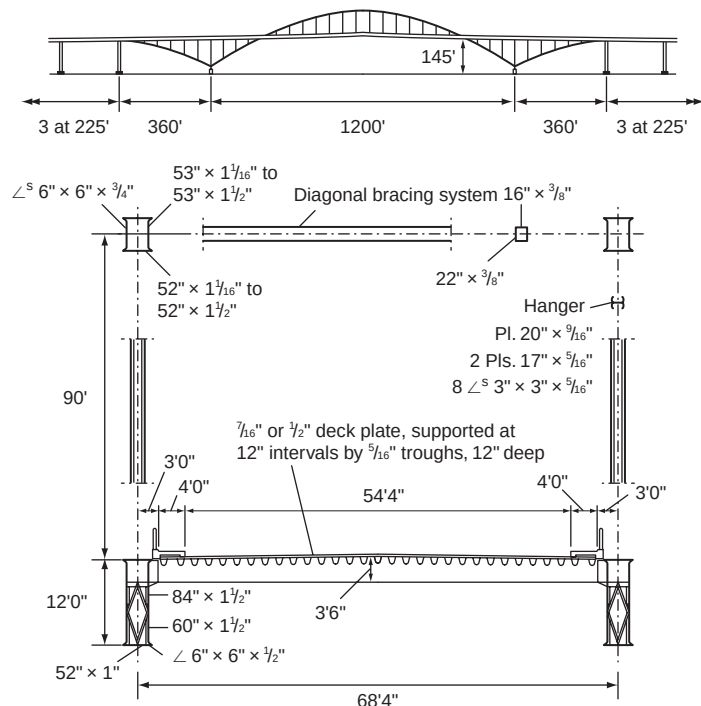


Figure 13.16 Port Mann Bridge (O'Connor, 1971)

Brno–Vienna expressway bridge at Rajhrad, Czech Republic

The bridge carries a local road across the Brno–Vienna expressway near the town of Rajhrad in the Czech Republic. The total length of the bridge is 110 m, with a central arch span of 67.5 m.

At an early stage in the design, it was decided that, for aesthetic reasons, an arch bridge was the preferred solution. The designers considered four options (Figure 13.17) (Strasky and Husty, 1999):

- two concrete arches, supporting a double-T concrete deck on single spandrel columns
- a narrow steel box arch, supporting a box deck on single spandrel columns
- a steel tube arch connected to a slender concrete deck with a tubular truss
- a steel tube arch supporting a channel deck with triangular steel struts.

Although the bridge is rectangular in plan, it crosses the expressway at a skew angle of about 30°. Computer renders highlighted the visual disorder of the first and third options, so these were eliminated. The remaining options had similar cost estimates but the final option was considered more structurally efficient and easier to maintain.

The arch comprises a 900 mm diameter 30 mm thick steel tube filled with concrete. It has a radius of 74.75 m, a span of 67.5 m and a rise of 8.05 m (Figure 13.18). The tube is internally stiffened every 2 m by a pair of ring diaphragms, which maintain the shape of the tube, transfer shear forces and act as bearing stiffeners under the struts. The struts (spaced 6 m apart

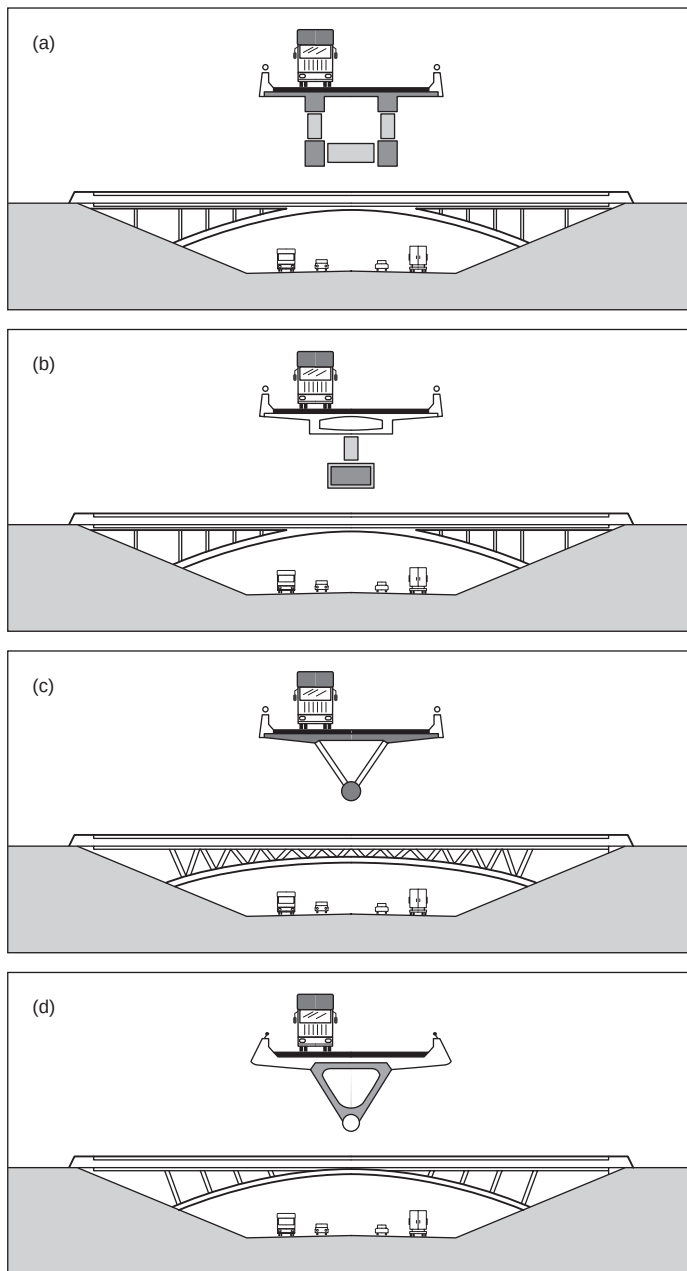


Figure 13.17 Brno–Vienna expressway – schemes (Strasky and Husty, 1999)

at their centres) are perpendicular to the arch, which means that the deck spans vary. They are welded to the arch at the locations of the internal diaphragms and are connected at the top by transverse beams that support the prestressed concrete deck. At midspan, the deck is monolithically connected to the arch over a 5 m length. The concrete in-fill has a characteristic strength to 50 N/mm² and was pumped from the springings to the top of the arch. Prestressed concrete end struts support the side spans. They are connected to the arch abutments, thus allowing longitudinal movement by rotation, while providing transverse stiffness through triangulation.

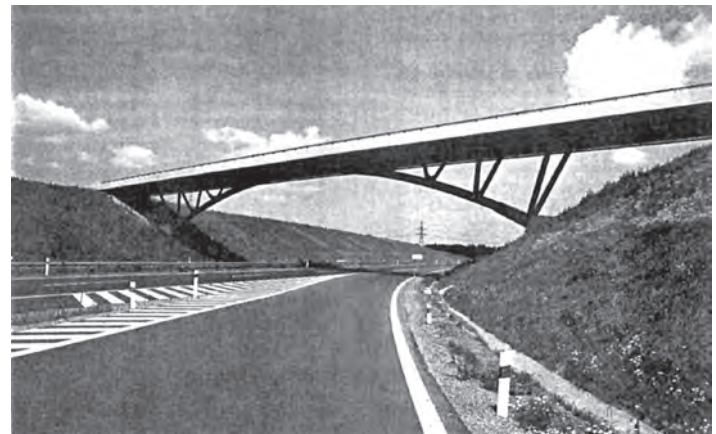


Figure 13.18 Brno–Vienna Expressway – structure (Strasky and Husty, 1999)

The short haunches are formed by welding two tapered channels to the top and bottom of the tube. Each end of the tube is welded to a steel base plate with longitudinal stiffeners and openings for hydraulic jacks. The base plates are secured by four 100 mm diameter bolts to the footings, which are essentially mass concrete. The bridge was analysed as a three-dimensional frame on spring supports.

Analysis

The simplest type of arch is the three-hinged arch. It is statically determinate and therefore can be analysed easily. It is usual to take a free-body diagram from a support to the internal hinge. Equilibrium of the free-body diagram together with the equilibrium of the arch as a whole enables the support reactions and horizontal hinge forces to be determined. The force at any other cross-section can be calculated by cutting the structure and considering equilibrium. It should be remembered that the shear is determined normal to the local axis of the arch, which means that, apart from at the crown, both the horizontal and vertical applied forces will contribute to the shear and axial arch rib forces.

Influence lines can also be easily constructed to determine the approximate positions of loading to create maximum bending moments, reactions, and so on. Different shapes – segmental, parabolic, and so on – can be dealt with easily, the only complication being the geometry.

Consider a three-hinged parabolic arch span, L , and rise, h (Figure 13.19). The equation for the arch is

$$y = (4h/L^2)x(L-x) \quad (13.1)$$

To determine the influence line for the bending moment at the quarter span point, apply a unit load at x , as shown in Figure 13.19

$$V_A = 1((L-x)/L) \quad \text{and} \quad V_B = x/L \quad (13.2)$$

Taking moments at C for the free body BC

$$V_B(L/2) - H_B h = 0 \quad (13.3)$$

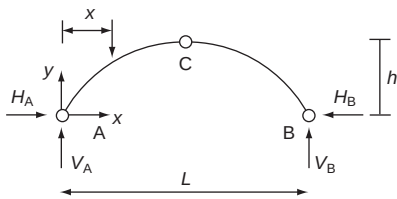


Figure 13.19 Three-hinged arch

therefore

$$H_B = (L/2h) V_B = x/2h = H_A \quad (13.4)$$

When the unit load is applied at the quarter span, the bending moment is

$$V_A(L/4) - H_A(3h/4) = (3L/32) \quad (13.5)$$

It can be shown that, as the unit load passes over to the side of the arch remote from the quarter span point being considered, the influence line bending moment varies linearly from $L/16$ at the crown to zero at the springing. Note that the bending moment is independent of h .

Conversely, the maximum horizontal reaction occurs when $x = L/2$; then $H_A = H_B = H_C = (L/4h)$ and consequently depends on the span-to-rise ratio.

By removing one of the hinges, the arch becomes statically indeterminate, having one redundancy. The two hinges are usually placed at the springings (Figure 13.20). From a practical point of view, it may be considered that this is the most likely condition of an arch, given that most foundations will accommodate a little rotation.

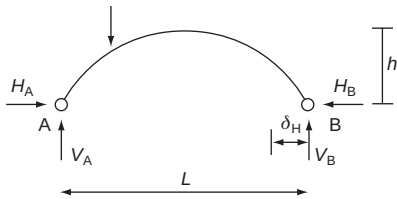


Figure 13.20 Two-hinged arch

For a linear elastic structure, the strain energy U and complementary energy C are equal. It follows from Castigliano's theorems that the partial derivative of the strain energy, U , with respect to a force, is equal to the displacement in the direction of the force.

The strain energy of an arch rib is made up of three components:

- strain energy owing to bending

$$U_B = \int_A^B (M^2 ds) / 2EI \quad (13.6)$$

- strain energy owing to axial thrust

$$U_T = \int_A^B (T^2 ds) / 2AE \quad (13.7)$$

- strain energy owing to shearing force

$$U_S = \int_A^B (S^2 ds) / 2GA \quad (13.8)$$

so that

$$U = U_B + U_T + U_S \quad (13.9)$$

For an arch rib $\partial U / \partial H = \delta_H = 0$ for the special case where there is no movement of abutments. Hence

$$\frac{\partial U}{\partial H} = \int_A^B M \frac{\partial M}{\partial H} \frac{ds}{EI} + \int_A^B T \frac{\partial T}{\partial H} \frac{ds}{EA} + \Delta \int_A^B S \frac{\partial S}{\partial H} \frac{ds}{GA} \quad (13.10)$$

where Δ is a constant, dependent on the shape of the cross-section.

At a preliminary stage, or when checking computer analysis output, it is convenient to ignore the axial thrust and shearing force terms and assume no movement of abutments, in which case, this equation reduces to the much simpler

$$\frac{\partial U}{\partial H} = 0 = \int_A^B M \frac{\partial M}{\partial H} \frac{ds}{EI} \quad (13.11)$$

The bending moment in any general arch may be considered as the sum of two loading cases (Figure 13.21), namely

- externally applied loading with the arch on a roller at one of the supports
- the arch with a roller at the same support and an unknown horizontal thrust applied to the roller.

The total bending moment at x is given by $(BM_x) = M_s - H_A y$, where M_s is the statically determinate bending moment; therefore, $\partial BM_x / \partial H_A = -y$. But the strain energy

$$U_B = \int_A^B (M^2 ds) / 2EI \quad (13.12)$$

$$\frac{\partial U_B}{\partial H_A} = \delta_A = \frac{\partial}{\partial H_A} \int_A^B \frac{M^2 ds}{2EI} = \int_A^B M \frac{\partial M}{\partial H_A} \frac{ds}{EI} \quad (13.13)$$

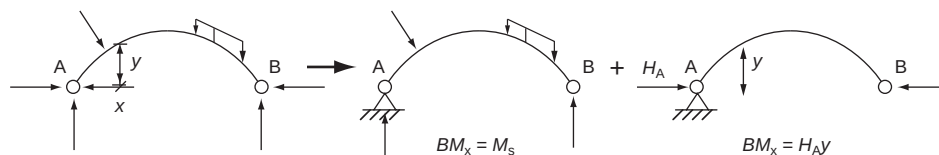


Figure 13.21 General case

$$\begin{aligned}\delta_A &= \int_A^B (M_s - H_A y)(-y) \frac{ds}{EI} \\ &= -\int_A^B M_s y \frac{ds}{EI} + H_A \int_A^B y^2 \frac{ds}{EI}\end{aligned}\quad (13.14)$$

If $\delta A = 0$, then

$$H_A = \frac{\int_A^B M_s y \frac{ds}{EI}}{\int_A^B y^2 \frac{ds}{EI}} \quad (13.15)$$

If $\delta A \neq 0$ or if there is a change in temperature, bending moments are induced in the arch rib. Consider the two-hinged arch AB free to move horizontally at B (Figure 13.20). If the arch material has a coefficient of linear expansion of α per degree Celsius and is subjected to a rise in temperature of $t^\circ\text{C}$, the free expansion of the arch is equal to αtL , (which is very small compared with L). If the thrust H brings the arch back to its original position, then $\partial U/\partial H = \alpha tL$.

At any section, $BM_x = -Hy$

$$\frac{\partial U}{\partial H} = \frac{\partial}{\partial H} \int_A^B \frac{M^2 ds}{2EI} = \int_A^B M \frac{\partial M}{\partial H} \frac{ds}{EI} = \int_A^B (-Hy)(-y) \frac{ds}{EI} = \alpha tL \quad (13.16)$$

$$H = \frac{\alpha tL}{\int_A^B y^2 \frac{ds}{EI}} \quad (13.17)$$

In practice, if the supports yield (Figure 13.22), the value of horizontal thrust, H , is reduced. As the arch spreads, there is a displacement of the points of application of H , which is in the opposite direction to that of H .

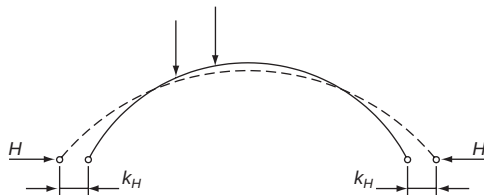


Figure 13.22 Supports yield

If each abutment yields k units of length per unit of thrust, then

$$\frac{\partial U}{\partial H} = \frac{\partial}{\partial H} \int_A^B \frac{M^2 ds}{2EI} = \int_A^B M \frac{\partial M}{\partial H} \frac{ds}{EI} = \int_A^B (M_s - Hy)(-y) \frac{ds}{EI} = -2kH$$

$$-\int_A^B M_s y \frac{ds}{EI} + H \int_A^B y^2 \frac{ds}{EI} = -2kH \quad (13.18)$$

$$\therefore H = \frac{\int_A^B M_s y \frac{ds}{EI}}{\int_A^B y^2 \frac{ds}{EI} + 2k}$$

In the case of combined temperature rise and yield of supports, for a rise of temperature $t^\circ\text{C}$ then $\partial U/\partial H = \alpha tL$. For yield of supports (k), $\partial U/\partial H = -2kH$. Hence

$$\begin{aligned}\frac{\partial U}{\partial H} &= -\int_A^B M_s y \frac{ds}{EI} + H \int_A^B y^2 \frac{ds}{EI} = \alpha tL - 2kH \\ \therefore H &= \frac{\int_A^B M_s y \frac{ds}{EI} + \alpha tL}{\int_A^B y^2 \frac{ds}{EI} + 2k}\end{aligned}\quad (13.19)$$

In the case of the thrust being resisted by a tie (Figure 13.23), then the extension of the tie, $\partial U/\partial H$, is equal to $-(HL_t)/(A_t E_t)$, that is

$$\frac{\partial U}{\partial H} = -\int_A^B M_s y \frac{ds}{EI} + H \int_A^B y^2 \frac{ds}{EI} = -\frac{HL_t}{A_t E_t} \quad (13.20)$$

Therefore

$$H = \frac{\int_A^B M_s y \frac{ds}{EI}}{\frac{L_t}{A_t E_t} + \int_A^B y^2 \frac{ds}{EI}} \quad (13.21)$$

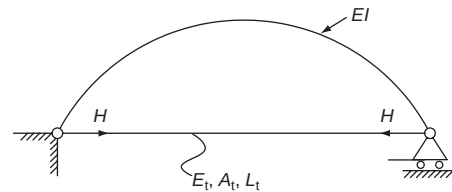


Figure 13.23 Tied arch

It is interesting to note that if the tie and arch material both have the same coefficient of thermal expansion then no extra thrust is developed by changes in temperature.

Consider the specific case of a two-hinged parabolic arch with a secant variation of I (see Figure 13.24). This gives

$$H = \frac{\int M_s y \frac{ds}{EI}}{\int y^2 \frac{ds}{EI}} = \frac{\int M_s y \frac{dx \sec \alpha}{EI_C \sec \alpha}}{\int y^2 \frac{dx \sec \alpha}{EI_C \sec \alpha}} = \frac{\int M_s y dx}{\int y^2 dx} \quad (13.22)$$

$$\begin{aligned}\int M_s y dx &= \int_0^{kL} (1-k)x \frac{4h}{L} \left(x - \frac{x^2}{L}\right) dx + \int_0^{(1-k)L} kx \frac{4h}{L} \left(x - \frac{x^2}{L}\right) dx \\ &= (1-k) \frac{4h}{L} \left[\frac{x^3}{3} - \frac{x^4}{4L} \right]_0^{kL} + k \frac{4h}{L} \left[\frac{x^3}{3} - \frac{x^4}{4L} \right]_0^{(1-k)L} \\ &= \frac{hL^2}{3} (k - 2k^3 + k^4)\end{aligned}\quad (13.23)$$

$$\int y^2 dx = \int_0^k \left\{ \frac{4h}{L} \left(x - \frac{x^2}{L}\right) \right\}^2 dx = 16 \frac{h^2}{L^2} \left[\frac{x^3}{3} - \frac{x^4}{2L} + \frac{x^5}{5L^2} \right]_0^L = \frac{8h^2 L}{15} \quad (13.24)$$

Therefore

$$\begin{aligned}H &= ((hL^2/3)(k - 2k^3 + k^4)) / (8h^2 L/15) \\ &= \frac{5}{8} \frac{L}{h} (k - 2k^3 + k^4)\end{aligned}\quad (13.25)$$

The expression for H may be used to obtain the thrust owing to a uniformly distributed load w per unit length by considering a length δkL and integrating; thus

$$\text{Thrust due to the element} = \delta H = \frac{5L}{8h}(k - 2k^3 + k^4)w\delta kL \quad (13.26)$$

Integrating

$$H = \frac{5L^2 w}{8h} \int_{k_1}^{k_2} (k - 2k^3 + k^4) dk \quad (13.27)$$

Note that when $k_1=0$ and $k_2=1$, that is, the uniformly distributed load (UDL) is applied across the entire span, then $H = wL^2/8h$.

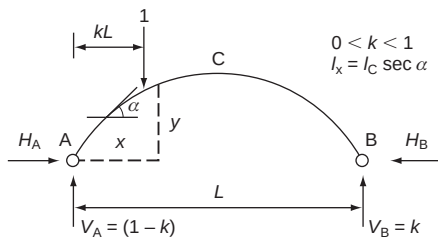


Figure 13.24 Two-hinged arch ($l_x = l_c \sec \alpha$)

The conventional first-order theory may be unsafe for some geometries. It must be remembered that arches are mainly compressive structures and, as such, are susceptible to buckling. Consequently, global and member stability should be carefully considered.

In-plane buckling is possible in modern arches of slender proportions. Closed-form solutions for the problem have been derived, which are of the general form

$$\text{Critical load is proportional to } EI/L^2 \quad (13.28)$$

For a parabolic arch, span L , rise R , with a uniformly distributed dead load, if the dead load moments are zero the horizontal reaction, H , is given by

$$H = wL^2/8R \quad (13.29)$$

Therefore

$$H_{cr} = (W_{cr}/8R)L^2 = C_1(EI/L^2) \quad (13.30)$$

Figure 13.25 plots C against R/L for parabolic arches with 0, 1, 2 or 3 hinges, for the two cases where I is constant and where I is proportional to the secant inclination of the arch. The critical load is significantly affected by the number of hinges and the stiffness of the springing. Although imperfections in the initial shape might not be significant in terms of global stability, they might be critical for local buckling of members or plates and thus lead to collapse.

It is most important to check stability at every stage (especially during construction) and at every level, from the smallest element to the whole structure.

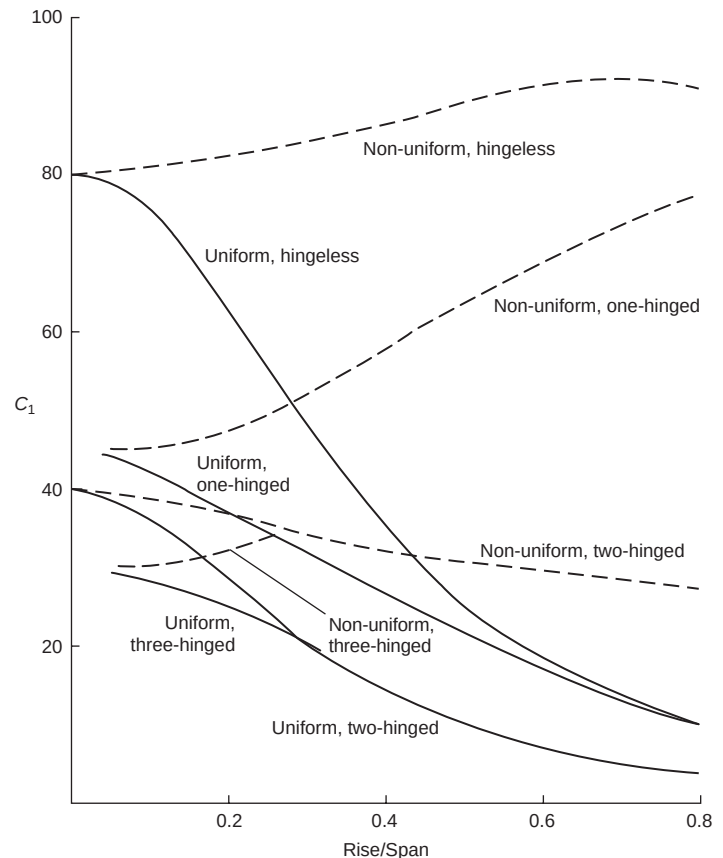


Figure 13.25 Coefficients for in-plane buckling of parabolic arch (O'Connor, 1971)

Design

It is beyond the scope of this chapter to cover all aspects of design, given the range of combinations of material and structural arrangements. Masonry arches will be considered in some detail later; in this section, the criteria for 'modern' materials are considered.

The economic viability of the scheme often depends on the suitability of the site's geology and soil. Clearly, the ideal site is a valley with sound rock valley walls but most engineers have to tussle with poor ground and therein lies the challenge of the design. The span/rise ratio has a fundamental influence on the magnitude of the horizontal thrust. If this becomes prohibitively large, the horizontal thrust must be resisted by tying the abutments together in some way or using the deck to resist the arch thrust.

If the deck is positioned high relative to the springings, this allows a smaller spandrel/rise ratio, which reduces the horizontal thrust (it also increases the relative vertical forces, improving the frictional forces available to resist the horizontal thrust). A 'self-straining arch' (e.g. a bowstring arch) ensures

that the abutment or pier are only called on to resist vertical and braking forces.

The dominance of compressive forces implies the efficacy of concrete as a construction material. Formwork can be expensive but modern proprietary systems are now relatively economical. The segments can be precast, prestressed boxes (as in the case of Gladesville Bridge) or solid reinforced concrete. The need to reduce the self-weight is important for larger spans but not for small to medium spans, where the extra dead weight helps to precompress the arch and thus enhances its performance as an arch.

For modest spans, prefabricated steel sections can be used to great effect. Modern automated fabrication machinery allows complicated shapes to be produced economically and may include concrete 'filling' to act compositely with the steel should plate buckling be a concern.

Orthotropic decks can be an advantage when dead weight is a problem, although attention must be given to the aerodynamic stability of the deck and its interaction with the arch (and hinges, if used).

Truss arches must be designed with particular sensitivity to the aesthetic impact of the members. Merely considering elevations might not reveal the 'business' of the structure – software now exists to allow a three-dimensional inspection of the structure. It is strongly recommended that such software be used in all but the simplest frames. However, the weight advantage might become decisive in long-span bridges. Truss bridges are usually relatively easy to erect but can suffer from the penalty of high maintenance costs and relatively low allowable stresses, on account of the lateral stability requirements of the design.

Masonry arch construction

It is very important at the outset to dispel the idea that all masonry arches are of similar construction. Nothing could be further from the truth. Whether designing a new bridge, assessing an existing bridge or formulating a repair and maintenance strategy, it is vital that a holistic view is taken of the interaction between each of the elements of the bridge and that it is a three-dimensional structure.

Figure 13.26 shows a typical arch bridge construction. Where practicable, the bridge foundations were taken down to rock. This was not always possible and so timber piles or grids – even faggots or nothing at all – were used. Of course, only the successful solutions have survived; no doubt, there were many failures.

The barrel may take various shapes – semicircular, parabolic, segmental, elliptical, gothic pointed, and so on – and may comprise dressed stone, random rubble, brickwork or mass concrete. The backfill over the arch may be contained by spandrel walls, which extend beyond the abutments to provide wing walls. The backfill may be anything from ash and rubble through to concrete. Clay was sometimes used as a waterproofing membrane over the extrados of the barrel. To lighten the structure and also to eliminate the horizontal soil pressures on the external spandrel walls, internal spandrel walls were sometimes used. Historically, this form of construction was used on some bridges with spans greater than about 12 m. The proportions of these internal spandrel walls depended on the nature of the available masonry and whether or not the over-spans took the form of stone slabs or arches. Significantly, there are usually

no external indications of the form of internal construction. Even the barrel thickness cannot be relied on to correspond to that observed on the elevations. The latter was frequently proportioned to comply with the aesthetic demands of the client.

Alternatively, internal arches may be provided, which span longitudinally and spring from the extrados of the main arch barrel. These may be totally internal, or extend through the external spandrel walls to provide an aesthetic feature and, in the case of river bridges, an escape route for flood water. An extension of this form of construction takes the form of a series of smaller arches supported by piers resting on the extrados of the main span. This form is particularly prevalent in China.

Assessment of masonry arch bridges

It is vital that any assessment takes a holistic approach; the materials, form of construction, loading, and so on, should all be

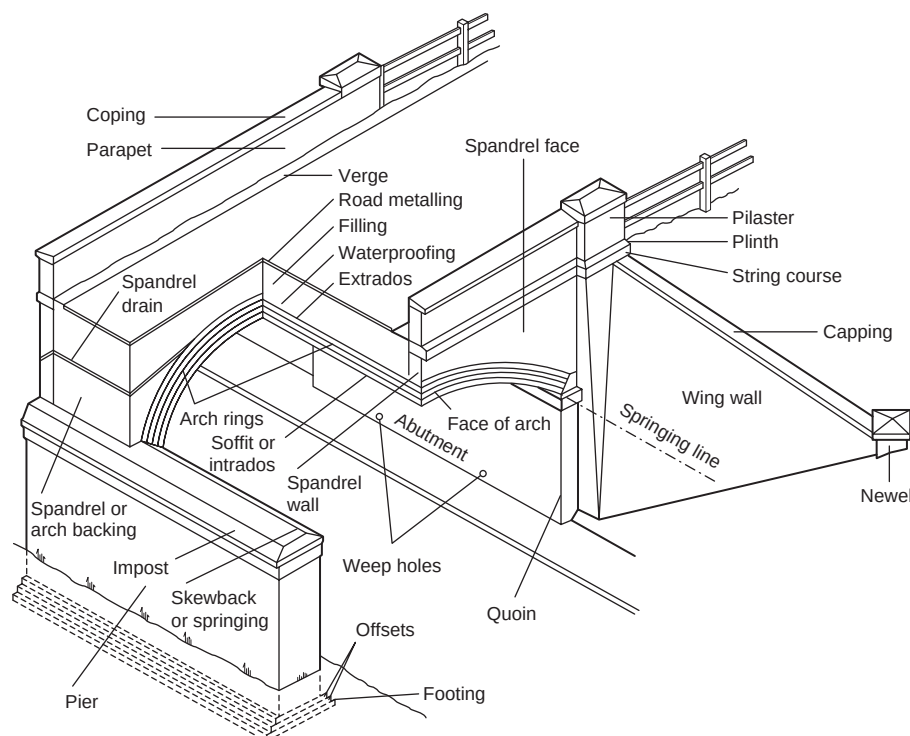


Figure 13.26 Typical masonry arch bridge (Sowden, 1990)

taken into account. All too often, the assessment focuses on the barrel, with lesser regard to its interaction with the other elements of the bridge – when they, themselves, may be critical. In the UK, the current method of determining the load-carrying capacity of bridges is embodied, for highway or railway traffic, respectively, in CS 454 (Highways England, 2020) and NR/GN/CIV/025 (Network Rail, 2006).

The MEXE method of assessment

Traditionally, the first step in an assessment of a masonry arch bridge was to apply the so-called Military Engineering Experimental Establishment (MEXE) method. The MEXE method evolved from work undertaken by Pippard in the 1930s (Pippard *et al.*, 1936) that included both field and laboratory tests to calibrate theoretical work. During the Second World War, this research was used to develop a quick field method to classify bridges according to their capacity to carry military vehicles; this was subsequently adapted for civil use. However, concerns over the predictive capability of the MEXE method have led to it not being recommended for short (<5 m) span bridges, long (>18 m) span bridges and various other scenarios in CS 454 (Highways England, 2020). Thus, alternative methods of analysis and assessment will need to be used in many cases; readers are referred to the next section.

The method involves calculating a provisional axle load (PAL) using either a nomogram given in CS 454 (Highways England, 2020) or the equation

$$\text{PAL} = 740(d+h^2)/L^{1.3} \text{ t (or 70 t, whichever is less)} \quad (13.31)$$

where L is the span, d is the thickness of the arch barrel adjacent to the keystone and h is the average depth of fill at the quarter points of the transverse road profile, between the road surface and the arch barrel at the crown, including road surfacing. The limits set for $(d+h)$ are between 0.25 m and 1.8 m and those for the span, L , are between 5 m and 18.0 m.

The precise origin of the nomogram and its associated equation is not known, but it is attributed to the aforementioned research work started in the 1930s and is probably based on the elastic analysis of a two-pinned parabolic arch with a span/rise ratio of 4, subjected to a point live load applied at the crown. The induced stresses were limited to 0.7 N/mm² in tension and 1.4 N/mm² in compression. The results of a parametric study using these criteria, modified by field observations, led to the given relationship.

Having determined the PAL, it is modified by the following factors, which take account of any departure from the 'standard' arch barrel geometry and condition. The span/rise factor (F_{sr}) assumes that steeper profiled arches are stronger than the flatter ones, with a ratio of 4 being taken as the optimum and hence a factor of 1. This reduces for higher span/rise ratios (Table 13.1).

Span/rise ratio	<4	5	6	7	8
F_{sr}	1.0	0.85	0.75	0.67	0.61

Table 13.1 Span/rise factor (F_{sr})

The profile factor (F_p) takes account of the different shape of the arch; so far, there has been an assumption that the arch is parabolic. This would have a rise at the quarter point (r_q) equal to 0.75 times the rise at the crown (r_c). For values of r_q/r_c less than or equal to 0.75, the profile factor F_p is taken to be unity. For ratios greater than 0.75, for example, for elliptical or semi-circular profiles, the profile factor may be calculated as

$$F_p = 2.3 \left[\frac{r_c - r_q}{r_c} \right]^{0.6} \quad (13.32)$$

The material factor (F_m) is determined using an expression that incorporates a barrel factor, F_b , and a fill factor, F_f , together with d and h

$$F_m = \frac{(F_b \cdot d) + (F_f \cdot h)}{(d+h)} \quad (13.33)$$

The norm for the barrel factor is taken as good-condition limestone or building brick, while the fill factor ranges from 0.5 for a weak fill to 1.0 for concrete backfill. Typical values are presented in Tables 13.2 and 13.3.

Arch barrel	Barrel factor
Granite and whinstone, whether random or courses, all built-in course masonry, except limestone, and all large shaped voussoirs	1.5
Concrete or engineering bricks and similar sized masonry (but not limestone)	1.2
Limestone, whether random or courses, good random masonry and building bricks, all in good condition	1.0
Masonry of any kind in poor condition (many voussoirs flaking or badly spalling, shearing, etc.). Some discretion is permitted if the dilapidation is only moderate	0.7

Table 13.2 Barrel factor (F_b)

Filling	Fill factor
Concrete	1.0
Grouted materials (other than those with a clay content)	0.9
Well-compacted materials	0.7
Weak materials, evidenced by tracking of the carriageway surface	0.5

Table 13.3 Fill factor (F_f)

It is recognised that the strength of masonry is influenced by the size and condition of the mortar joints. As the width of the joint increases, the strength of the masonry decreases. Similarly, the poor condition of the mortar and incompleteness of the perpendicular bed both adversely affect the strength. The joint factor, F_j , is determined using the formula

$$\text{Joint factor } (F_j) = \text{width factor } (F_w) \times \text{depth factor } (F_d) \times \text{mortar factor } (F_{mo}) \quad (13.34)$$

Typical values for each of the factors are presented in Tables 13.4 to 13.6. Finally, a condition factor F_{cm} is applied (see Table 13.7 for typical values). It is important in arriving at a value (between 0 and 1.0) that defects are not double-counted. For example, friable mortar and weak backfill have already been considered.

Width of joint: mm	Width factor
<6	1.0
6–12.5	0.9
>12.5	0.8

Table 13.4 Width factor (F_w)

Condition of mortar	Mortar factor
Good	1.0
Loose or friable	0.9

Table 13.5 Mortar factor (F_{mo})

Condition of joint	Depth factor
Good, fully filled	1.0
Unpointed joints, pointing in poor condition and joints with up to 12.5 mm from the edge insufficiently filled	0.9*
Joints with from 12.5 mm to one-tenth of the thickness of the barrel insufficiently filled	0.8*
Joints insufficiently filled with more than one-tenth of the thickness of the barrel	At engineer's discretion

*Interpolation between these values is permitted.

Table 13.6 Depth factor (F_d)

Defect	Recommended condition factor
Longitudinal cracks owing to settlement of one edge to bridge	0.4 (crack spacing <1 m) 0.4–0.6 (crack spacing >1 m)
Transverse cracks or deformation of arch owing to partial failure of arch or movement of abutments	0.6–0.8
Diagonal cracks	0.3–0.7
Cracks in the spandrel walls near the quarter points	0.8
Where $F_{cm} < 0.4$, immediate consideration should be given to the repair or reconstruction of the bridge.	

Table 13.7 Recommended condition factors (F_{cm})

The modifying factors are applied to the provisional axle load to give a modified axle load

$$\text{Modified axle load (MAL)} = F_{sr} F_p F_m F_j F_{cm} (\text{PAL}) \quad (13.35)$$

There is an assumption that the load is applied through a double-axled bogie and that the surface vertical alignment is such that there is no lift-off of either axle. If this is not the case, further modification is necessary.

There is the further assumption that the bridge comprises a single span between stable abutments. Clearly, this is not always the case and further modifications should be made, depending on the situation. Values suggested at the time of the original development of the MEXE method were

- 0.9 for arches supported by one abutment and pier
- 0.8 for arches supported on two piers.

It should be borne in mind that there appears to be no theoretical justification for these factors.

The MEXE method has been modified for use in the assessment of railway bridges.

Although the MEXE method was traditionally used as a starting point for an assessment (or even sizing a new bridge), there are many areas of uncertainty. For example, the whole basis of the MEXE method is that ‘the arch is first assumed to be parabolic in shape with span/rise ratio of 4, soundly built in good quality brickwork/stonework, with well-pointed joints, to be free from cracks, and to have adequate abutments’ (DoT, 1997). It is clear from studying the background to MEXE that other conditions were also assumed. These include a secant variation of the second moment of area between the crown and the springing. Additionally, the stress levels are such that the arch barrel can be assumed to comply with linear elastic theory, and that the material is isotropic. Even if all these conditions are fulfilled, it can be shown that the extension of Pippard’s work was flawed by allowing dead load and live load stresses to be added together to give a range of permissible axles. This is because their effects are in opposite senses, so where the depth of fill (h) exceeds the arch barrel thickness (d) there is a real chance that the dead load conditions alone will cause a violation of the tensile stress criterion. It would therefore be expedient initially to limit MEXE to an $h \leq d$ criterion. Sometimes haunching is provided that complies with the secant condition, otherwise the barrel thickness must be checked at the crown, quarter span and springing under the carriageway to ensure that the correct figures are being used.

Additionally, the interaction between fill and arch profile is ignored (probably because the initial work assumed that the backfill contribution to stability was confined to the vertical dead load and ignored any horizontal active or passive contribution). Clearly, arches with a smaller span/rise ratio will derive a greater contribution from the active and passive soil pressures than a flat arch.

In the case of flatter arches, greater attention should be paid to the stability of the abutments and piers, particularly if the

arch is being checked for an abnormal load or a change in the normal loading regime.

It should also be remembered that the profile factor F_p is based on a parabola being the best shape for crown loading. Crown loading is assumed to give the largest stress levels; this is a logical approach for a permissible stress criteria method. Collapse is more likely to occur by the formation of a mechanism for which the worst loading case will be $\frac{1}{4}$ to $\frac{1}{3}$ of the span axle loading, so it is important not to mix factors and criteria between the two approaches.

Similarly, it is important when applying each of the factors that double-counting of the defects, and so on, does not take place. For example, in the material factor, if the brickwork of the arch barrel is deemed to be in poor condition and the factor F_b is taken to be 0.7, the condition factor F_{cm} should not include an allowance for the conditions of the brickwork – it should be based on other deficiencies. If the condition factor F_{cm} is very low, for example, less than 0.4, then CS 454 (Highways England, 2020) indicates that this may indicate that the bridge is at immediate risk, to be managed in accordance with CS 470 (Highways England, 2020). If the bridge has been subject to regular inspections, the records of which are available, it is important to determine whether or not the defects are the subject of continual deterioration, historic and now stable, or new. This should have great bearing on the condition factor and the recommended course of action.

Defects do make modelling difficult if the application of the condition factor results in remedial action being needed to make the bridge safe. Longitudinal cracks are fairly easy to deal with, as a two-dimensional idealisation is the usual starting point and any contribution of spandrel walls (as usual) can be ignored. Very weak material or mis-shaped arch barrels can be accommodated in the input files. Things become more difficult when ring separation, diagonal cracking or non-uniform spandrel wall movements are present. In these cases, a more sophisticated model is required, probably, three-dimensional. All of these problems are further complicated if the bridge is skewed.

Other assessment methods

As is evident from the preceding section, there are many situations when the modified MEXE method is unsuitable and should not be used. Fortunately, a wide range of other modelling methods are available to establish load-carrying capacity, which will be considered in more detail later in this chapter. However, before embarking on any structural modelling, it is important that the analyst has an appreciation and understanding of the behaviour of the structure that is to be modelled and, if possible, an awareness of the experimental data used to validate the model used. Box 13.1 gives some of the parameters that should be considered; CS 454 (Highways England, 2020) recommends characteristic compressive strength values for brick and stone masonry, which are provided in Figures 13.27 and 13.28, respectively. What is important is the realisation that the construction of masonry arch bridges can be diverse; at the

macroscopic level, this diversity can present the analyst with many problems. Microscopic (i.e. material) level issues also arise, as will be considered in more detail in the next section.

Compressive strength	Elastic modulus
Flexural strength	Shear modulus
Tensile strength	Poisson's ratio
Shear strength	Fracture energy
Area of adhesion	
Bond strength (at mortar–unit interface)	
Friction coefficient (at mortar–unit interface)	

Box 13.1 Material parameters (unit, mortar and masonry) to be considered in a masonry arch bridge analysis

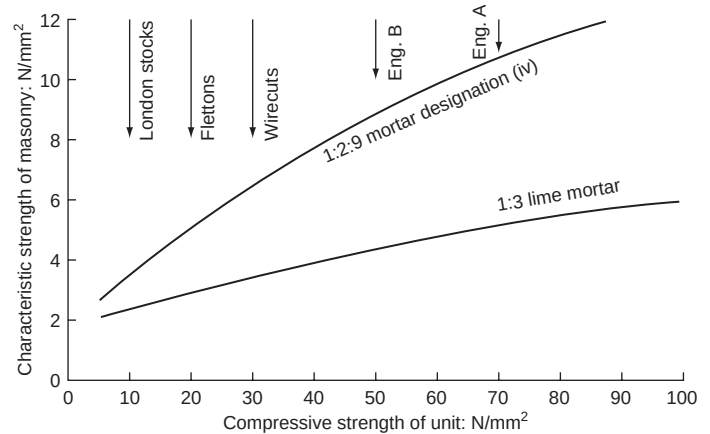


Figure 13.27 Characteristic strength – brickwork (Highways England, 2020)

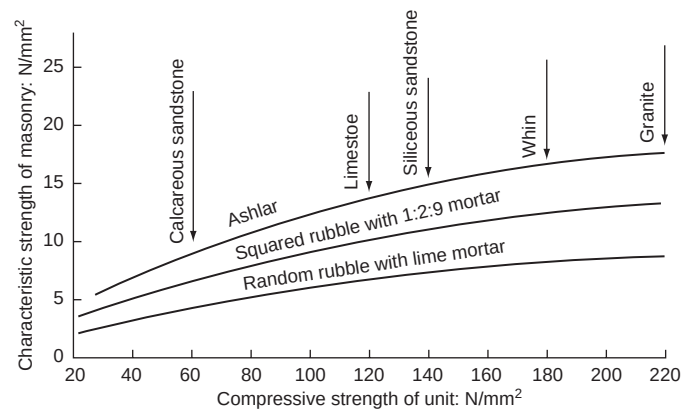


Figure 13.28 Characteristic strength – stonework (Highways England, 2020)

Although most current masonry arch bridge assessment methods focus on the ultimate carrying capacity of the bridge, most bridge owners would like to be able to predict long-term behaviour and residual life. The Sustainable Masonry Arch Resistance Technique (SMART) was recently proposed, as shown in Figure 13.29. This technique provides a multi-level

approach, incorporating all the current methods of assessment or analysis, and gives clear guidance on the philosophy that governs the determination of the safe working loads and ultimate load-carrying capacity (Melbourne *et al.*, 2007a). As part of this approach, a new permissible limit state (PLS) specific for masonry arch bridges is proposed. From this, the method enables permissible working loads, long-term behaviour and the residual life of a bridge to be found. This can be used to prioritise conflicting maintenance demands on limited budgets. The method is based on recent research related to the long-term performance of masonry arch bridges subjected to cyclic loading.

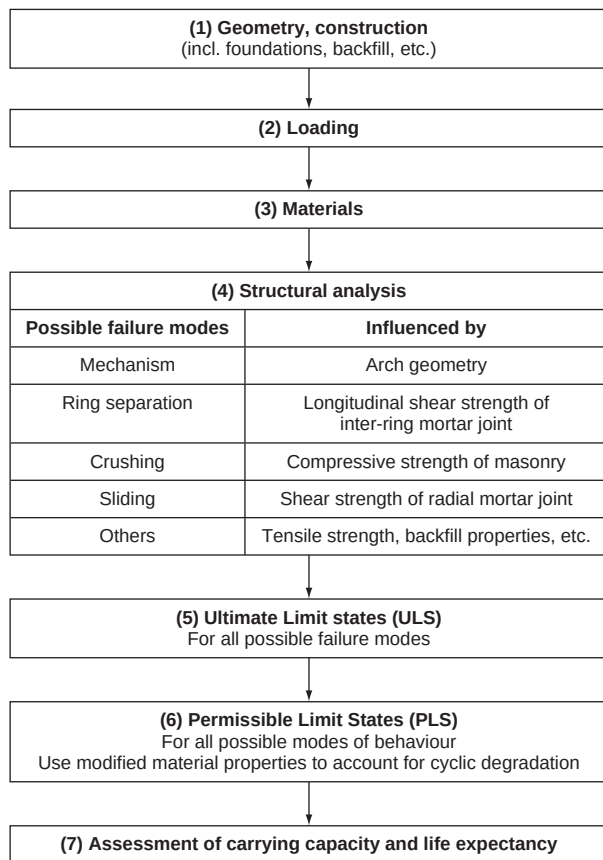


Figure 13.29 SMART assessment procedure

The influence of masonry materials

Masonry (whether comprising bricks or stone blocks) may be considered to be a composite where units are held together (or kept apart, depending on one's perception) by mortar joints. In any event, the properties of the unit, mortar and unit–mortar interface must all be robustly represented in the model. This may be in a form that allows a computer program to decide the nature and extent of the response or it may be greatly influenced by predetermined constraints. It is this range of interventionist modelling (this also applies to experimental strategies)

that can influence the model response and hence the predicted behaviour and carrying capacity.

Figure 13.30 shows a range of possible modelling strategies that may be used for masonry structures. Ideally, the analyst would wish to represent each physical unit with elements possessing the mechanical properties of the unit material (Figure 13.30(a)). Similarly, the mortar would be appropriately represented. The two materials may then be joined at the interface (Figure 13.30(b)). This level of accuracy may be misplaced, given the variability in the constituent materials' properties, which are not insignificantly influenced by the quality of the building work.

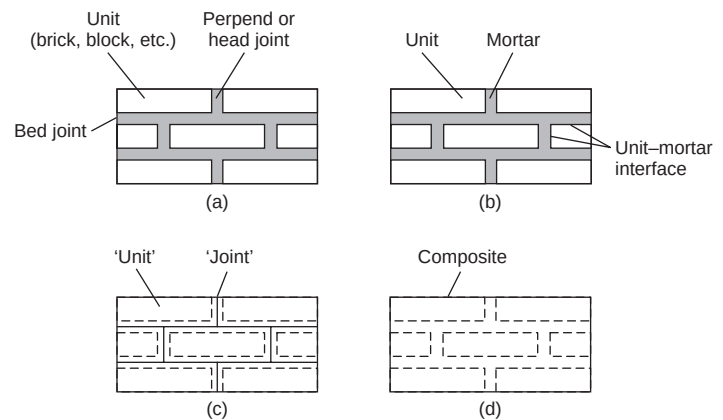


Figure 13.30 Modelling strategies for masonry (Lourenco, 1996)

An alternative strategy replaces the mortar and units by an equivalent material and the interface by an equivalent interface. This retains the ability to locate and trace the development of crack patterns, which can be correlated with actual structural behaviour (Figure 13.30(c)).

Finally, a general composite may be provided, in which the overall properties of the brickwork or stonework are smeared over the structural component. Crack criteria can be set but these may result in a crack pattern that is oblivious of the masonry bonding and so will need some post-processed interpretation (Figure 13.30(d)).

At a microscopic level, both the units (whether brick, concrete or rock) and the mortar display softening, which is a deterioration of mechanical performance under a monotonic or cyclic increase in deformation. Such behaviour is usually attributed to the nature of and defects within ceramic materials and the complexity of their interaction. By the very nature of the construction process, masonry arches contain many flaws. As suggested previously, bricks contain flaws that are a consequence of the manufacturing process and, in engineering quality bricks, are stable at low stress levels. Similarly, the stone blocks may contain flaws initially formed during the geological process that created the rock from which the blocks were quarried. The latter process can reveal or aggravate defects.

Initially, the microcracks are stable and propagation is only caused by increased stress levels. (A durable unit will not be

one where cyclic loading within a known stress regime does not cause crack propagation. Although it sounds quite straightforward to quantify, the situation is significantly complicated by the interaction between the mortar and the units. Deterioration of any of the mechanical properties of the interface bond may change the 'internal' response to a given 'external' stress regime and thus allow crack propagation. This can be an important consideration when assessing arches containing macroscopic defects, such as ring separation in multi-ring arch bridges.) The effect of the presence of cracks is to cause stress redistribution, with the crack surface attracting zero normal stress, thus locally releasing the strain energy. (This phenomenon is utilised in the construction of mass concrete arches containing inclusions (Melbourne and Njumbé, 1998) and in 'protecting' the distressed material.)

Typical stress–displacement graphs for ceramic materials are shown in Figure 13.31. In each case, there is a well-defined softening, which, under soft loading systems, would result in

brittle failure. The more sophisticated mathematical models (e.g. Lourenco, 1996; Rots, 1997) incorporate such parameters as flexural tensile (mode I) fracture energy, G_f^I , and compressive fracture energy, G_c , as material properties. Clearly, in compression, the behaviour of the masonry is significantly influenced by the unit–mortar interface performance under shear loading (Figure 13.31). This leads to the definition of mode II fracture energy G_f^{II} as the integral of the τ – δ graph in the absence of a stress normal to the bedding plane.

Notwithstanding all this, there is a strong correlation between the strength of the masonry and the strength of its constituent parts.

Although the unit–mortar interface may be considered a weakening feature of masonry, it must be remembered that it is this very weakness that provides masonry with the ability to articulate and accommodate movement, without overall loss of structural integrity.

Van der Pluijm (1992) conducted a series of controlled laboratory tests on small solid clay and calcium silicate units. Mode I fracture energies G_f^I were obtained, which ranged from 0.005 to 0.02 N mm/mm² for tensile bond strengths ranging from 0.3 to 0.9 N/mm². Interestingly, it was observed that the bonded area was concentrated in the inner part of the specimen; this was probably a consequence of shrinkage, resulting from surface moisture loss. The effect of this was to reduce the bonded area for the small square interfaces to about one-third of the total area, with the implication that the bonded area for a long wall would be about 60%. This, of course, makes no allowance for the effects of workmanship and presupposes that the mortar–unit interface is engaged under optimum conditions if the unit is neither totally dry or saturated. Hendry (1990a) suggests that there is about 75% full saturation and 100% contact at the time of laying.

Mode II failure is notoriously difficult to simulate under experimental conditions (Jukes and Riddington, 1997). Van der Pluijm (1993) studied the behaviour of clay and calcium silicate units in a specially designed test apparatus, which allowed shear stress measurements under confining compressive stresses of 0.1, 0.5 and 1.0 N/mm². This work raises several interesting observations, which indicate dependency on the nature of the units and mortar.

Figure 13.32 is a plot of shear stress against displacement. The area under the curve is defined as the mode II fracture energy G_f^{II} , with values ranging from 0.01 to 0.25 N mm/mm², for values of initial cohesion, c , ranging from 0.1 to 1.8 N/mm², respectively. The initial friction angle associated with a Coulomb friction model was also recorded; the tangent of this angle ranged from 0.7 to 1.2 for the unit–mortar combinations tested. Significantly, the tangent of the residual internal friction angle remained approximately constant and equal to 0.75. The dilatancy angle was recorded as one unit sheared over another. What is important is its dependency on the confining pressure, the unit surface roughness and the amount of slip. At low confining pressures, the tangent of the dilatancy angle ranged from 0.2 to 0.7, depending on the unit surface roughness. As the confining pressure increased, this tangent decreased, an

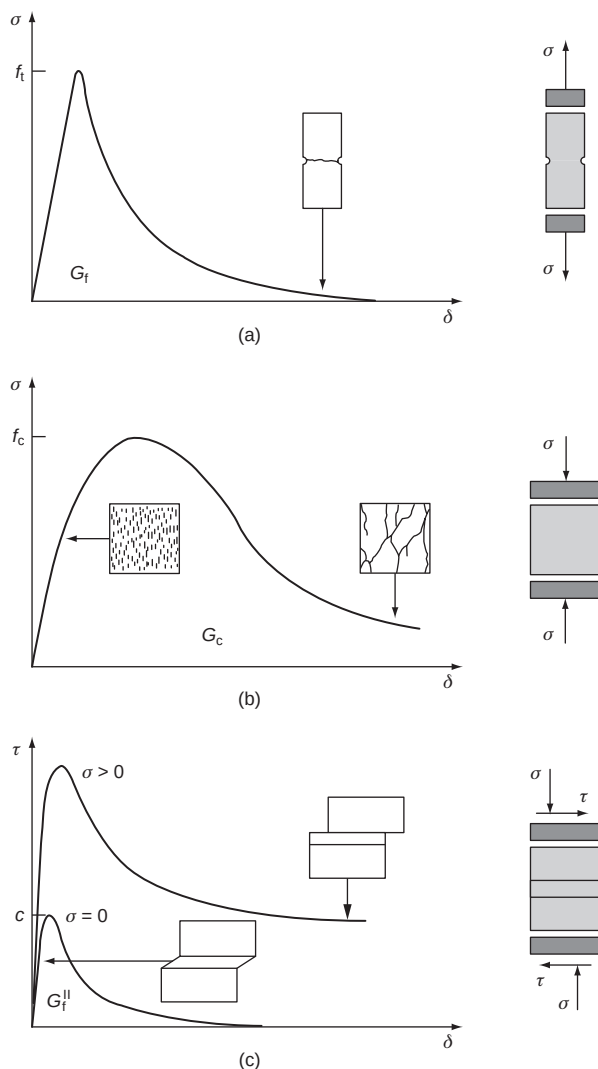


Figure 13.31 Typical behaviour of quasi-brittle materials under uniaxial, tensile, compressive and shear loading (Lourenco, 1996)

obvious consequence of local crushing of the sand particles as they attempted to pass over each other. Similarly, at large amounts of slip, the shearing surfaces smoothed, with the tangent of the dilatancy angle decreasing to zero.

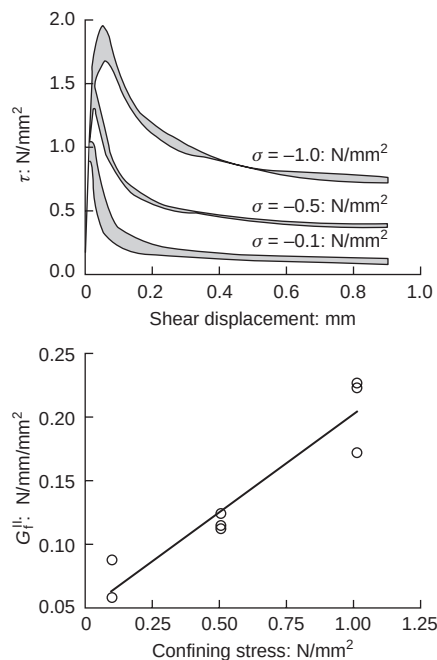


Figure 13.32 Typical shear bond behaviour of joints for clay bricks (van der Pluijm, 1993)

Currently, there are little test data on mode III (torsional) behaviour of masonry. In the absence of such data, it has to be assumed that extrapolation of the mode II data are the best that can be offered. The dependency on confining pressure, unit roughness and amount of slippage can be assumed, although the contact area and shear distribution are open to some speculation.

Having considered, in some detail, the nature of the mortar–unit interface, it is now necessary to consider the overall behaviour of the composite. There is the traditional dilemma between standardised test data and real structure performance; for example, in a typical series of tests, the strength of two-unit specimens was about 11% larger than that of one-unit wide specimens. Under uniform compressive loading, failure usually occurs through the development of tensile cracks parallel to the axis of loading (see Figure 13.33). The degree to which the development of such cracks is prevented owing to constraint will positively influence the carrying capacity. Similarly, it is a lack of constraint of the unit by the mortar that results in the masonry having a lower strength than the unit.

The converse is true for the mortar, which is constrained by the units and consequently achieves a much larger strength, although its relatively high Poisson's ratio leads to a propensity to tensile cracking of the units, as stated already. It follows that the thickness of the mortar beds (and the volume of mortar per

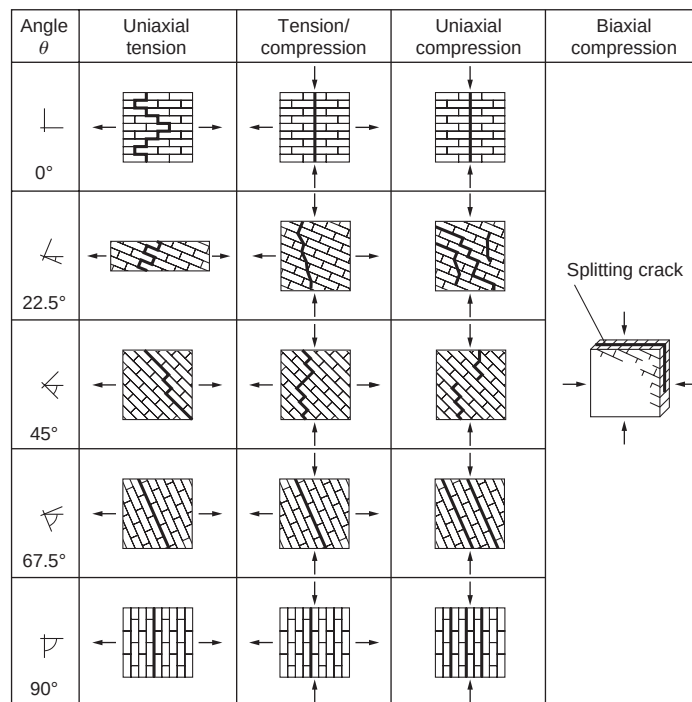


Figure 13.33 Models of failure of solid clay brickwork under biaxial loading (Dhanaseker *et al.*, 1985)

cubic metre of masonry, in the case of random rubble masonry) has a significant effect on strength.

Figure 13.33 shows the modes of failure of solid clay units of masonry under biaxial loading. Although the behaviour of a masonry panel subjected to uniaxial loading perpendicular or parallel to the bedding plane may be well understood and well documented, the response is anisotropic and highly dependent on the bonding and unit or mortar strengths, making the extrapolation of a continuous failure surface somewhat circumspect. Page (1981, 1983) demonstrated the strength enhancement for biaxial loading compared with uniaxial loading. It is believed that the failure mechanism is the same; namely, tensile cracking resulting from the tensile stresses induced at a flaw as the compressive stress flows round it. It is important to notice that, in the case of biaxial compression, the expected mode of failure is splitting in the plane of the applied loads. Figure 13.33 uses bonded brickwork to illustrate the relative significance that strength parameters have on behaviour. Additionally, it must be remembered that masonry can exist in several types, including brickwork, dressed stone and random rubble. Test data are usually obtained from small parallel bedded specimens; however, as the masonry in the barrel is curved, it is inevitable that, in the case of brickwork, the mortar joints are wedge-shaped because the bricks form a parallel pipe. Dressed stone, however, may be cut to give wedge-shaped stone units (voussoirs), which will have a uniform bed thickness (historically, some were so accurately cut that no mortar was required). Finally, random rubble masonry will have varying thickness of both mortar and stone. Some care should be exercised when extrapolating small unit

tests to large bridge structures. Eurocode EC6 (BSI, 1996) has attempted to standardise procedures by designating the reference unit dimensions as $200 \times 200 \text{ mm}$ (height and width) in parallel to the introduction of a shape factor to accommodate national variations. Although EC6 excludes bridges, it may be used as a guide in the absence of any specific code.

The code gives information on the mean compressive strength at 28 days for a range of mortars, although, for assessment purposes, it will probably be quite difficult to obtain *in situ* data. It is recognised that the mortar will usually be weaker and less stiff than the units; hence, under compressive loading, the masonry ultimately fails through cracking of the units. The thicker the mortar joint, therefore, the greater the lateral strains and the smaller the failure load (all other things being equal). For example, in brickwork, thickness variation of between 5 and 20 mm in bedding joints results in approximately $\pm 10\%$ variation in brickwork uniaxial compressive strength compared with that of brickwork with 10 mm thick bedding joints (Hendry, 1990b). The standard test data relate to axially loaded specimens. In an arch, the masonry is subjected to eccentric loading; although this will result in higher stresses at the intrados or extrados, it is recognised that stresses larger than those permitted for axial loading can be tolerated. A figure of 20% has been suggested (Hendry, 1990a) if the eccentricity of the thrust exceeds 0.15 of the arch thickness and is not greater than 0.4 of the arch thickness. Additionally, if the strength has been determined using small-diameter radial cores, it is important to appreciate that the results are not only affected by size but are also perpendicular to the thrust plane in the arch barrel; since masonry is anisotropic, this needs to be taken into account.

To determine the mortar strength in existing masonry is very difficult. It is virtually impossible to recover a competent specimen of sufficient size to test from the bridge. For old bridges, lime mortar was invariably used, with an estimated strength of $0.5\text{--}1.0 \text{ N/mm}^2$; even cement mortars are unlikely to exceed 2.5 N/mm^2 . Fortunately, it has been shown in tests on masonry piers that weak mortars make little difference to masonry strength (Hendry, 1990a).

In most research relating to the stress–strain relationship, a compressive load has been applied perpendicularly to the bedding joint. For this load case, it is generally accepted that the relationship is closely represented by the parabola

$$f/f_{\max} = 2(\varepsilon/\varepsilon^1) - (\varepsilon/\varepsilon^1)^2 \quad (13.36)$$

where $f_{\max}$ and ε^1 are, respectively, the stress and strain at the maximum point of the stress–strain curve. The initial tangent modulus is given by

$$E = 2f_{\max}/\varepsilon^1 \quad (13.37)$$

and the secant modulus is 75% of this.

Approximate values for brickwork and thin-jointed coursed stone masonry are given by $900f_k$ or $400\text{--}600f_m$, where f_k and f_m are the characteristic and mean strengths, respectively (Hendry, 1990b). The value for random rubble may be less than half this value.

There is no guidance for the Young modulus parallel to the bedding plane and certainly no help for considering the general case.

It will be appreciated that for multi-ring brickwork arches, where there are no headers provided, there is a propensity for the rings to separate (or delaminate). Apart from mortar wash-out, the mechanical bond between the rings can be broken by the development of excessive longitudinal shear caused by the thrust moving from one ring to the next or by the stresses induced by compatibility. If the compressive strength of the mortar is expected to be greater than 1.5 N/mm^2 , a shear strength of 0.35 N/mm^2 may be assumed; otherwise, a shear strength of 0.15 N/mm^2 is more reasonable.

Work at Salford (Melbourne *et al.* 2004, 2007b) and Cardiff (Roberts *et al.* 2006) confirmed that brickwork exhibits fatigue behaviour and that S–N type curves can be drawn for such materials. To quantify the fatigue performance of possible modes of failure of the masonry (i.e. crushing, tensile cracks or shear cracks that will initiate the various modes of failure of the structure), a series of S–N curves are proposed. For example, the properties of the masonry in compression can be represented by

$$\sqrt{\frac{\Delta S \times S_{\max}}{S_{\text{ult}}^2}} = A - B \log N \quad (13.38)$$

where A and B are empirically determined constants and N is the number of cycles of loading that develop a change in stress of ΔS , which experiences a maximum stress $S_{\max}$, compared with the ultimate strength of S_{ult} (Roberts *et al.*, 2006).

Analysis of masonry arch bridges

Most masonry arch bridges were conceived as gravity structures, for which mass and geometry were the design criteria. Certainly, the proportions passed down from antiquity reflected no thought of stress criteria and were probably based on bitter experience. However, materials will, of course, influence response, both the masonry materials considered in the previous section and the surrounding soil or other fill materials.

Barlow (1846) demonstrated that there is no unique thrust line associated with a stable arch but that there are many possibilities. Navier (1826) had shown that for linear elastic materials, where plane sections remained plane, tension could be avoided by ensuring that the thrust line lay within the middle third of the section. Combining these two pieces of work led to the well-known middle-third rule of design, which aimed to eliminate tension in the masonry and thus avoid any cracking. It also afforded analysts the luxury of modelling the arch as an elastic continuum.

Castigliano (1879) applied the theorems of minimum strain energy to the arch. (These have already been used to analyse two-hinged elastic arches.) The position of the thrust line was determined as lying within the middle third; if this criterion were violated, the tensile zone was removed and the calculation iterated until no tension was present at any point in the arch.

An advantage of elastic analysis is that stress levels and deflections can be calculated – how meaningful they are is open to much debate but it has to be conceded that they provide a feel for the problem. However, it is universally accepted that masonry arch bridges crack, even before the centring is removed. This is a very important observation that should not be obscured by the sophistication of some currently available modelling software.

The simplest form of modelling involves idealising the arch barrel to a two-dimensional plane frame made up of a number of line elements (normally at least 12 to provide reasonable results). The work can be carried out using any suitable frame analysis or finite-element program. Depending on the level of sophistication required to solve the problem, this may suffice; however, where the structure is more complex, a more robust analysis may be justified, in which case the initial simple analysis can be used to identify regions of interest. Many standard finite-element analysis packages are currently available, for example, ABAQUS, ANSYS, DIANA and LUSAS. Each is constantly evolving and has advantages and limitations. The analyst is faced with a dilemma between using a smeared modelling approach, which averages the unit–mortar composite, and the expense of modelling the unit and mortar separately, with the uncertainty of replication of the unit–mortar interface. The finite-element packages listed require the designation of interface models drawn from a library developed mainly for concrete and so should be viewed in that light. As shown in the previous section, the mode of shear failure at the interface depends on several factors, all of which can be swamped by workmanship factors and the condition of the materials (the materials may be wet or dry, or they might be subject to chemical attack, or the mortar might simply have been washed away). What is important is that the theoretical justification of contact elements, solid elements, and so on, should be investigated thoroughly, in order to understand the limitations of the model. As mentioned earlier, a simple model can be used to determine the regions of particular interest. This allows for some economy in computer time by subsequently concentrating the closely meshed sections into the regions identified using the simple model. It is often the case that a standard finite-element package does not possess a ready-made contact element with the desired properties. It is then up to the ingenuity of the analyst to assemble elements that, in combination, replicate the interface. Care must be taken in setting limits to the bond strength, the tensile strength of the mortar, the shear strength of the interface, and so on. For example, the shear bond strength for a 1:2:9 lime mortar would normally be about 0.2 N/mm², and even this will not be achieved if there is deterioration or poor workmanship. Also, fracture energy should usually be included in the model to obtain realistic and mesh-independent results.

While a computer is commonly used for the analysis of arch bridges, it is useful to undertake some hand calculations to develop a feel for structural behaviour and to check the computer output. For this purpose, closed solutions may be used in conjunction with influence lines to determine carrying capacity.

An alternative analysis approach can be adopted that recognises the particulate nature of masonry and observed collapse mechanisms. The first scientific discourse on the failure of an arch that refers to the formations of mechanisms is reported by Couplet (1729) and illustrated by Frézier (1737, 1738, 1739), based on engravings of tests undertaken by Danyzy (1732). Prior to this, any references to failure modes were purely descriptive. Throughout the nineteenth century and the early part of the twentieth century, the analysis and design of masonry arch bridges depended on elastic analysis and the middle-third rule. It was from the work of Pippard and Baker (1962) that the modern mechanism method evolved. Pippard and Baker idealised the arch as a two-dimensional arch made up of blocks (voussoirs), which possessed the following properties:

- no tension
- rigid constituent elements
- infinite compressive strength
- sufficient inter-surface friction to prevent sliding.

As an elastic continuum, the arch would have possessed three redundancies. Consequently, to become a mechanism, four releases are required, which are provided in the model by four hinges. These may be provided by a combination of rollers and hinges; additionally, material failure is possible.

The simplest model considers only the dead weight of the different elements. The collapse load W (Figure 13.34) can be determined from statics by taking moments about each of the hinges in turn and solving for V_A , V_D and H . This method considers a kinematically admissible mechanism and is therefore an upper bound solution. Consequently, various positions of the hinges must be considered and values for W calculated until the minimum value is determined. This can be done in a systematic way, as can be shown with reference to Figure 13.35, in which OA represents a reference line (usually the intrados) of the arch. The dashed line represents the thrust line. The vertical eccentricities of the thrust line at O and A, respectively, are ε_O and ε_A . Taking moments about the thrust line at A, equilibrium is maintained if

$$(V_A + \varepsilon_A + \varepsilon_O)H + \left[x_1 \sum_{r=1}^n W_r - \sum_{r=1}^n x_r W_r \right] = V x_A \quad (13.39)$$

This can be rearranged to express, in general terms, the vertical eccentricity:

$$\varepsilon = \varepsilon_O - y + \frac{V}{H}x - \frac{1}{H} \left[x_n \sum_{r=1}^n W_r - \sum_{r=1}^n x_r W_r \right] \quad (13.40)$$

By taking moments about the assumed hinged positions, V , H and W can be determined. Using these equations, it can be checked that the eccentricity at any point on the arch is within the limit sets. Care must be taken when solving these equations by hand as small differences of large numbers are usually involved, so rounding errors can have a significant effect on the results. Because the method gives an upper bound solution, it is necessary to vary the position of the live loading and hinges in

order to determine a minimum value for the carrying capacity. As this is a tedious procedure, a number of computer programs have been made available to carry out this type of analysis.

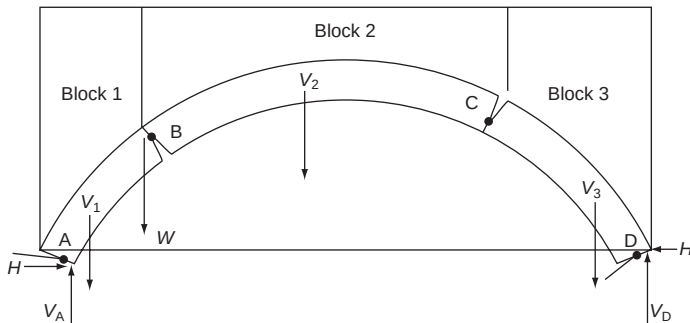


Figure 13.34 Simple mechanism (Page, 1993)

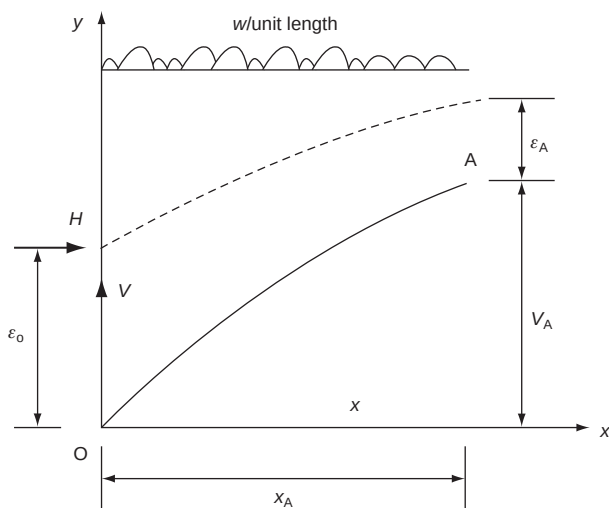


Figure 13.35 General thrust line (Heyman, 1982)

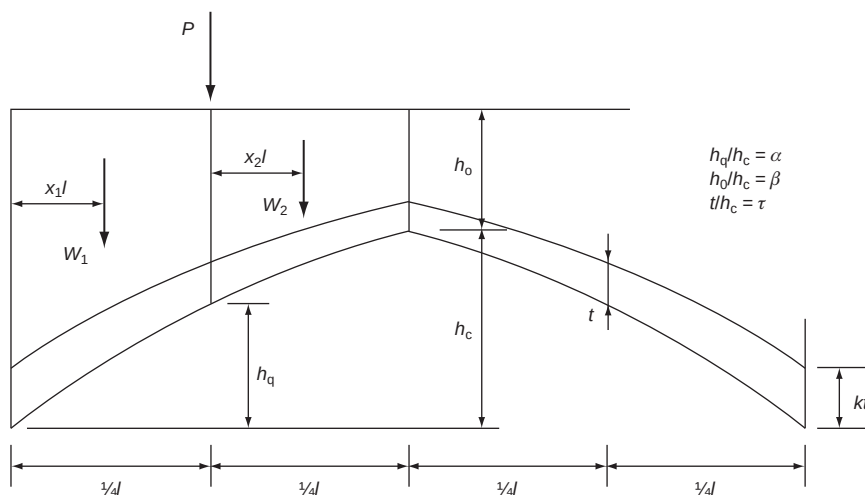


Figure 13.36 Heyman's quick method (Heyman, 1982)

To check any computer output, it is useful to apply a quick approximate analysis. Heyman (1982) presents a simple idealised solution. Figure 13.36 shows the dimensions of the arch and defines the dimensionless parameters that are used to determine the value of P . It is assumed that hinges occur at each of the springings, directly under the load (with no dispersion through the fill or arch) and at the crown. It is also assumed that the backfill and masonry have the same unit weight. This yields

$$P = 16 \frac{W_2 x_2 \left\{ \alpha + \left(1 - \frac{1}{4} k \right) \tau \right\} - \left(W_1 x_1 + \frac{1}{4} W_2 \right) \left\{ (1 - \alpha) - \left(1 + \frac{1}{4} k \right) \tau \right\}}{(3 - 2\alpha) - (2 + k)\tau} \quad (13.41)$$

This equation gives the load that will just cause collapse; it also implies that P is infinite when $(3 - 2\alpha) = (2 + k)\tau$; this corresponds to a geometry that produces a perfect structure, which will be discussed later. (A perfect structure may be defined as one where the thrust line cannot move out of the arch, whatever the value of P . In reality, failure can occur by crushing of the real material). If it is assumed that the intrados comprises a series of straight lines, then

$$P = \frac{\gamma L h_c}{6} \left[\frac{(1 - 3\beta - \alpha) \left\{ \alpha + \left(1 - \frac{1}{4} k \right) \tau \right\} - (6 + 9\beta - 5\alpha) \left\{ (1 - \alpha) - \left(1 + \frac{1}{4} k \right) \tau \right\}}{(3 - 2\alpha) - (2 + k)\tau} \right] \quad (13.42)$$

Heyman (1982) presents tables for quick calculations based on this equation, with a further constraint that $k = 1$, which is safe, and only has a small effect for arches with span/rise ratios of 4 or more.

So far, the barrel has been modelled as a two-dimensional arch comprising discrete rigid voussoirs; the reality is far more complex and requires a more sophisticated model that takes a more holistic approach to bridge behaviour. Early attempts to model other effects (Melbourne and Walker, 1989) demonstrated the need for such an approach.

An extension of the work of Pippard and Heyman has been the development of the rigid block method of analysis (Gilbert and Melbourne, 1994; Livesley, 1978). This method recognises the formulation of the governing equations as being in a form that can be solved using standard linear programming techniques.

The model can be generalised in two dimensions to take into account the effects of sliding between voussoirs, ring separation, restraint from soil, load dispersion and local crushing (e.g. see Gilbert, 2001).

In earlier discussion, it was pointed out that a two-dimensional arch has three redundancies and hence requires four releases to produce a mechanism. So far, releases have been assumed to be hinges; however, they could be roller bearings – or, more realistically, frictional sliders

between blocks. Drucker (1954) showed that an interface that models frictional sliding, and embraces the principle of normality by incorporating dilatancy, will produce an upper bound of the exact solution. It has been shown that, provided the coefficient of friction between the blocks is greater than 0.4, slip is unlikely in the radial directions, given that the axial thrust is much larger than the radial shear.

The same cannot be said of longitudinal shear. This, coupled with the propensity for multi-ring brickwork arches to suffer from mortar washout, presents real difficulties for the assessing engineer. Here, a judgement has to be made. Although the mathematical model maintains a theoretical relationship between the surfaces, this might not exist in reality, so a more pragmatic interpretation of the computer output may be necessary.

Pressures at the extrados present difficulty, for both the analysis for design and the assessment of existing structures. The former is slightly easier, as the designer has some control of the specification of the material and method of construction. The latter situation is far more difficult, as the nature of the backfill might not be known, let alone the pressure distribution. The important thing is to ensure that the free-body idealisation takes into account the true stress or pressure distributions at the boundaries. This is reinforced by a holistic idealisation of the bridge. In the mechanism, idealisation should take account of the pressure distribution on each slice or block. The initial stressed state will invariably not be known, so some suitable assumption must be made. As the arch deforms to create the failure mechanism, the soil or backfill will be subjected to significant (albeit local) strains, which might create large local pressures on the extrados. Significantly, the free body might not 'see' these.

So far, the arch has been considered as a two-dimensional structure; this is clearly not the case. Even square span bridges behave as complex three-dimensional structures, owing to eccentric loading and the interaction between the elements of the bridge. Before considering the available analytical techniques, it is informative to consider the forms of construction of the arch barrel. Figure 13.37 shows the development of three arrangements for a 45° skew segmental circular arch. The simplest form comprises units that are laid parallel to the abutment. This may be acceptable for small angles of skew

but is clearly dependent on the bed shear capacity to transfer load as the skew increases. Additionally, if bricks or square cut stone are used, the elevation of the arch barrel will be stepped and extra work may be required to achieve an acceptable appearance.

The second method is the helicoidal or English method. It ensures that the bed at the crown is perpendicular to the longitudinal axis of the bridge and, by inference, the principal axis of the stress at the crown, owing to crown loading. There is an inferred assumption that (particularly if the arch is filled) the stress perpendicular to the axis is reduced (or is better distributed closer to the abutment). The geometrical constraints are such that, for the beds to remain parallel, their orientation causes the beds to roll over and thus rest on the springing at an angle.

Finally, the orthogonal or French method attempts to keep the bed orthogonal with the local edge of the arch and thus following the perceived line of principal stress onto the abutment. Although this may appear to be desirable, it does mean that all the units will have slightly different shapes; thus, such construction is usually confined to stone arches (and the availability of highly skilled masons). Brickwork cannot be used, as the bedding planes are not parallel.

The analysis of a skewed arch presents many difficulties. There is no universally accepted method, but several methods have emerged that can be used to inform any assessment decisions or new design.

The simplest two-dimensional model is the four-hinged mechanism model. Even as a rib, this becomes a more complex problem once skew is introduced. This is because, in three-dimensional space, the rib has six redundancies and therefore requires seven releases in order to become a mechanism. Additionally, torsion will be induced on the rib and, if the no-sliding criterion is to be maintained, rotation about the axis of the rib can only be accommodated by the existence of pins (in addition to the traditional hinges), which will allow a kinematically admissible mechanism to form.

It has been observed in large-scale tests that, even when full-width line loading is applied parallel to the abutments, the collapse mechanism that develops has more than four hinges, none of which might be parallel to the abutments. This observation helps the analyst to idealise the structure. If Figure 13.38 represents a kinematically admissible mechanism, the following observations can be made.

- Four or more fracture lines are required.
- Elements of the arch barrel between fracture lines are assumed to be rigid.
- If the fracture line is not parallel to the bedding plane and the arch barrel is moving upwards, the axis of rotation is determined by the intrados edge contact points on each elevation of the arch barrel (axis O-O in Figure 13.38).
- If the fracture line is not parallel to the bedding plane and the arch barrel is moving downwards, there is a single point of contact on the extrados (point A in Figure 13.38). This point can act as a universal joint, thus allowing three rotational releases.

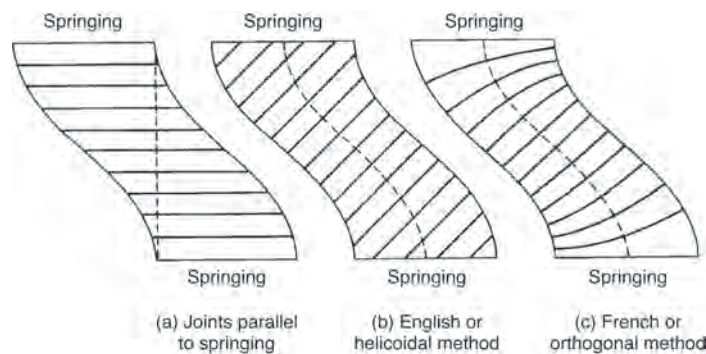


Figure 13.37 Projection of the soffit of a 45° skew angle (Page, 1993)

- The fracture lines usually follow the bedding and perpendicular mortar joints unless the units are relatively weak or there are local high stress levels, in which case the fracture line might go through the unit.
- Sliding is possible.
- Normal shear and torsional rotation are possible.
- In the special case of a multi-ring brickwork arch, barrel ring separation is possible and, given the likelihood of concentrated local load transfer at fracture lines, will be very likely to occur.
- The abutments might not be rigid, in which case, they will accommodate some movement, offering release to the arch barrel.
- Given the concentrated local load transfer, it is likely that local material failure will occur. This might manifest as local spalling, mortar failure or unit failure (splitting).

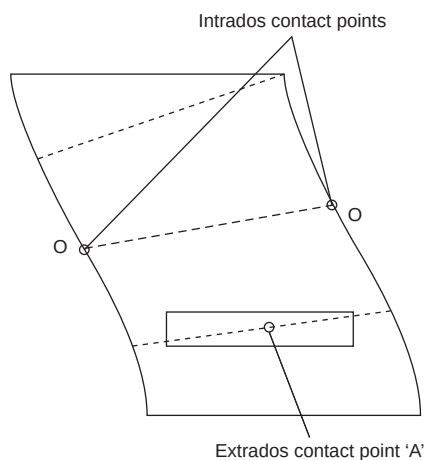


Figure 13.38 Skew bridge fracture lines

Whether the abutment moves as a result of the twist is important with regard to the soil pressures on the extrados, particularly in the vicinity of the springing. If the abutments are fixed and the arch barrel accommodates any rotation purely by forming a hinge at the point where the intrados (or extrados) meets the skewbacks, the local soil strains will be small and the changes in pressure correspondingly small. Conversely, if the abutments move (either by spreading or rotating) and thus contribute to the release of the arch barrel, the local soil strains might be significant and, consequently, so will the changes in soil pressure.

Notwithstanding all this, it must be remembered that the backfill is confined by the spandrel walls and the extent and stiffness of that confinement will influence the longitudinal stresses or pressures that the backfill will exert on the extrados. Additionally, if the thrust causes the abutments to spread, the changes in the backfill pressures at the springing will be much larger than those resulting from pure rotation of the barrel at the springing.

Depending on the nature of the barrel material, there might

be a possibility of local crushing in the vicinity of the hinges. From a compatibility point of view, it would be logical to assume that hinging occurs at the limit of the rigid block (it being assumed that the material is either rigid or plastic). By removing the rigid material criterion, the problem becomes non-linear, but this can be overcome by introducing an iterative procedure in the algorithm.

There has been a tacit assumption that the backfill material is homogeneous – this is not the case. Not only might the extrados backfill contain or comprise internal spandrel walls and over-slabbing but also the spandrel wing walls themselves will provide significant stiffening and resistance to movement. The final surface, whether a road blacktop or a railway (or a canal, for that matter), will influence the local dispersal to the extrados – which might have a greater influence than the crushing strength of the barrel masonry. The most conservative assumption is that each axle manifests its effect as a point or line load on the extrados; this is perfectly acceptable for quick hand calculations to check computer output. The more accepted dispersal is two vertical for one horizontal, that is, the load is applied to an area of width equal to the depth to the extrados.

The influence of the spandrel and wing wall is significant – even when spandrel wall separation has occurred. This was recognised in the nineteenth century and is reflected in the design recommendations that passed to later generations (Alexander and Thomson, 1927). It was suggested that the spandrel or wing walls ‘stretch horizontally such a distance, that the friction of their base shall at least be equal to the excess of the horizontal thrust of the loaded half arch over that of the unloaded half; from two-thirds to one-half of the horizontal thrust of the arch at the crown.’

Having considered the behaviour of single-span bridges, attention will now be turned to multispan masonry arch bridges. In the UK, many of these bridges were constructed in the nineteenth century, for the railways. Bridges containing many spans often included a substantial pier at intervals to prevent the entire viaduct from collapsing in the event of one of the spans failing. This is a clear indication that our forebears had an understanding of the interaction between the spans. Nowadays, multispan masonry arch bridges can be assessed using computer-based models that automatically identify whether single or multispan failure modes are critical.

If the piers and abutments are substantial, it is likely that the critical loading relates to the carrying capacity of the weakest single span. However, it is vital that a holistic approach is taken, to identify possible failure mechanisms. If more than one span is considered, it follows that more than four releases will be needed to convert a section of the viaduct into a mechanism. For example, if two adjacent spans are assessed, seven releases will be required to create a mechanism. Large-scale laboratory tests have confirmed this (Melbourne *et al.*, 1997). Software programs now exist (Gilbert, 2001), which can deal with multi-ring brickwork arches at the same time as multi-span construction. From the limited test data available, several observations can be made.

- In the absence of robust piers, the failure mechanism involves both the loaded span and one or other of the adjacent spans. Figure 13.39 shows a typical result; note that the crown of the loaded span moves downwards, while the crown of the adjacent span moves upwards. It is the resistance to this upward movement that is significant in determining the carrying capacity of the loaded span, and hence the viaduct.
- The ultimate failure load is usually less than that for the comparable single-span arch bridge.
- The critical loading position is in the vicinity of the crown (rather than the quarter span), as this offers the greatest horizontal loading to the top of the intermediate pier and hence the greatest springing loading.
- The spandrel walls have a significant effect on the carrying capacity. In viaducts, they often represent (particularly over the piers) a significant proportion of the overall width of the bridge and can consequently (if only included as dead load) make a crucial contribution to the stability of the arch barrel.
- Ring separation can be just as significant in multi-ring brickwork multispan bridges as in single-span bridges.
- In addition, skewed multispan bridges appear to be weaker than their equivalent square span structure (for brickwork). They produce complex three-dimensional failure mechanisms, which initially encourage the barrel to carry the load square to the abutments (i.e. the shortest span). This, between adjacent spans, causes torsion to develop in the piers (and non-uniform load distribution across the width of the abutment), even with full-width uniform loading. A test (Melbourne, 1998) on a two-span multi-ring brickwork arch bridge with a 45° skew showed that although the strain and deflection contours initially indicated that non-parallel hinges were forming, owing to full-width knife-edge loading parallel to the abutments, the arch barrel articulation became more aligned to parallel hinge formation once the pier had become torsionally released. The further significance of this observation relates to the fact that, as the loading increased, the mode of response of the structure could be different, depending on the most efficient mechanism.

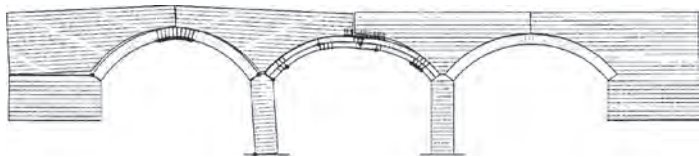


Figure 13.39 Failure in multispan bridge

Design of masonry arch bridges

In considering the design of masonry arch bridges, it is important to remember that there are over 40 000 examples in the UK alone. Most have already exceeded the Department of Transport's design life of 120 years and therefore may be considered as an archive of good practice and proportion (albeit some may be considered oversized). Traditionally, the span-to-rise ratio varies from 2:1 (semicircular) to 10:1 but is usually in

the range of 3:1 to 6:1, with the ideal taken as 4:1. Clearly, the shape of the intrados is of great importance, as this dictates the complexity of the temporary support or centring. Nowadays, this usually takes the form of a proprietary metal system of supports, on which timber transverse joints are secured and a plywood facing attached. Care must be taken to limit the centring deflections during construction. It is good practice to provide a method statement that requires the barrel to be constructed at an even rate from each springing. The centring should not be removed until the spandrel walls (if the bridge is to be a filled spandrel bridge) have been taken to the string course and the wing walls extended sufficiently to resist the horizontal thrust and also contribute to the stability of the abutment. It is important to take a holistic approach and consider the interaction between the several elements of the bridge. It should not be forgotten that small settlements or rotations of the piers and abutments offer releases to the structure and can thus contribute to the formation of a mechanism.

The initial sizing of the structure can be undertaken using any of a number of empirical equations.

Rankine advocated that the barrel thickness, d , for a segmental arch of radius R should be

$$d = 0.19\sqrt{R} \text{ (single span)} \quad (13.43)$$

$$d = 0.266\sqrt{R} \text{ (multispan)} \quad (13.44)$$

Heinzerling suggested

$$d = 0.4 + 0.025R \text{ (dressed stone)} \quad (13.45)$$

$$d = 0.4 + 0.028R \text{ (brickwork)} \quad (13.46)$$

$$d = 0.4 + 0.032R \text{ (random rubble stone)} \quad (13.47)$$

Trautwine suggested (for span L)

$$d = 1.0[0.061 + 0.138\sqrt{(R + 0.5L)}] \text{ (first-class cut stone)} \quad (13.48)$$

$$d = 1.13[0.061 + 0.138\sqrt{(R + 0.5L)}] \text{ (second-class cut stone)} \quad (13.49)$$

$$d = 1.33[0.061 + 0.138\sqrt{(R + 0.5L)}] \text{ (brickwork)} \quad (13.50)$$

Rennie and Stephenson related arch barrel thickness to span and radius, respectively:

$$d = \text{span}/30 \rightarrow \text{span}/33 \text{ (Rennie)} \quad (13.51)$$

$$d = R/26 \rightarrow R/30 \text{ (Stephenson)} \quad (13.52)$$

Historically, the addition of haunches to the arch ring at the abutments has been considered good practice (and in keeping with the usual assumption for two-hinged arch analysis that the second moment of area of the barrel varies as the secant of the tangent to the centre line).

The suggested ratio of springing to crown thickness varies from 1.2 to 2. In stonework and concrete, this is relatively easy to achieve but in brickwork the haunching is usually concrete

and there is a reliance on the bond between it and the brickwork.

Abutment sizes for a gravity solution (as opposed to a reinforced concrete or masonry solution) have been suggested by Baker (1909), where the abutment thickness, t , in metres is given by

$$t = 0.012(5L + 4H) + 0.3 \quad (13.53)$$

where L is the span (in metres) and H is the height from the top of the foundation to the springing line (in metres).

Both the haunching rules and Trautwine's rule for abutment sizes are shown in Figure 13.40.

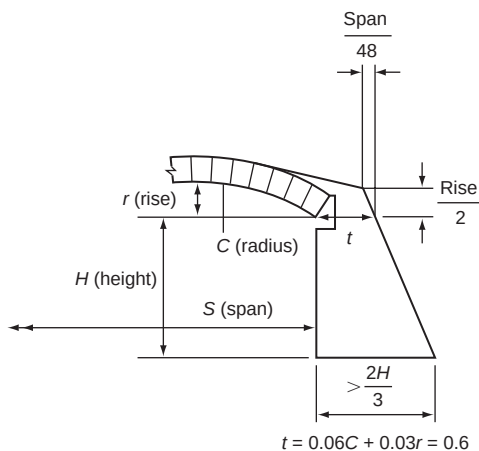


Figure 13.40 Empirical rule for abutment sizes

In the case of multispan bridges, the thicknesses of piers vary. Rankine (1904) suggests a range from 1/10 to 1/4 of the span, the latter offering sufficient support to cater for the removal of one of the spans. Historically, the most common thickness for intermediate piers is from 1/6 to 1/7 of the span. These suggested thicknesses make no reference to the height of the pier, which is an important parameter when considering out-of-balance forces between adjacent spans that have to be accommodated by the pier. Of equal importance, then, is the slenderness ratio of the pier. Recent laboratory tests have demonstrated that, even with a slenderness (height/thickness) ratio of 3.4, the failure mechanism involves at least two spans. In any event, irrespective of pier geometry, a range of potential single and multispan failure modes should be checked for any structure.

Modern limit state codes require both serviceability and ultimate limit states to be considered. In the case of the masonry arch, serviceability criteria are difficult to quantify. Generally, serviceability criteria limit deflection, for aesthetic or functional reasons, and cracking, for durability considerations. For masonry arches, the stress levels are generally very small; hence deformations, are small. Additionally, any cracking that might occur in response to decentring and bedding-in of the structure is usually only an indication that it is working and should cause no concern. Conversely, it is important that the structure is designed to avoid proliferation of microcracking under repeated loading (fatigue), particularly if this could lead

to ring separation in multi-ring brickwork arch barrels. It has been observed in field and laboratory tests that the first hinge forms at approximately half the ultimate load-carrying capacity of each bridge. This observation has been used to inform the factors used in current assessment codes, including the live load capacity factor used in CS 454 (Highways England, 2020). Traffic loadings should be applied in conjunction with a partial load factor for all the dead load and superimposed dead load of 1.2, when they are adversely affecting the structure, or 1.0, when they are resisting hinge formation. The initial design procedure is as follows.

1. Choose span and rise.
2. Select materials to be used.
3. Determine trial section using a selection of empirical equations.
4. Ignoring horizontal soil pressures, calculate the required arch barrel thickness using a simple block mechanism for the ultimate load condition. This can be assumed to comprise appropriately factored vehicle loadings.
5. Check the compressive stress based on $0.1 \times (\text{arch thickness})$ or 100 mm, whichever is the greater:

$$\text{Compressive stress} \leq \lambda_u f \quad (13.54)$$

where λ_u is a coefficient, equal to 0.35 for concrete grades 15 and 20, 0.4 for concrete grades 25 and above and 0.44 for masonry and f is the characteristic cube strength of concrete, f_{cu} , or the compressive strength of masonry f_k , as appropriate (allowance for γ_M has been made).

6. Check that the radial shear at the crown is less than 0.4 (horizontal reactions).
7. In the case of multi-ring brickwork arches, check that the horizontal mid-depth shear stress at the crown is less than 0.15 N/mm^2 .
8. Check abutment stability and stress levels.
9. Check foundation stability and stress levels.

Example

Consider an 8 m span, 2 m rise segmental brickwork arch bridge with a total available construction depth of 0.9 m at the crown. Follow the initial design procedure.

1. Span = 8 m; rise = 2 m; intrados radius = 5 m.
2. Materials to be used: brickwork.
3. Empirical equations:

$$\text{Rankine: } d = 0.19\sqrt{R} = 0.19\sqrt{5} = 0.42 \text{ m} \quad (13.55)$$

$$\text{Heinzerling: } d = 0.4 + 0.028R = 0.54 \text{ m} \quad (13.56)$$

$$\begin{aligned} \text{Trautwine: } d &= 1.33[0.061 + 0.138\sqrt{(R + 0.5L)}] \\ &= 1.33[0.061 + 0.138\sqrt{(5 + 4)}] \\ &= 0.632 \text{ m} \end{aligned} \quad (13.57)$$

It is therefore likely that the arch ring thickness will be about 0.5 m – it follows that a construction depth of 0.9 m is probably sufficient.

4. A simple block mechanism analysis using a 1 m slice of the bridge can be undertaken to determine the barrel thickness. Here a minimum lane width of 2.5 m is used with a composite load factor of 3.4 for the knife-edge load (KEL) and 1.9 for the uniformly distributed load (UDL) elements of an HA loading configuration (these values being taken from the now withdrawn BD21/01 assessment code (Highways Agency, 2001), though alternative, less conservative, values could be instead chosen).

$$KEL = (3.4 \times 120) / 2.5 = 163.2 \text{ kN/m width} \quad (13.58)$$

$$UDL = 1.9 \times (336 / 2.5) (1 / \text{load length} = 1/4 \text{ m})^{0.67} = 100.9 \text{ kN/m run} \quad (13.59)$$

The hinges are assumed to occur at A, B, C and D. It is also assumed that the soil offers no horizontal support and that γ_{FL} for the self-weight on the loaded side of the arch is 1.2, while on the unloaded side of the arch it is assumed to be unity, as shown in Figure 13.41.

By taking moments about C, B and A for the free bodies to the right of each of the hinges, three independent equations can be written and solved for the unknowns V_D , H and t :

$$t = 0.373 \text{ m} \quad (13.60)$$

$$H = 447 \text{ kN} \quad (13.61)$$

$$V_D = 231 \text{ kN} \quad (13.62)$$

Making allowance for the potential loss of mortar depth to the intrados and rounding to the nearest half brick – assume a barrel thickness of 450 mm.

5. At the crown, the compressive force H is 447 kN, so the characteristic compressive strength required is

$$(447 \times 10^3) / (0.44 \times 100 \times 10^3) = 10.2 \text{ N/mm}^2 \quad (13.63)$$

6. At the crown, the vertical shear is equal to

$$V_D - (\text{self-weight to the right of C}) = 231 - 115.5 = 115.5 \text{ kN} \quad (13.64)$$

(this is less than $0.4H = 0.4 \times 447 = 178.8 \text{ kN}$).

7. At the mid-depth of the barrel at the crown:

$$\text{longitudinal shear stress} = (\text{vertical shear} / I(\text{width})) A \bar{y} \quad (13.65)$$

$$\frac{((115.5 \times 10^3) \times 1000 \times 225 \times 112.5)}{(((1000 \times 450^3) / 12) \times 1000)} = 0.39 \text{ N/mm}^2 > 0.15 \text{ N/mm}^2 \quad (13.66)$$

Headers should be provided or some form of mechanical bond introduced. If neither of these is practical, the arch should be thickened to reduce the stress. Additionally, the longitudinal shear stress should be checked at the interface between the brickwork rings as the thrust line moves from one ring to the next, for example, between A and B.

8. If the abutment height at the springing to foundation is 3 m and the base is 2 m thick (i.e. 2/3 height), it can be shown that the abutment is unstable and will rotate about the rear

face. The stabilising effects of the wing walls may be considered at this stage to check that stability is achievable.

9. The foundation stability and stress levels would then be checked.

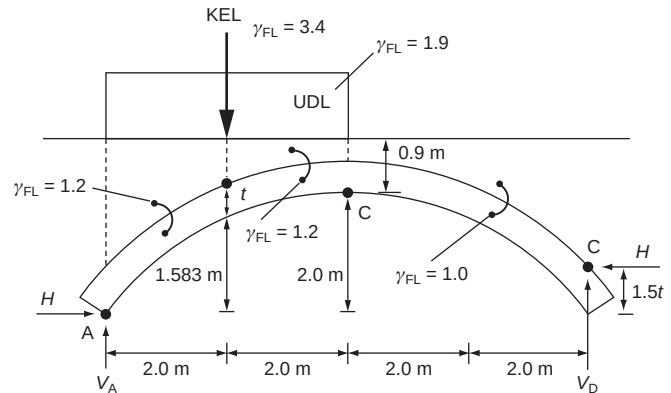


Figure 13.41 Preliminary design example

Having determined the initial proportions, some consideration must be given to the practicalities of construction and the specification, prior to undertaking a more detailed analysis for the different load cases using a more accurate analytical model based on a realistic representation of the material properties, including foundations and soil properties.

Depending on the sophistication of the analysis, the output may indicate the stress levels that can be compared at the critical section with the allowable stress. For simpler analysis using a frame analysis program, the nodal forces must be assessed using the standard equation:

$$\sigma = P/A \pm M_y/I \quad (13.67)$$

where stress, σ , at distance y from the neutral axis

$$= \left(\frac{\text{axial force}}{\text{cross-sectional area}} \right) \pm \left(\frac{\text{bending moment} \times y}{\text{second moment of area, } I} \right) \quad (13.68)$$

If tensile stresses are induced, then that part of the section should be ignored and the analysis iterated until the designer is satisfied that all tensile stresses have been eliminated.

The design of spandrel walls, wing walls and parapets requires careful consideration as, historically, it is recognised that they usually show signs of deterioration first. Spandrel and wing walls may be designed as gravity structures. Tensile stresses should be avoided. It is important that these are checked at each section change and at regular heights up the wall. Sliding along bed joints should be checked. The interaction between the arch barrel and the spandrel wall should be designed to minimise the risk of spandrel wall separation.

The forces on the walls may be reduced by using reinforced soil or foamed concrete as backfill. The walls may be connected to the soil reinforcement. It is preferable to ensure that the backfill is less stiff than the barrel to ensure that the arch behaves as a particulate structure and can articulate in response to movement without creating hard (or soft) spots of interaction with the backfill.

Parapets should be designed to contain vehicles. To comply with the current design standards (Highways England, 2021), it is inevitable that either a conventional parapet is provided or some form of masonry grouted cavity wall must be adopted. The cavity reinforcement and ties should be austenitic stainless steel. The reinforcement should be secured to a torsion beam or slab that spans between abutments. Research (DfT, 2012) has shown that unreinforced parapets have a greater containment capacity than had previously been thought. The impact resistance of plain masonry parapets depends on two principal factors. These are the internal forces, which are governed by wall mass and geometry, and the bond conditions between the masonry units forming the parapet. Figure 13.42 shows how the unit–mortar bond conditions can influence the mode of response. It is unlikely that a simple equivalent static load will predict the actual behaviour; only a dynamic analysis can do this. However, it is reassuring that mortar-bonded masonry parapets with a thickness of 400 mm or more and a minimum length of 10 m have the capacity to contain a 1.5 t vehicle, travelling at 60 miles per hour (100 km/h) impacting at an angle of 20°, irrespective of mortar strength (Middleton, 1995).

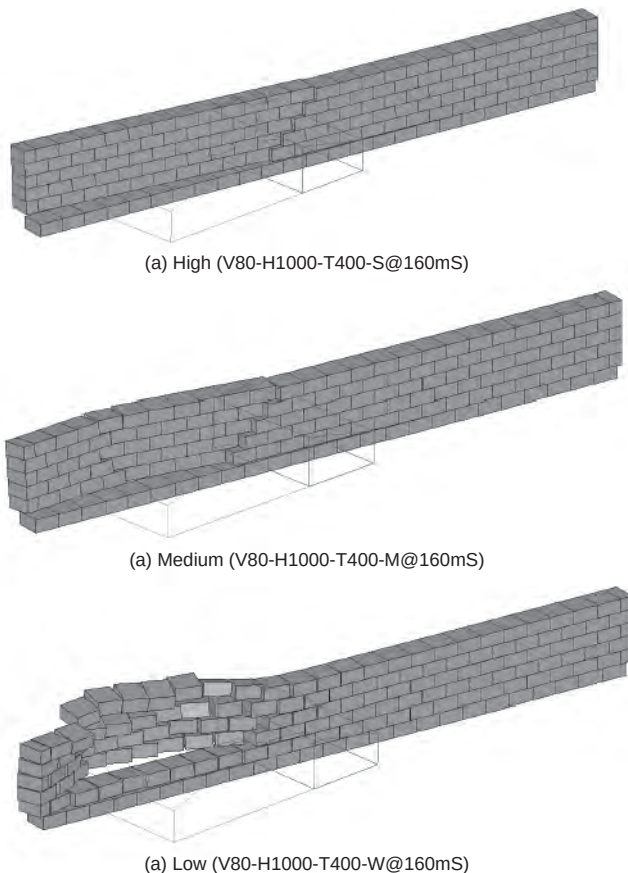


Figure 13.42 Masonry parapet failure modes following vehicle impact for (a) strong, (b) medium and (c) low unit–mortar adhesion (DfT, 2012)

Specification

Unreinforced arches may comprise brickwork, stone, reconstituted stone, precast and in situ concrete. Additionally, there are proprietary systems, which would need independent certification by the Bridge Authority.

Class A or B engineering brick with a durability designation of F2/S2 complying with BS EN 771-1 (BSI, 2011) should be used. Calcium silicate bricks are not recommended. Concrete bricks and precast blocks should have a minimum characteristic strength of 30 N/mm² with a minimum cement content of 350 kg/m³ and be manufactured using dense natural aggregates. Reconstituted stone blocks should be independently certified.

Natural stone should comply with BS EN 771-6 (BSI, 2015). Stone should be selected on the basis of proven durability and weather resistance.

Mortar for masonry should be in accordance with Table 13.8. Mortars based on Portland cement will be stronger and more durable, but less accommodating of movement than lime mortars.

Location or element	Mortar designation	
	BS 4721	BS EN 998-2
Work below or within 150 mm of finished ground level	(i) or (ii)	12 or 6
Work 150 mm or above finished ground level:		
abutments, spandrel/wing walls, piers and parapets	(i) or (ii)	12 or 6
unreinforced arch ring	(iii)	4
reinforced/prestressed brickwork	(i) or (ii)	12 or 6
1. Traditional BS4721 mortar designations correspond to proportions by volume of Portland cement:hydrated lime:sand as follows: (i) 1:0–1.4:3 (ii) 1:2:4–42 (iii) 1:1:5–6. 2. Alternative mortar mixes, such as cement:sand with plasticiser may be suitable for some uses (see BS EN 998-2). 3. Where category S1 (or FN) classification bricks are used or when sulphates are naturally present in soil or groundwater in sufficient quantities to be damaging the sulphate-resisting Portland cement based mortars should be used.		

Table 13.8 Mortars

Clay bricks should not be used within 14 days of firing, in order for the irreversible expansion owing to moisture take-up to be complete. Masonry in the arch should be flush-jointed. Bucket-handle or other finishes may be considered for spandrel and wing-walled masonry. It is difficult to achieve continuous flush mortar joints in the intrados, in which case a neoprene rubber strip may be inserted during brickwork construction. This can be removed after the centring has been struck and the joint gun pointed using similar mortar.

The timing for removal of the centring should be such that the barrel is not required to carry stresses of $\frac{1}{3}$ of its strength at the time of loading and not within 7 days of completion of the arch. Historically, it is advised that the spandrel walls should be taken up to the string course prior to removal of the centring (Alexander and Thomson, 1927).

Concrete should be designed using 20 or 40 mm durable natural aggregates with a minimum cement content of 325 or 295 kg/m³, respectively, and a minimum free water/cement ratio of 0.5. The concrete should be air-entrained to guard against freeze–thaw action (4.5% for 40 mm aggregate concrete and 5.5% for 20 mm aggregate concrete). To reduce shrinkage, 40 mm aggregate concrete is preferred. The concrete should be well compacted and cured. Specialist cements may be used where ground conditions or climate demand, or where shrinkage is to be minimised.

Care should be exercised to accommodate movement. This is equally true for all materials and should include allowances for

- seasonal thermal movements
- movement of the foundation
- creep of highly stressed elements (e.g. prestressed masonry)
- relative stiffness
- long-term irreversible expansion of clay brickwork owing to moisture
- seasonal moisture movement (usually very small).

Joints in brickwork are usually required at 10–15 m centres. It is important that if joints are provided that they are able to work. All too often, the three-dimensional nature of the structure is overlooked and restraints are present; this prevents the efficient functioning of the joint.

In situ concrete arch barrels have been constructed using planar crack inducers to ‘convert’ what would otherwise be a brittle elastic continuum into a series of in situ voussoirs. The crack inducers may take the form of plastic sheets in which holes have been cut to ensure good load transfer and prevention of slip (Melbourne and Njumbe, 1998). It is important to have a continuous strip at the intrados (which should be filleted) to prevent stress concentration and the opportunity for local spalling (Figure 13.43).

Attention should be paid to brickwork bonding in the arch ring. Individual rings may be built in English or Flemish bond but where this results in no headers the desired monolithic behaviour could be affected by ring separation. The bonding should therefore accommodate some bonding between rings at every fourth course. This may be a header or some form of durable mechanical connection. Bricks (units) can be manufactured with a taper that matches the arch curvature. It is preferable to use larger mortar joints and accept a smaller compressive strength than to provide a multi-ring arch without headers. Concrete haunching may be provided.

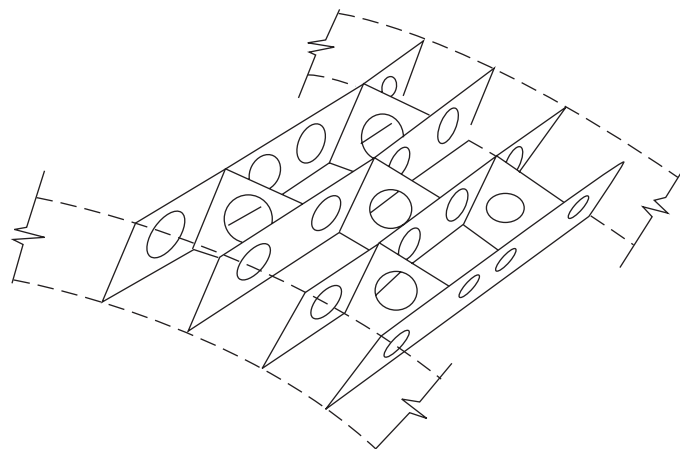


Figure 13.43 Planar inclusions

Alternatively, the arch may be divided by voussoir-shaped blocks. These may act as ring separation arrestors but are no substitute for the provision of headers throughout.

The extrados and all buried structures in contact with the backfill should be protected by a waterproofing system that is suitably protected – preferably by a permeable layer.

It is vital to the long life of the bridge that any water that enters the backfill is efficiently removed by means of an appropriate drainage system.

Defects and rehabilitation

Most masonry arches are over 100 years old, have survived the vicissitudes of the UK climate and continue to carry loads very different from those envisaged by their constructors. It is inevitable that such old structures often show signs of distress, the causes of which can usually be attributed to damage to the substructure, changes in loading regime or material deterioration.

Unlike trabeate construction, arches impose significant horizontal thrusts onto the abutments and piers. These may result in horizontal movements, which cause the masonry barrel to crack. The situation is further complicated by non-uniform distribution of the thrust, with the consequence that the abutments and piers are subjected to torsion across the width of the bridge. (This might be brought about by eccentric loading of the span, or by skew.)

As with most structures, the preferred founding material is rock. Unfortunately, this is not always possible; consequently many bridges are founded on ‘soil’.

Historically, timber piles, grillages or faggots have been used to improve support conditions. Bridges that were given inadequate foundations have long since collapsed, so bridges that have developed defects in recent times must have done so as a result of changes. It is not doubted that well-constructed masonry is one of the most durable materials. However, there are conditions that accelerate material deterioration and lead to problems, the main agent being water. In new construction, low-porosity bricks should be used. In existing structures,

the bricks might show signs of failure, for example, spalling, cracking or friability. This might be local, in which case there may be a local cause that can be remedied, or it might be more general and thus require a more drastic solution.

The two consequences of water in masonry are the aggravation of freeze–thaw effects and the enhancement of chemical activity. The former is a particular problem in temperate climates, owing to the frequency of freeze–thaw cycles during winter months (of which there may be several in a single 24 h period). As water freezes, it expands, so if the water is present near the surface of the masonry, incremental deterioration may take place. It is therefore important that water should be excluded from the bridge. Although this may be possible in the case of highway bridges, where the highway construction is impervious, it might not be possible in the case of railway bridges, as the installation of a waterproofing membrane might be prohibitively expensive. Historically, puddle clay was used as a waterproofing membrane laid directly onto the extrados. (This should be remembered when modelling the backfill material properties of soil pressure profiles.)

In addition to aggravating the freeze–thaw effects, water can transport sulphates (and chlorides) from the backfill, de-icing salts or the masonry itself; the effects of this are not only to cause mortar washout but also to facilitate chemical attack and deterioration of the bricks or stone and mortar. The mortar is usually more vulnerable than the units. Certainly, the appearance of a well-pointed intrados should not lull the assessing engineer into a false sense of security, as the underlying mortar might be weak, friable or perhaps non-existent. Additionally, if the pointing has been undertaken using a modern cement based mortar, this is likely to produce hard spots in the intrados, which will aggravate stress concentrations and freeze–thaw effects, potentially causing spalling. Under no circumstances should an impervious membrane be applied to the intrados – the masonry must be allowed to breathe. In any case, any repointing of the masonry must be undertaken using a mortar compatible with that in the original structure – which is probably a lime mortar.

It should be remembered that bricks fresh from the kiln expand irreversibly (this is often referred to as moisture expansion). This can be a very beneficial phenomenon when patch repairing, as the slight expansion can lock the patchwork into the original structure and thus enable the new brickwork to carry some dead load as well as the live loading. Additionally, as the historical brickwork will be constructed using lime mortar, it will be quite accommodating of the patchwork and, finally, the creep characteristic and natural particulate nature of the brickwork will allow stress concentrations to be relieved by redistribution. Some attempt should be made to achieve compatibility between the old and new brickwork with regard to Young modulus, coefficient of thermal expansion, and so on.

If colour matching is an important issue, enough of the existing structure should be cleaned to establish brick and mortar colour before the repairs are undertaken and trial panels made to ensure a good match.

The deterioration of stonework is related primarily to the chemical nature of the material and its physical condition, in

particular, its porosity. Notwithstanding the natural variability of stone, the natural durability of stone depends on the susceptibility of the stone's constituents to acid attack from the carbonic, sulphurous or nitrous acid rain that results from the carbon dioxide, sulphur dioxide and nitrogen oxides in the atmosphere. The subsequent leaching that takes place increases the porosity of the stone, while reducing its cohesiveness.

Leaching is more severe in stone with fine pores as this increases the capillarity and hence the ease with which moisture can be held by surface tension. (This also holds for susceptibility to freeze–thaw effects.) Porosity can be measured using the saturation coefficient, which is determined by comparing the amount of water absorbed naturally with that resulting from vacuum impregnation.

It should be remembered when replacing defective stone that it may have taken well in excess of 100 years for the stone to deteriorate and therefore the stone may be perfectly adequate. Additionally, the chemical and physical state of the original stone should be carefully established to ensure that any replacement stone does not interact unfavourably with it. Particular care should be taken with regard to bedding planes and their orientation to exposed faces and setting (whenever possible) perpendicular to the principal stress. Any damp surface of the masonry will also be a breeding ground for bacteria, which actively produce sulphates and nitrates. Additionally, algae will also develop in a few hours, given suitable conditions of dampness, warmth and light. Temporary dryness may cause algal death; however, the resulting humus offers a suitable host for lichen, whose fungal hyphae penetrate the stone, generating organic acids. On dark, damp surfaces, fungal slime might develop. Having established that the physical, chemical and biological state of the stone are all significant in ensuring its durability, it is clear that great care must be taken when repairing or cleaning the masonry.

It is important to keep the structure as dry as possible. High-pressure water jetting can achieve the best results under normal circumstances. Biocidal treatment should be considered after cleaning to control bacterial or fungal attack. Under no circumstances should surface sealants be used, particularly if water penetration is expected from the buried or rear face (i.e. the extrados of the arch or the back of a retaining wall).

It was generally accepted that some movement would take place during the construction of a masonry arch bridge and that the construction sequence is very important in limiting and controlling any movement: 'The centre of an arch is always struck when the superstructure is only partly built, else the superstructure would crack, due to the after-subsidence, [...] the earth must be filled behind the spandrels before the remainder of the superstructure is built' (Alexander and Thomson, 1927).

Given that most masonry arch bridges are old structures, it may be reasonable to assume that all dead load settlement will have taken place. If the structure is showing signs of distress associated with foundation movement, serious thought should be given to the reasons. Founding strata might not be uniform, and this might have resulted in differential settlement or spread of the abutments, with consequent distortion of the spandrel

walls and arch barrel, the effect of which, with the passage of time, might have been cracking and load path adjustment. This might have aggravated the condition, causing further deterioration. Add to this the prevailing climate, for example, prolonged rain or drought, coupled with long-term deterioration of the fabric of the bridge, and it is possible to see how defects can worsen.

The effect of spreading or differential settlement is to cause cracking in the spandrel walls and (more particularly) arch barrel, with a consequent reduction in carrying capacity – albeit empirically assessed (Table 13.7).

Figure 13.44 shows the different modes of spandrel wall failure. The lateral soil pressures and the interaction between the barrel and the spandrel walls are important. With heavier traffic, which often runs close to the spandrel walls, many filled masonry arch bridges show signs of distress. The longitudinally stiff spandrel wall and more flexible arch barrel crack at the interface and separate. The precise mechanism governing this type of defect is not well understood. It is certainly aggravated by the ingress of water from the fill above, which washes out the mortar and causes further distress if it freezes in the cracks.

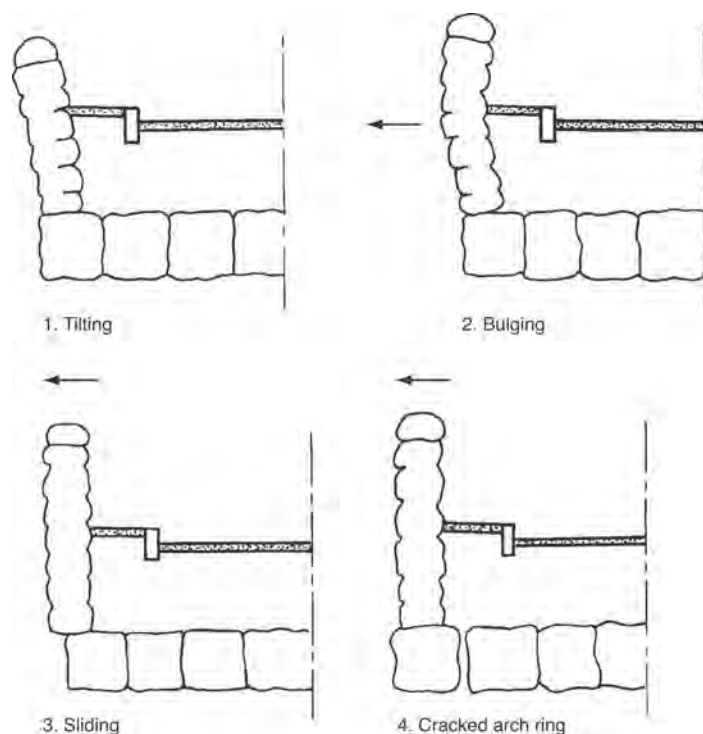


Figure 13.44 Spandrel wall failure (Page, 1993).

One significant problem that is specific to brickwork arch bridges is ring separation. This occurs in multi-ring arch barrels where the mechanical bond between adjacent rings of brickwork is lost, with a resulting loss in load-carrying capacity. For example, the shear bond strength for a 1:2:9 lime mortar would normally be about 0.2 N/mm^2 , though even this will not be achieved if there is deterioration or the quality of the work was poor.

It is very unlikely that the backfill over the arch will fail in the structural sense. This is usually wet, independent of whether or not the road construction is supposed to be impervious or the backfill to be provided with drainage. Notwithstanding the effect that water has on the properties of the backfill, the presence of water saturating the masonry, washing out the mortar and generally causing long-term deterioration of the fabric of the bridge is undesirable.

Although any change in vertical alignment of a rail track is routinely adjusted, the same cannot be said of the maintenance of road surfaces, which frequently develop potholes. These can increase the dynamic effects of tyres hitting the road surface and thus accelerate the deterioration of the bridge.

This is aggravated by the heavier vehicles that are now using our roads and the increase in the volume of such traffic. Having considered the range of defects that masonry arch bridges develop and their causes, it is essential to review the range of repair and strengthening techniques. Table 13.9 presents a list of common faults and their associated repair or strengthening techniques. See CIRIA C656 (McKibbins *et al.*, 2006) for further details regarding repair and strengthening techniques. It is essential that the cause of the deterioration is understood and the effects of any strengthening or repair techniques are considered before beginning any work on the bridge. At all times, it is important to consider that masonry arch bridges derive their strength and tolerance to movement from their ability to articulate – it is their particulate nature that gives them their unique structural characteristics. If this is lost then they behave in a different way that should be taken into account when considering their strength and residual life.

If remedial or strengthening works are selected without due consideration and understanding of their short and long-term effects, they might result in more harm than good and can address one failure mode only to allow, or even cause, another. Particular care is required when work is necessary to one span of a multispan bridge.

Sometimes repair and strengthening options are limited or their execution complicated by the presence of previous works on the bridge. The response of the bridge to past works and their success can provide useful information for assessing the potential effects of the proposed works and their chances of success.

The MEXE method of assessment quantifies the effect of loss of mortar, with the implication that reinstatement of the lost material will reinstate load-carrying capacity. Although this is the desired outcome, care must be taken in undertaking the work.

First, it is important to determine the extent of the material loss and the nature of the original mortar. (This is because the intrados may have been repointed several times during its life.) It is particularly important not to use a mortar that is too stiff, as this could lead to local splitting of the units and spalling.

Additionally, care should be taken not to remove so much material that the units become loose. This might be exacerbated by poor initial workmanship, which might have left incompletely filled joints, hence producing voids that have acted as a

conduit for water to flow through the arch barrel.

The most effective way of removing the mortar is by jetting. A skilled and conscientious workforce is essential.

Hand pointing may be acceptable for small areas but repointing can only be effective if it is consistent and complete. This is best achieved using some form of mechanical pointing. A pump hole mortar mix, usually containing pulverised fuel ash (PFA) and non-ionic wetting agent, is used and delivered through a nozzle at a pressure of 0.1–0.3 N/mm². The joints are filled from the inner voids outward. It should be remembered that, in removing the friable mortar, the units are vulnerable to being dislodged. Depending on the viscosity of the mortar and the rate of stiffening, considerable forces can be exerted between the units. For example, a brick presenting a full face of 215×104 mm, if pointed to full depth at 0.2 N/mm² would have an out-of-balance force of 44.7 kN, which would probably be greater than the shearing resistance available at the mortar bed between the rings. The result could be delamination of the bottom ring, ring separation and hence a reduction in load-carrying capacity. The residual compressive pressure helps the inner part of the arch barrel to play an active role in carrying the self-weight of the structure. Creep allows long-term redistribution of hard-spot stresses.

In situations where running water or surface water is present, quick-set mortar can be used. Care should be taken in planning the work to ensure that the future presence of the water is taken into account, so drainage points should be incorporated into the joints to allow it to escape. Otherwise, by sealing the intrados, untold damage could be done to the rest of the arch barrel.

If it is felt that significant amounts of mortar have been lost from within the arch, repointing will not be sufficient to rehabilitate the structure. In such cases, it is usual to offer grouting as a viable option. Pressure or vacuum grouting are the usual methods adopted. Selection of grout material is important. In the past, hydraulic limes and pozzolans have been used but nowadays thixotropic grouts and low-viscosity grouts are available. As with mechanical pointing, the effect of the internally induced stress must be considered and the grouting sequence adjusted accordingly. Suitable arrangement must be made for the displaced air and water to escape as the grout is injected (otherwise voids are left and brick pressure forces the grout back out). It is inevitable that some grout escapes; this should be cleaned off immediately. It is unlikely that pre-pointing will be able to contain the grout. At best, there will be some leakage; at worst, the pointing will be blown out. Grouting usually proceeds from the lowest to the highest point. If it is judged that the pressure required to force the grout into small voids or fissures would cause damage, vacuum injections may be the solution. A vacuum is first established within the section to be treated. Grout is then introduced and sucked into the voids to balance the vacuum. A low-viscosity grout is usually used. The method for the arch barrel can only be used if the extrados of the barrel is in good condition and can provide an effective seal when the vacuum is established.

It is important to realise that the purpose of the grout is to reinstate a pliable glue between the units. If a rigid material is

used, the arch barrel will become a brittle continuum, highly redundant and susceptible to secondary stresses.

Where it is felt that the arch barrel is unable to carry the required loading, one method of rehabilitating the bridge is to provide support from underneath. This is only possible if the resultant loss of headroom and the change in appearance is acceptable. It may be achieved by spraying concrete onto the intrados or by providing a structural liner. In both cases, it is vital that adequate provision is made for any water to escape after installation.

Whichever method is used, it is vital that adequate foundations are available. They should be checked for the change in load path. Clearly, if the distress in the arch barrel has been brought about by movement of the abutments, providing an underarch may actually aggravate the situation rather than ease it.

Sprayed concrete (gunite, in the UK or shotcrete in the USA) has been used in many situations. It is particularly suited to providing a relieving arch. The concrete is sprayed at high velocity onto the prepared intrados using either a dry or wet process. It is usual to provide a grid of pins to which is attached a steel rebar mesh. The pins and mesh also act as a guide to thickness. The maximum aggregate size is usually 10 mm (although 20 mm can be accommodated). Both processes can produce dense good-quality concrete (typically 30–50 N/mm² 28 day strength), with a satisfactory bond to a prepared intrados. A composite process has been developed in Hungary, which is claimed to give better control on concrete quality, as well as a reduction in rebound loss.

Admixtures can be used to reduce shrinkage cracking, although a survey by Ashurst (1992) reported visible cracking in all the bridges surveyed.

To reduce visible impact, the edge of the relieving arch may be recessed (into the shadows) or cosmetically treated to blend with the bridge elevation.

An alternative relieving arch can be provided using a series of curved universal beams or columns bent about their major axes to fit the intrados of the arch. It is usually necessary to provide needles to support the steel ribs. If the headroom is sufficient, permanent shutters in the form of corrugated steel sheeting or glass reinforced plastic (GRP) may be used to provide an annulus that can be filled with pumped concrete.

Determining the distribution of load between the original arch and the relieving arch is somewhat circumspect, so it is best to assume that the relieving arch will eventually be carrying all the load.

Recently, techniques using near surface reinforcement have been developed. These involve the use of steel reinforcement or steel strip. The former is installed in purpose-cut grooves using compatible adhesive, while the latter can be bolted to the intrados (the length of the bolts can be such that they penetrate to within 100 mm of the extrados). The analysis of the strengthened arch barrel can be undertaken using finite-element software (Sumon, 2000). Care must be taken when modelling the interfaces between the reinforcement and the adhesive and between the adhesive and the masonry. As discussed earlier,

Technique	Category	Structural defect and location	Engineering aspects	Advantages and disadvantages
Arch distortion remedial works	Remedial	Inadequate overall load-carrying capacity of arch barrel Distortion, misalignment or tilting of regular shape of arch barrel	Establish causes of distortion. Lining support should be designed to be either composite with the existing bridge or offer independent support to it. This must be clearly recorded in the bridge records. The foundations should be checked for competence.	Advantages Ease of application or installation Possible enhanced live load Disadvantages Potential bond problems Appearance and clearance Measures for protected species (if present)
Arch grouting	Remedial	Arch ring separation Fractures and extensive cracking	It is recommended that the grouting holes are stopped at least 100mm short of the extrados. If no radial pins are installed, the new load-carrying capacity should be determined assuming full ring separation of the top ring with due regard to the shear capacity of the inter-ring mortar. If radial pins are installed, the shear capacity should be checked with due regard to other possible modes of failure. The brickwork should be checked for the new loading regime.	Advantages Simple established methods Structure life expectancy extended Disadvantages Traffic disruption during construction Further inspection or testing required to confirm repair effectiveness
Backfill replacement	Remedial	Incompetent existing fill material or inadequate overall load-carrying capacity	Check stability of the structure when the backfill has been removed (arch may require temporary support). Method statement should take removal of the backfill into account. Determination of new carrying capacity should take into account the effects of the new backfill, particularly in terms of the load dispersal and lateral support to the structure. Consider change in loading on the spandrel walls.	Advantages Possible enhanced live load capacity Relative cost Disadvantages Traffic disruption during construction Structure life expectancy unaffected by works Further maintenance works may be required
Concrete saddle	Strengthening	Inadequate overall load-carrying capacity of arch barrel in conjunction with spandrel wall and waterproofing failures	The adequacy of the existing structure should be checked to ensure that it is capable of sustaining the enhanced loading with the saddle in place. Decide whether the saddle is to act compositely with the existing structure or not. Check the structural interaction, bearing in mind that the barrel is particulate and heterogeneous while the reinforced concrete saddle is not. Particular attention should be given to the load paths through the abutments, piers and pier foundations. Ensure adequate support of structural elements during saddling.	Advantages No change to appearance as hidden Facilitates other repairs, parapet upgrades or waterproofing Enhanced live load capacity Disadvantages Traffic disruption during construction Relative cost Increase in crown depth
Parapet upgrading	Strengthening	Inadequate impact resistance of parapets	The consequence of impact should be given detailed consideration, particularly in respect of vehicular containment and falling material. Where the parapet is a new build, its interaction with the existing structure should be checked.	Advantages Enhanced vehicle containment or reduction in likelihood of falling material Disadvantages Traffic disruption during construction Provision of access required Relative cost

Table 13.9 Considerations for common remedial and strengthening measures

Technique	Category	Structural defect and location	Engineering aspects	Advantages and disadvantages
Repointing	Preventative or remedial	Deterioration of mortar joints	Careful consideration should be given to the selection of the repair materials. These should be compatible, particularly with respect to stiffness and thermal characteristics.	Advantages - Simple established methods - Structure life expectancy extended Disadvantages - Labour-intensive construction - Possible traffic disruption during construction - Possible provision of access required
Patch repair	Remedial	Bulging or loose masonry or brickwork Heavily spalled brickwork	Careful consideration should be given to the selection of the repair materials. These should be compatible, particularly with respect to stiffness and thermal characteristics. Initially, the patched masonry will carry very little load, so the load-carrying capacity should be based on the capacity of the original barrel – over time, this may redistribute as a result of creep and other movement. (Removal of bulging material etc. could lead to overstressing and fatigue damage to the remaining masonry).	Advantages - Simple established methods - Structure life expectancy extended Disadvantages - Labour-intensive construction - Works phasing required - Possible traffic disruption during construction - Possible provision of access required
Prefabricated liners	Strengthening	Inadequate overall live load-carrying capacity of arch or abutments where depth of fill over arch barrel is excessive Spandrel wall and waterproofing failures	The nature and condition of the backfill and any services should be determined. Determine the extent of the spread of the loading and its effect on the load-carrying capacity. Attention should be given to the change in load paths, which might increase the load effects on spandrel walls, substructure and foundations.	Advantages - No change to appearance as hidden - Facilitates other repairs - Enhanced live load capacity Disadvantages - Traffic disruption during construction - Relative cost - Increase in crown depth
Relieving slab	Strengthening	Incompetent existing fill material or inadequate overall load-carrying capacity	The adequacy of the existing structure should be checked to ensure that it is capable of sustaining the enhanced loading with the relieving slab in place. Check the structural interaction, bearing in mind that the barrel is particulate and heterogeneous while the relieving slab is not. Particular attention should be given to the load paths through the abutments, piers and pier foundations.	Advantages - No change to external appearance - Enhanced live load capacity Disadvantages - Traffic disruption during construction - Relative cost - Increase in crown depth possible
Retro-reinforcement	Strengthening	Inadequate overall load-carrying capacity of arch barrel	It should be recognised by the designer and bridge owner that the use of a retrofit solution changes the nature and behaviour of the barrel from that of a particulate heterogeneous material to that of reinforced masonry. Particular care should be exercised to ensure that the bond between the masonry, adhesive and reinforcement can be relied on.	Advantages - Repairs hidden - Much less disruption than saddle or slab reconstruction - Relative cost - Speed of implementation

Table 13.9 Continued

Technique	Category	Structural defect and location	Engineering aspects	Advantages and disadvantages
Retro-reinforcement (continued)	Strengthening	Inadequate overall load-carrying capacity of arch barrel	In the case of multi-ring brickwork barrels, the possibility of ring separation should be investigated, owing to the change in structural behaviour accompanying the introduction of the reinforcement. (Radial pinning is recommended to ensure through-thickness continuity.) It should be noted that unreinforced areas still possess little, if any, tensile strength.	Disadvantages Independent verification or validation of analysis, design, installation, fatigue and durability of systems May not enhance performance under service loads Likely to be irreversible
Spandrel tie-bars and pattress plates	Preventative or remedial	Bulging or displaced spandrels Inadequate lateral load capacity	The long-term effect of the introduction of a stiff local restraint on the spandrel wall stability should be investigated. It should be noted that tie-bars have a negligible strengthening effect on the barrel serviceability performance as they only affect long-term stability and soil–structure interaction as the ultimate state is approached (Melbourne <i>et al.</i> , 1995).	Advantages Simple Minimal traffic disruption Relative cost Disadvantages Specialist subcontractor Service avoidance Localised high stresses to spandrel at plates Possible water paths into bridge Change in structure appearance
Sprayed concrete liner	Strengthening	Inadequate overall carrying capacity	Changes the nature and behaviour of the arch barrel, which may or may not act integrally with the spray concrete liner. In the case of multi-ring arches, radial pinning should be used to guard against ring separation. The impermeable liner may affect the internal moist movement in the arch. Load paths and foundation competence should be checked.	Advantages Little disruption to traffic flow over the bridge Enhance load-carrying capacity Reinforcement can be incorporated Disadvantages Alter appearance Reduces opening under the bridge Cannot inspect condition of original arch barrel
Stratford method (spandrel strengthening)	Strengthening or remedial	Bulging or displaced spandrels Inadequate lateral load capacity	The change in structural behaviour of the strengthened spandrel wall should be investigated. The effect of any potential hard-spot loading onto the barrel should be investigated.	Advantages Non-specialist implementation Can accommodate parapet strengthening Disadvantages Traffic disruption during construction Relative cost Service avoidance or diversion Not widely used Change in structure appearance

Table 13.9 Continued

Technique	Category	Structural defect and location	Engineering aspects	Advantages and disadvantages
Thickening surfacing	Strengthening	Inadequate overall live load-carrying capacity of arch barrel	Determination of new carrying capacity should take into account the effects of the increased surfacing depth, particularly in terms of the load dispersal and lateral support to the structure. Loading on the spandrel walls will also change.	Advantages Possible enhanced live load capacity Relative cost Disadvantages Traffic disruption during construction Structure life expectancy unaffected by works Further maintenance works may be required
Through-ring stitching	Remedial	Arch ring separation at any location over plan area of arch barrel Extensive cracking of arch barrel	Cross-stitching is not recommended as it changes the behaviour of the masonry to that of a brittle continuum. Radial pinning is preferred as it allows particulate behaviour while reinstating the longitudinal continuity between the rings and has the through-thickness performance of the ring. The longitudinal shear capacity of the inter-ring connection should be determined and used in determining the bridge load-carrying capacity.	Advantages Simple established methods Structure life expectancy extended Disadvantages Labour-intensive construction Possible traffic disruption during construction Possible provision of access required
Underpinning	Remedial	Inadequate bearing capacity of substructure formation	The cause of the foundation movement must be established and the effectiveness of underpinning justified. The effect of a stiffer foundation should be considered in respect of the rest of the structure (especially if the underpinning is restricted to part of the structure).	Advantages Allows existing structure to remain in service Commonly used technique Disadvantages Phasing of works Relative cost Service avoidance or division
Waterproofing	Preventative or remedial	Water seepage resulting in masonry degradation	Sufficient falls must be provided by the substrate to allow run-off. Selection of system must suit substrate.	Advantages Removes many problems associated with water penetration Reduced maintenance costs Disadvantages Disruption or closures Need to provide positive drainage at spring level (if barrel waterproofed) Suitable substrate for backfill waterproofing required
Penetrating coatings	Preventative or remedial	Minor surface cracking Surface contamination	A full assessment should be undertaken by a competent structural designer to establish possible adverse effects of the use of the coating on the structure's behaviour, particularly where water cut-off products are used.	Advantages Reduced maintenance Structure life expectancy extended Disadvantages Possible adverse effects

Table 13.9 Continued

the situation is further complicated by the need to represent the mortar–unit interface and, in the case of multi-ring brickwork arch barrels, the inter-ring properties. All this is before considering spandrel wall and skew effects and soil–structure interaction. A simple mechanism model should be used to inform any finite-element modelling. This can be achieved by modifying the mechanism model to include a moment of resistance equivalent to that of the reinforced masonry at hinges where the reinforcement is in tension.

It should be noted that the load distribution can be improved by including transverse reinforcement, provided this is sufficiently stiff. Additionally, by extending the reinforcement into the abutments, extra strengthening can be achieved. In the case of multi-ring brickwork arches, some form of stitching can be undertaken to ensure that ring separation does not occur (it is likely that the installation of the intrados reinforcement will have adversely affected the mortar, the units and the bond between them – grouting may be required).

If there is sufficient depth of cover over the arch and an acceptable traffic management programme can be agreed with all interested parties, the provision of a relieving slab is likely to offer the best long-term solution. Alternatively, a saddle may be considered. Again, there are some pitfalls, which must be avoided. It is important to determine why the bridge is showing signs of distress or failing its load-carrying capacity assessment. If it is the result of abutment failure or spandrel wall instability, providing a saddle might make things worse. Additionally, some thought should be given to the consequences of relieving the arch barrel if there is long-term stress, with the future possibility of units falling from soft spots in the intrados. These considerations will influence whether or not the saddle is bonded to the abutments and extrados (the latter may be so rough that bonding is inevitable unless some form of compressible layer is installed). The saddle usually comprises a reinforced concrete layer cast onto the extrados. Sometimes curved steel universal beam or column sections are used. If the cover is large or abnormal loading conditions must be accommodated, a relieving slab deck may be provided, which spans over the entire arch barrel onto new supports. In all cases, careful attention to detailing of waterproofing is important, to ensure that ingress of water to the old arch barrel is minimised and any water in the backfill is efficiently drained away.

The barrel may become delaminated or, in the particular case of multi-ring brickwork arch barrels, ring separated. Grouting or local patching may be possible but, in the case of brickwork, stitching may be considered as a means of reinstating the mechanical bond between the rings with a view to ensuring that the total thickness of the barrel is acting as one. It is best that the pins are installed radially and closely enough to ensure good load distribution. Saturated stitching should be avoided, unless it is accepted that a potentially brittle continuum may result.

Stitching can be used more generally to strengthen structural elements and stabilise suspect foundations. The latter usually take the form of mini-piles. As many masonry arch bridges were constructed using timber piles or grillages, problems have arisen, for example, owing to rotting of the timber or settlement

as a result of increased loading. Such structures can be rehabilitated by a system of mini-piles installed by drilling through the existing abutment or pier. The reader is directed to specialist texts for details of the installation (which vary depending on the site conditions). It is important to recognise that the new foundation will change the load path through the existing structure and that there will be a period of load transfer from the original to the new support. This may induce increased stresses in other elements of the bridge.

The installation of mini-piles is usually associated with stitching of the piers, abutment, barrel and walls. It should be appreciated that blanket stitching changes the masonry structure from a gravity, particulate material, which can accommodate small settlement, thermal and load-induced movements, without distress, to a brittle continuum, with all consequential material behavioural problems. Careful thought should be given to strategies that will mechanically connect the units together in such a way as to maintain their particulate articulation. In the case of multi-ring brickwork arch barrels, radial pins reinstate the through composite thickness behaviour of the brickwork by ensuring mechanical connection between the rings, while at the same time allowing particulate behaviour between the pins. If the stitching is criss-crossed through the arch barrel, particulate behaviour is prevented. Additionally, the percussive nature of the installation necessitates high-quality grouting to reinstate the units' strength and their interface bond and to produce a waterproof structure.

Many problems with masonry arch bridges are associated with the stability of the spandrel and wing walls (and their interaction with the arch barrel). The reasons for their distress are multifarious and include excessive backfill pressures, diurnal and seasonal movements, settlement, vehicles being allowed to run close to the inner face, accidental impact and deterioration. It might be that reconstruction is necessary, but this will only be done in extreme cases. More usually, tie-bars are used. These can be installed by drilling horizontally through the bridge. Typically, the tie-bar would be 32 mm in diameter and the spreader (or pattress) plates 600 × 600 mm. It is usual for the tie-bars to be installed with only a nominal tensioning – the logic being that the tie is there to prevent further movement rather than to pull the wall back to some predetermined position. Laboratory tests (Melbourne *et al.*, 1995) have shown that the ties have very little effect on the barrel behaviour until the barrel has formed a mechanism and enhanced soil interaction has been mobilised, at which stage the ties (by holding the walls together) confine the backfill and hence stiffen it – it is only at this stage that the tie force increases. These tests were undertaken over a short period and therefore did not investigate long-term effects. Tie-bars can be installed within the arch barrel itself. The dimensions of the pattress plates, in this case, are usually determined by the barrel thickness. At least five tie-bars are provided (at the crown, quarter points and springings). Their effect is to offer some transverse strengthening, which enhances the transverse arching contribution to load distribution; it also improves the longitudinal material properties by confining the material.

If the road surface is rutted or the track vertical alignment is a constant cause for concern, it may be that backfill is failing and needs improvement or replacement. Soil improvement techniques can be applied but usually (given the age of most of the arch bridge stock) the problem relates to ingress of water or changes in loading patterns. In any case, an investigation of the causes should be undertaken. As with all rehabilitation work, statutory undertakers and other users of the bridge must be consulted and agreement reached regarding the proposed works. Grouting can cause particular problems for statutory undertakers.

If it is deemed necessary to remove the backfill, saddling, relieving slabs, and so on, are the likely solutions. The reinstatement of the backfill may then take the form of weak concrete or reinforced soil, that is, methods of relieving the spandrel and wing walls of any soil pressures.

It is very important when considering repairing a masonry arch bridge that a holistic approach is taken. The engineer should consider the bridge as a gravity, particulate structure, the many elements of which interact with each other and the soil on which they rest. To change the nature of that interaction by intervention must be robustly justified.

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Chapter 14

Cable-stayed bridges

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Following a brief history of cable-stayed bridges, this chapter describes the various materials and forms of construction that have been adopted for the major structural components of these bridges, focusing in turn on the cable system, the pylon and the deck. By the use of examples, the most appropriate use of these materials and component forms is discussed. A step-by-step approach is given for the preliminary design of the cable-stayed bridge, from outline proportions of the structure to static and dynamic analysis, including requirements for erection calculations and wind loading on stays. The dynamic behaviour for the cable-stayed bridge includes the phenomenon of stay oscillation, which is reviewed in detail, including discussion of the various types of dynamic cable response together with the available preventative measures.

Introduction

The use of inclined stays as a tension support to a bridge deck was well known in the nineteenth century and there are many examples, particularly using the inclined stay as added stiffness to the primary draped cables of the suspension bridge. Unfortunately, at this time, the concept was not well understood. As it was not possible to tension the stays, they would become slack under various load conditions. The structures often had inadequate resistance to wind-induced oscillations. There were several notable collapses of such bridges, for example, the bridge over the Tweed River at Dryburgh (Drewry, 1832), built in 1817, collapsed in 1818 during a gale only 6 months after construction was completed. As a result, the use of the stay concept was abandoned in England.

Nevertheless, these ideas were adapted and improved by the American bridge engineer Roebling, who used cable stays in conjunction with the draped suspension cable for the design of his bridges. The best known of Roebling's bridges is the Brooklyn Bridge, completed in 1883.

The modern concept of the cable-stayed bridge was first proposed in postwar Germany, in the early 1950s, for the reconstruction of a number of bridges over the River Rhine. These bridges proved more economic, for moderate spans, than either the suspension or arch bridge forms. It proved very difficult and expensive in the prevailing soil conditions of an alluvial floodplain to provide the gravity anchorages required for the cables of suspension bridges. Similarly, for the arch structure, whether designed with the arch thrust carried at foundation level or carried as a tied arch, substantial foundations were required to carry these large heavy spans. By comparison, the cable-stayed alternatives had light decks and the tensile cable forces were part of a closed force system that balanced these forces with the compression within the deck and pylon. Thus,

expensive external gravity anchorages were not required. The construction of the modern multi-stay cable-stayed bridge can be seen as an extension, for larger spans, of the prestressed concrete, balanced cantilever form of construction. The tension cables in the cable-stayed bridge are located outside the deck section, and the girder is no longer required to be of variable depth. However, the principle of the balanced cantilever modular erection sequence, where each deck unit is a constant length and erected with the supporting stays in each erection cycle, is retained.

The first modern cable-stayed bridge was the Strömsund Bridge (Wenk, 1954) in Sweden, constructed by the firm Demag, with the assistance of the German engineer Dischinger, in 1955. At the same time, Leonhardt designed the Theodor Heuss Bridge (Beyer and Tussing, 1955) across the Rhine at Düsseldorf but this bridge, also known as the North Bridge, was not constructed until 1958. The first modern cable-stayed bridge constructed in the UK was the George Street Bridge over the River Usk (Brown, 1966) at Newport, south Wales which was constructed in 1964. These structures were designed with twin vertical stay planes. The first structure with twin inclined planes connected from the edge of the deck to an A-frame pylon was the Severin Bridge (Fischer, 1960) crossing the River Rhine at Cologne, Germany. This bridge was also the first bridge designed as an asymmetrical two-span structure.

The economic advantages described here are valid to this day and have established the cable-stayed bridge in its unique position as the preferred bridge concept for major crossings within a wide range of spans. The longest-span cable-stayed bridge so far completed is the Tatara Bridge in Japan with a main span of 890 m. At the time of writing (2008), several other bridges are planned, or are in construction, with main spans in excess of 1000 m, notably the Sutong Bridge (1088 m) and the Stonecutters Bridge (1018 m), which are both in China.

Stay cable arrangement

Two basic arrangements have been developed for the layout of the stay cables:

- the fan stay system (including the modified fan stay system)
- the harp stay system.

These alternative stay cable arrangements are illustrated in Figure 14.1.

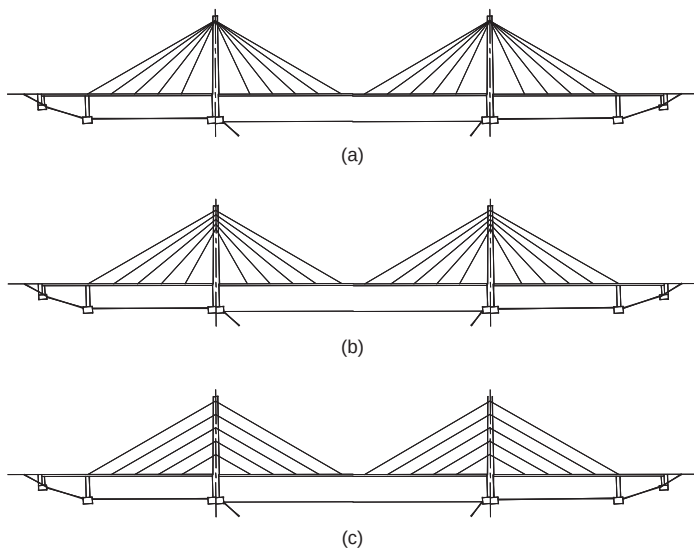


Figure 14.1 Alternative stay cable arrangements: (a) fan stay system; (b) modified fan stay system; (c) harp stay system

Fan cable system

The fan system was adopted for several of the early designs of the modern cable-stay bridge, including the Strömsund Bridge (Wenk, 1954). The method of supporting the stays on top of the pylon was taken from suspension bridge technology, where the cable is laid within a pylon top deviator saddle. The floor of the saddle is machined to a radius so that each cable-stay anchored in the main span can pass over the pylon and be anchored directly within the back or anchor span. This arrangement is structurally efficient, with all the stays located at their maximum eccentricity from the deck and a minimum moment applied to the pylon.

The fan arrangement initially proved suitable for the moderate spans of the early cable-stay designs, with a small number of stay cables or bundled cables supporting the deck. There were, however, obvious difficulties, with the corrosion protection of cables at the pylon head, their susceptibility to fretting fatigue arising from bending and horizontal shear stresses within the cable bundle and with the replacement of any individual stay in the event of damage. In addition, when this arrangement was adopted for larger spans, the size of the limited number of cables increased, eventually becoming uneconomically large

and difficult to accommodate within the fan configuration. The anchorages were also heavy and more complicated and the deck needed to be further strengthened at the termination point.

Therefore, when a greater number of stays were required, the modified fan layout was introduced, whereby the stays are individually anchored near the top of the pylon. This is now the more commonly adopted system. To give sufficient room for anchoring, the cable anchor points are spaced vertically at 1.5–2.5 m. Providing the anchor zone is maintained close to the pylon top, there is little loss of structural efficiency as the behaviour of the cable system will be dominated by the outermost cable, which is still attached to the top of the pylon and anchored at the supported end of the back span. The advantages of this arrangement are as follows.

- The large number of stays distribute the forces with greater uniformity through the deck section, providing a continuous elastic support; hence, the deck section can be both lighter and simpler in its construction.
- As each stay supports a discrete deck module, each module can be erected by the progressive cantilever method without resort to any additional temporary supports. Thus, increased speed and efficiency of the deck erection is possible.
- The concentrated forces at each anchor point are much reduced.
- With the modified fan layout, it is also possible to completely encapsulate each stay, thus giving a double protective system throughout its length. Should damage occur, replacement of the stay can be undertaken as a routine maintenance task.
- The large number of stays of varying length and natural frequency increases the potential damping of the structure.

Freyssinet International has reintroduced the concept of a deviator saddle at pylons in conjunction with a modified strand system, for use in smaller-span cable-stayed bridges and extradosed bridges. The modified strand, known as Cohestrand®, is protected by a polyethylene sheath but is filled internally with polymer resin instead of petroleum wax. The resin compound is hydrophobic, resistant to water vapour and oxygen and capable of transferring both compression and shear forces from the polyethylene sheath to the steel wires of the strand. The strand can thus be continuous through the deviator saddle without the need to remove the polyethylene sheath. This enables more slender pylons to be constructed without having to provide a crossover stay arrangement. The disadvantages of earlier saddle designs have been addressed in that the corrosion protection of each strand is continuous through the saddle, individual strands are not in contact and thus not subject to fretting corrosion and the system is replaceable strand by strand. The deviator saddle is made of a bundle of tubes placed within a larger steel saddle tube. All voids between the tube bundles are filled with a high-strength fibre concrete in the factory. If necessary, the external surface of the polyethylene sheath can be locally treated, at the saddle location, to ensure that the friction coefficient between the saddle and the strand is greater than 0.5. The arrangement of the strands and saddle is illustrated in Figure 14.2.

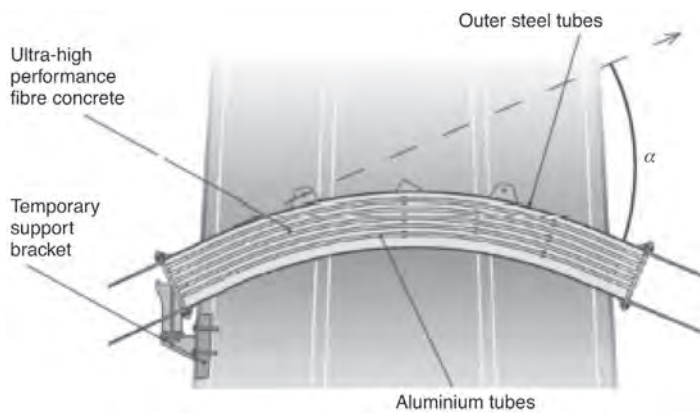


Figure 14.2 Deviator saddle for Cohestrand (courtesy of Freyssinet International)

This arrangement of multi-tube saddle and Cohestrand® has been incorporated in a number of bridges worldwide: in Malaysia, Vietnam, Korea, Lithuania and the Sudan. A typical design is that of the Sungai Muar Bridge, a 132 m main-span cable-stayed bridge in Malaysia.

Harp cable system

With the harp system the individual stays are anchored at equal spacing over the height of the pylon and are placed parallel to each other. This arrangement provides a visual emphasis of the flow of forces from the back span to the main span and, in examples that are well proportioned, is aesthetically pleasing. However, the arrangement is not as structurally efficient as the fan layout and relies on the bending stiffness of the pylon or deck for equilibrium under asymmetrical live loading. When loading one end only of the stay system, the load may be divided into symmetrical and antisymmetrical components of loading. The symmetrical loading will be resisted by the triangle of forces formed by the stays, pylon and deck, but the antisymmetrical loading can only be resisted by bending of the deck, the pylon or a combination of both, depending on their relative stiffnesses. This disadvantage can be overcome by anchoring the back stay cable at approach pier locations so that any unbalanced load is resisted by the pier. An elegant example of this arrangement is the Knie Bridge over the River Rhine at Düsseldorf with its single pylon and 320 m main span.

Multispan bridges

The main concern with multispan cable-stayed bridges is the lack of longitudinal restraint to the top of the inner pylons, which cannot be directly anchored to an approach pier. Without providing additional longitudinal restraint, a multispan structure would be subject to large deformations under the action of live load. Increasing the stiffness of either the pylons or the deck can provide this additional restraint. However,

an increase in the deck stiffness will be accompanied by an unacceptable increase in the dead load; thus, the more practical approach is to stiffen the pylon. A typical example of the stiffened pylon is the A-frame braced pylon shown in Figure 14.3(a). However, such an arrangement requires a substantial increase in the quantity of pylon materials and a much larger foundation. An alternative to increasing the bending stiffness of the pylon is the introduction of an auxiliary cable system to provide the additional stiffness and stability. Two cable systems are illustrated. The first system, shown in Figure 14.3(b), connects the tops of the pylons and thus directly transfers any out-of-balance forces to the anchor stays in the end spans. The second system, shown in Figure 14.3(c), connects the top of the internal pylons to the adjacent pylon at deck level so that any out-of-balance forces are resisted by the stiffness of the pylon below deck level. An example of this latter arrangement can be seen with the design of the Ting Kau Bridge, Hong Kong, which is a four-span cable-stayed bridge. The disadvantage of such an auxiliary cable system is that the individual cables are very long and the large sag will be visually dominant, compared with the adjacent stay cable plane. Special measures are necessary to limit the propagation of wind-induced oscillations in these very long stays (see the section on stay oscillations).

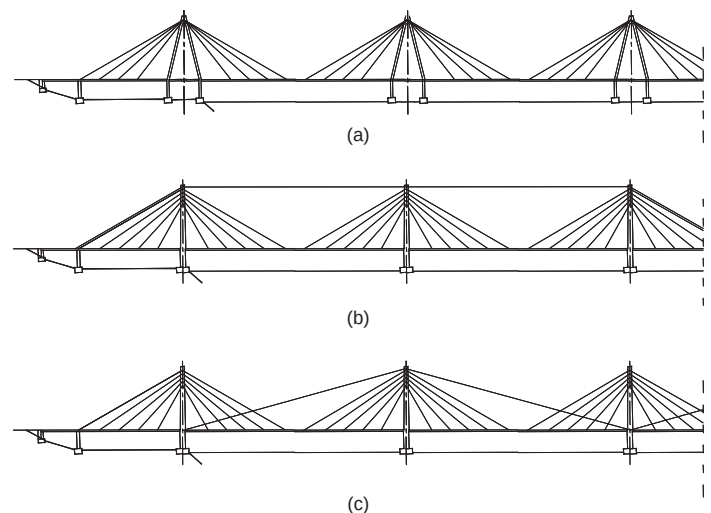


Figure 14.3 Multispan bridges: (a) A-frame braced pylon; (b) additional cable system where tops of pylons are connected; (c) additional cable system where tops of internal pylons are connected to adjacent pylons at deck level

Number of cable planes

The cable layout may be arranged as either a single-plane system or a twin-plane system.

The twin-plane system may be formed as either two vertical planes connected from the edge of the deck to two pylon legs located outside the bridge cross-section or as twin inclined planes connected from the edge of the deck to either an A-frame or an inverted Y-frame pylon. The A-frame pylon was first

adopted for the Severin Bridge (Fischer, 1960). The vertical twin-plane system, with its tensioned stay geometry, provides considerable rigidity between the deck and pylon, compared with the free-hanging cables of the suspension bridge. Inclined stays further increase the stiffness and stability of the structure, with the stays and deck forming a transverse frame. Inclined stays are of particular benefit when adopted for longer spans, as they improve the torsion response of the structure to both eccentric live load and aerodynamic effects. Comparing the alternative stay systems, when supporting a deck with low torsional stiffness, the inclined stay system connected to an A-shaped pylon will have approximately half the rotation under eccentric loading as the vertical twin-plane system. The inclined stay system is also aerodynamically superior, reducing the magnitude of vortex shedding oscillations and increasing the critical wind speed of the structure. However, the geometry of the inclined stay planes must be carefully checked in relation to the traffic envelope and the clearances required may result in an increase in the overall width of the deck.

The single-plane system creates a classic structural form, avoiding the visual interference often associated with twin-cable planes. However, the single plane is not able to resist torsion loading from eccentric live loading; therefore, this configuration requires the deck to be in the form of a strong torsion box. A deck section of this form is likely to have excess resistance to the longitudinal bending of the deck, particularly when a multi-stay arrangement is used. The single pylon must be located within the central median of the carriageway and, as such, an additional width of deck is required to provide for the necessary clearances to traffic.

Two outstanding examples of cable-stayed bridges with a single stay plane are the Rama IX Bridge (Gregory and Freeman, 1987) (Figure 14.4) and the Sungai Prai Bridge (Figure 14.5). The Rama IX Bridge crosses the Chao Phraya River, Bangkok, with a 450 m main span and has an orthotropic steel box deck section that is 4 m deep. The deck section carries three lanes of traffic in each direction and is 33 m wide. The Sungai Prai Bridge crosses a major river at Butterworth, Malaysia, with a 185 m main span and a 3 m deep concrete box with a curved soffit and side struts. The deck section carries three lanes of traffic in each direction and is 28.6 m wide.



Figure 14.4 Rama IX Bridge, Bangkok



Figure 14.5 Sungai Prai Bridge, Malaysia. (Courtesy of Concessionaire, LLB)

It is possible to combine the use of twin- and single-plane arrangements in a single structure, as incorporated into the Rama VIII Bridge, Bangkok, completed in 2002 (Figure 14.6). This bridge has a single inverted Y-pylon with twin inclined stay planes supporting a main span of 300 m, whereas the back span has a single stay plane anchored directly to a piled abutment.



Figure 14.6 Rama VIII Bridge, Bangkok (courtesy of Yee Associates)

Stay design

Many factors must be considered in the design of the stay system, including the characteristic breaking strength and the effective stay modulus. The proportion of the breaking strength that can be realised depends on the relaxation of the stay under permanent loads. The irreversible strain arising from relaxation increases rapidly when the permanent load in the stay exceeds 50% of the breaking load. The Post-Tensioning Institute Committee on Cable-stayed Bridges (PTI, 2001) recommendations limit, for normal load combinations, the maximum load in the stay to 45% of the stay breaking load and to 50% for exceptional load combinations. The CIP (2002) *Recommendations for Cable Stays* limit the maximum load in the stay, at the serviceability limit state, to 50% of the stay breaking load.

and, at the ultimate limit state, to 70% of the stay breaking load. This slightly higher loading is permitted on the provision that detailing recommendations to limit bending effects owing to eccentricity at the anchorages and cable-stay vibration are incorporated in the design.

The permissible load in the stay may also be limited by the fatigue performance of the stay under repeated live load cycles. However, this will depend on the magnitude of the live load cycles as a proportion of the permanent loads. Thus, it is only the most heavily loaded stays of a highway structure that are possibly limited by fatigue, whereas the stays for a railway structure, where the live load has greater dominance, will be much more fatigue-sensitive. Fatigue endurance of a component is usually given in a plot of the stress range ($\Delta\sigma$) against the number of load cycles (N), known as the Wöhler curve. When the Wöhler curve is represented on a log-log scale, the plot is represented by a series of straight lines. The PTI recommendations relate the design limit to the test acceptance criteria of each type of stay material, such as strand, bar or wire, and the test criteria of the assembled stay. The recommendations for parallel wire strand (PWS) and parallel strand are given in Figure 14.7. The fatigue endurance of the assembled stay will not only result from variations in the applied axial tension but also be influenced by any secondary bending in the stay, arising from either wind- or structure-induced vibrations at the anchorage or bending of the stay in a saddle. The response to these factors is extremely important, though complex, and varies according to the manufacturing characteristics of the stay and its anchor. Because of this, the PTI recommendations propose that at least three representative samples of the stay assembly to be used in a project be fatigue tested. Testing is usually undertaken over 2 million cycles. The stress range depends on the generic type of stay being tested but the upper limit of the stress range is always taken as 45% of the breaking load. Acceptance criteria for the test are based on a limit to the number of individual wires in the stay that may break and a requirement that a tensile test, undertaken after the fatigue

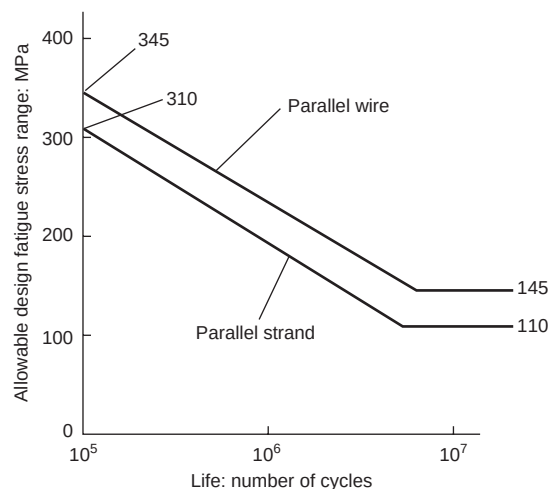


Figure 14.7 Wöhler endurance curves for parallel wire and parallel strand stays

test, achieves at least 95% of the guaranteed breaking load of the stay.

The CIP (2002) *Recommendations for Cable Stays* include the effect of coincident bending through a modified fatigue test, whereby the cable specimen is made to deviate sinusoidally from the anchorage centre line at the same frequency as it is being subject to an axial stress variation.

Stay types

Problems arose with the stays of early cable-stay bridges as a result of deficiencies with the anchorage design, steel material problems and inadequate corrosion resistance. The development of modern stay systems has largely overcome these problems, providing designs that minimise bending of the stay at the anchorage face and incorporate a double corrosion protection system throughout. Available stay systems include:

- locked coil (prefabricated)
- helical or spiral strand (prefabricated)
- bar bundles
- parallel wire strand (PWS)
- new PWS (prefabricated)
- parallel strand
- advanced composites.

Locked coil stays

Locked coil stays have been incorporated in many of the earliest cable-stay bridges. The stays are factory-produced using planetary stranding machines, each layer being applied in a single pass through the machine and contralaid between each layer. The core of the stay is composed of conventional round steel wires, while the final layers comprise Z-shaped steel wires, which lock together, creating an extremely compact stay cross-section. A typical example of a locked coil stay is illustrated in Figure 14.8. Modern locked coil stays provide all the wires in a finally galvanised condition and will achieve a

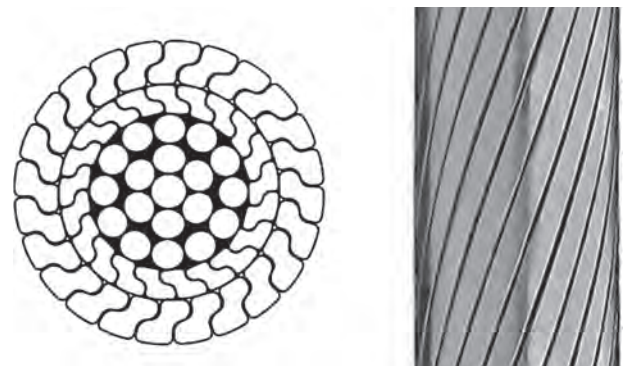


Figure 14.8 Locked coil cable (courtesy of Bridon International Ltd)

tensile strength of up to 1770 N/mm^2 . The stays are commonly anchored by zinc-filled sockets, although sometimes stays that are sheathed with a polyethylene protection have their sockets filled with epoxy resin. The largest locked coil stays manufactured to date are the 167 mm diameter stays supplied for the Rama IX Bridge over the Chao Phraya River, Bangkok.

Helical or spiral strand stays

Helical or spiral strand stays, which are illustrated in Figure 14.9, are also factory-fabricated on a planetary stranding machine similar to the locked coil stay but are entirely manufactured from finally galvanised round steel wires. The wires are usually of 5 mm diameter with a tensile strength of either 1570 or 1770 N/mm^2 . The largest spiral strand stays manufactured to date are 164 mm in diameter, as supplied for the Queen Elizabeth II Bridge over the River Thames at Dartford.

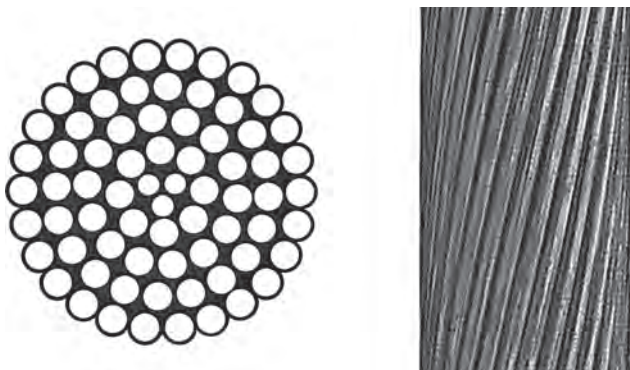


Figure 14.9 Spiral strand cable (courtesy of Bridon International Ltd)

Bar bundles

Bar bundles contain up to ten threaded steel bars with a tensile strength of 1230 N/mm^2 coupled together in 12 m lengths. The bars have conventionally been placed in a steel tube and protected with a cement grout. The use of couplers connecting the bars will give a much reduced fatigue resistance, compared with the equivalent wire or strand systems. Coupled bar systems are thus rarely used where significant variations in the stay load are likely to occur. Tests have also been undertaken to assess the effectiveness of cement grout as a protective medium. These tests concluded that transverse and longitudinal cracking of the grout rapidly develops, owing to temperature effects, live load strains and wind vibration. It may be assumed that the cement grout provides little protection against corrosion.

Parallel wire strand stays

Parallel wire strand (PWS) most commonly comprises 7 mm diameter finally galvanised round steel wires, with a tensile strength of 1570 N/mm^2 , and may either be prefabricated or assembled on-site, the wires being installed without a lay or helix within a polythene tube and injected with cement grout or wax. When manufactured without a lay, the prefabricated stays

are difficult to handle and coil onto the reel; the system also suffers from the doubts associated with grouted stays.

New parallel wire strand stays

The new PWS system, as illustrated in Figure 14.10, was developed with a tensile strength up to 1770 N/mm^2 . The stay is prefabricated, with a long lay helix to improve coiling onto the reel. The largest stays can contain up to 400 wires; a coating of high-density polyethylene (HDPE) is applied in the factory using the continuous extrusion process. The stays can be socketed using a patented system, such as BBR's DINA or HiAm anchorages. The individual wires within these anchorages incorporate button heads, transferring the full load to the anchor. The socket is then filled with a proprietary epoxy compound that is claimed to enhance the fatigue resistance of the stay. Other manufacturers provide these stays with conventional socketed anchorages filled with zinc or epoxy resin.

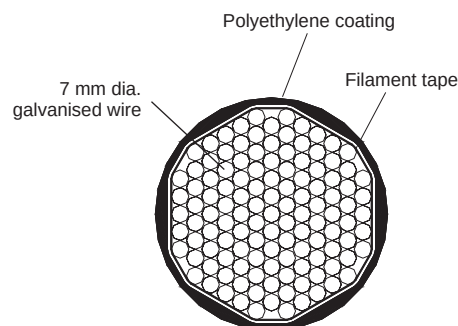


Figure 14.10 New parallel wire strand (PWS) system

Parallel strand stays

Parallel strand stays are usually manufactured from 15.7 mm (or 15.2 mm) diameter seven-wire strands, which are usually galvanised and have a tensile strength of 1770 N/mm^2 , to give a characteristic breaking load per strand of 265 kN. Some manufacturers also supply strand with a tensile strength of 1860 N/mm^2 . The strand bundle can typically comprise up to 110 strands; anchoring is by means of a prestressing anchor head, with individual strands gripped by wedges. To provide adequate fatigue resistance, it is essential that rotation of the strands at the face of the wedge grips, owing to changes in stay load or wind oscillation, is minimised. Guides or dampers located some distance in front of the anchor face provide the necessary restraint (see the section on stay flexure at the anchorage). An outer polyethylene tube covers the strand bundle, providing protection against impact damage and preventing dynamic oscillation or rattling of the individual strands. Early designs filled the tube with cement grout or wax as corrosion protection but in later designs each strand is manufactured with a continuously extruded HDPE coating over a corrosion-inhibiting petroleum wax. With this level of protection, further cement or wax injection is unnecessary. European and Japanese

practice has been to use galvanised strands but some American bridges have been constructed using epoxy-coated strand. The risk with epoxy-coated strand is that small pinholes or minor damage can propagate corrosion, forming a notch in the strand, which will create a local stress concentration. The use of Galfan, a licensed zinc–aluminium alloy coating, gives two to three times the protection for the equivalent weight of zinc coating, but the fatigue properties of the strand are reduced.

The outer covering to the stay has conventionally been manufactured from steel or polyethylene pipe. Where polyethylene pipe is used, ultraviolet (UV) resistance was achieved by the use of carbon black pigment in the material. The design temperature differential between the stays and the deck or pylon can vary considerably. The PTI (2001) recommendations note that values of 9°C and 22°C have been used for white painted or taped stays and black stays, respectively. The use of black stays is not preferred, particularly in tropical zones, where there is a high solar gain. Early attempts to wrap the stays, as in the case of the Pasco–Kennewick Bridge over the Columbia River, USA, where a white plastic wrapping was used, were unsuccessful, as the coating deteriorated within a few years. Later coverings using Tedlar, as in the case of the Prince of Wales Bridge over the River Severn between England and Wales (Mizon *et al.*, 1997), have been more successful. Subsequently, a range of light-coloured polyethylene pipes has been developed with a good UV resistance. The pipe is manufactured in a bi-extrusion process, in which a thin coating of light-coloured polyethylene is extruded over a black pipe core.

Advanced composite stays

Advanced composite stays are manufactured from aromatic polyamide (aramid) fibres, developed by DuPont in the 1970s under the trade name Kevlar. Kevlar is manufactured in three grades and has an exceptionally good strength-to-weight ratio. The structural grade Kevlar 49 has a tensile strength in the range 3600–4100 N/mm² and a density of 14.4 kN/m³. Owing to its good resistance to corrosion, the material has found favour in the manufacture of rope for use in a marine or offshore environment. An experimental cable-stayed footbridge has also been constructed in Aberfeldy, Scotland. The structure, which has a main span of 63 m, utilises Kevlar aramid stays, protected with a low-density polyethylene coating. For further information on the use of non-metallic stay systems reference should be made to Chapter 18.

Stay behaviour

The behaviour of the stay under load must be represented in the analysis of the structure. The modulus of the stay under load is a characteristic of the stay manufacture and a non-linear variation with respect to both stay length and axial tension. When comparing the modulus of various types of stay that are manufactured, parallel wire strand (PWS) achieves the high-

est modulus, at 205 kN/mm². This is close to the modulus of the steel wire itself. Seven-wire strand achieves a modulus of some 195 kN/mm², while locked coil will be approximately 155 kN/mm². The modulus of helical strands will be within the range 155–175 kN/mm². The modulus of both locked coil and helical strands is variable, depending on the lay angle, the galvanising and the stay diameter. Factory-produced locked coil and helical strand cable, which are prestressed as part of the manufacturing process, will give the stays a predictable elongation in service. Prestressing, where the cable is supported in a bed and preloaded, is the method used to remove the non-elastic stretch, resulting from the initial compaction of the strand. Prestressing is not required for PWS or parallel strand stays. The non-linear behaviour of the stay may be represented by an equivalent modulus, taking into account the sag or catenary effect in the loaded stay. The variation in the equivalent modulus of elasticity of the stay (E_{eq}) is given in Figure 14.11 and may be expressed as

$$E_{eq} = \frac{E}{1 + (\gamma^2 \times L^2 \times E / 12 \times \sigma^3)} \quad (14.1)$$

where E is the guaranteed modulus of the straight stay, L is the horizontal length of the stay, γ is the specific weight of the stay and σ is the tensile stress in the stay.

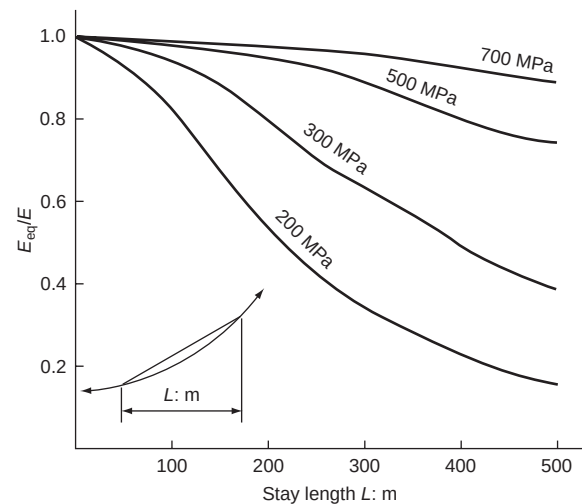


Figure 14.11 Equivalent modulus of elasticity of stay cable (based on $E = 205\,000 \text{ N/mm}^2$)

Allowance must be made during the erection of the deck for the dead load extension of the stay. In the case of a prefabricated stay, this is achieved in manufacture, by reducing the stay length to compensate for this extension. In the case of the stay fabricated on-site, using a strand system, the calculated extension of the stay is taken up within the anchorage. The length of the stays will also vary with changes in temperature; it is therefore necessary to measure the temperature of the stay and deck at the time of stay installation and calibrate the stay load accordingly.

Stay flexure at the anchorage

Some stay systems are designed with rotational adjustment, through a pin and clevis arrangement, which assists with the proper alignment of the stay during its erection. However, when the cable is subject to small angular deviations in service, the inertia of the anchorage components will inhibit the cable end from rotating and thus the anchorage connection, whatever arrangement is adopted, should be considered fixed-ended. Angular deviations at the anchorage may arise through a number of cumulative effects:

- installation error when the anchorage is built into the structure
- vibration of the cable-stay, owing to wind and other effects, causing an oscillating rotation at the anchorage
- structural displacements, causing a varying rotation of the anchorage with respect to the stay cable
- varying load in the stay cable, owing to the effect of imposed loading on the structure, causing varying rotation of the stay cable at the anchorage through the catenary effect.

When no guide is installed to limit the rotation of the stay, the maximum bending is at the face of the anchorage and decreases exponentially over a characteristic length, which depends on the bending stiffness of the cable. The bending stresses at the face of the anchorage are high, are located where they are most damaging and are typically of the same order as those for live loading. However, they are identical, whether the stay is monolithic, such as with a spiral or locked coil strand, or is composed of separate tensile components that can slide over each other, such as parallel strand or parallel wire stays. Monolithic stays vary from separate tensile components in that the characteristic length is much longer; this has to be considered when designing any guide system.

To limit these harmful bending effects, a guide system should be provided at a distance from the anchor face. The guide, acting as a simple support, is usually located at the end of the anchor pipe, which is an integral part of the bridge pylon or girder, so that the stay is subject to continuous bending over the guide. When adopting a rigid guide, subject to it being located a sufficient distance from the face of the anchor, the moment will be reduced by half the original moment at the face of the anchor. However, by manufacturing the guide from an elastic material, typically polybutadiene, with an optimum spring stiffness designed for each individual cable arrangement, the bending stress in the cable can be further reduced to a third of the original moment. This effect is illustrated in Figure 14.12.

Stay oscillations

A phenomenon peculiar to cable-stay bridge construction is the effect of stay oscillation. During cantilever erection, a slender deck may be particularly prone to wind-induced movement. This can, in turn, excite the stays, producing violent oscillations, which, in some projects, have had to be restrained with

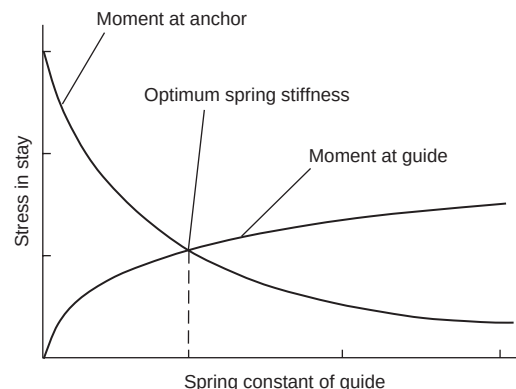


Figure 14.12 Effect of guides on stay anchor bending

temporary straps. In service, the wind-induced vibration of stay cables has also occurred on a range of cable-stayed bridges under a variety of wind and traffic conditions. Both standing waves and travelling waves have been observed; these oscillations can reach amplitudes of more than a metre. This behaviour was reported to have occurred on the Helgeland Bridge (Svensson and Jordet, 1996).

A cable's natural frequency for the fundamental mode may be calculated as

$$N = \frac{1}{2L} \sqrt{\frac{T}{m}} \quad (14.2)$$

where L is the chord length of the cable, T is the cable tension and m is the cable mass per unit length. It is normal to provide an anchor guide system that gives an apparent fixity to the cable. In this case, the chord length of the cable may be assumed to be the length between guides at the top and bottom of the cable.

Cable vibrations can occur as a result of a number of effects, which can be categorised as

- vortex shedding
- wake-induced vibrations
- rain–wind instability
- cable galloping
- parametric instability
- rattling.

Appropriate measures must be planned to suppress cable vibration owing to these phenomena.

Vortex shedding

This effect is due to the alternate shedding of vortices from opposite sides of the strand, inducing a periodic load on the cable. As steady wind is required for this effect to occur, the most damaging vibrations generally occur at low wind veloc-

ities. They can be mitigated by reducing or eliminating the underlying excitation along the lengths of the stay cables that drives stay oscillation by introducing projections, or texturing, of the stay pipe. This roughens the surface of the cable and presents an irregular surface to the wind flow.

Vortex shedding is expected to occur when its frequency becomes approximately the same as the natural frequency of a stay, that is to say, a wind velocity approaching the resonance-inducing velocity. To avoid this phenomenon, the CIP (2002) *Recommendations for Cable Stays* require that the natural frequency of stays should not be the same as the vortex shedding frequency:

$$N = \frac{US_i}{D} \quad (14.3)$$

where U is the wind velocity, D is the outer diameter of the stay and S_i is the Strouhal number, which is 0.20 for a circular cable.

Wake-induced vibrations

Wake-induced vibrations occur when the leeward stay lies in the wake of a windward obstacle, such as another stay. This effect has most often occurred in moderate winds, which are not turbulent, and the effect is sometimes related to rain or ice accretion. The effect can be mitigated by improving the cable system stiffness through tying the cables together using cross-cables. These couple the modes of the different stays and thus stiffen the combined cable system so that any excitation causes less oscillation and self-excited modes are avoided. This method of mitigating cable oscillation was employed on both the Pont de Normandie and the Helgeland Bridge.

Rain–wind instability

Rain–wind instability results from perturbing the smooth surface of a stay and occurs when water rivulets form on the top and bottom of a stay, with wind in the direction of the span, and for stays that slope downwards in the direction of the wind, as illustrated in Figure 14.13.

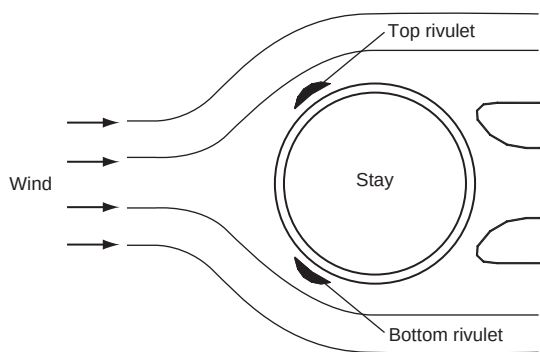


Figure 14.13 Wind–rain oscillation of stays

The CIP (2002) *Recommendations for Cable Stays* note the results of research showing that there is a possibility of rain–wind excitation if the steady wind velocity is in the range 8–15 m/s. The wind must be from an oblique direction, between 30° and 80° from the perpendicular to the cable, so that the wind will tend to lift the cable. The frequency of the cable oscillation is typically 1–3 Hz. Below the stated wind speed range, the top rivulet does not form because the wind is insufficient to prevent it from running down the side of the stay. Above the range, the wind forces tend to blow the rivulets off the stay. Wind turbulence also prevents the rivulets from forming. To avoid the onset of rain–wind oscillation, it has been suggested that the Scruton number (Sc) for the stay should be at least 10, according to the following formula (PTI, 2001):

$$\frac{m\zeta}{\rho D^2} \geq 10 \quad (14.4)$$

where m is the mass per unit length of the stay, ζ is the damping ratio (typically in the range 0.5%–1%), ρ is the density of air and D is the outer diameter of the stay.

Rain–wind excitation is therefore unlikely to occur if the damping ratio satisfies this formula. Where stay cable pipes have effective surface texturing, it has been suggested that this requirement may be relaxed, such that $Sc \geq 5$. However, this conclusion is based on limited testing of regularly spaced stay arrangements; a study by Jones *et al.* (2003) recommends that a careful case-by-case evaluation of these limits be undertaken.

Cable galloping

The possibility of galloping oscillations of either single cables or groups of cables should also be investigated. The PTI (2001) recommendations propose the following equation to check whether cable galloping occurs:

$$U_{\text{crit}} = cND\sqrt{\frac{m\zeta}{\rho D^2}} \quad (14.5)$$

where $c = 40$ (PTI, 2001) and 35 (CIP, 2002) for circular cables.

The CIP (2002) *Recommendations for Cable Stays* raise doubts as to the validity of this formula; however, no other advice is given. Clearly, as the wind speed increases there could be instances of cable galloping, but usually increasing wind speed is accompanied by the wind becoming more turbulent; in this state, the likelihood of galloping will be reduced.

Parametric excitation

Parametric excitation can occur when the frequency of an applied load, derived from either vehicular or wind excitation of the girder or pylon, causes small vibrations of the deck or pylon cable anchorages, which match a stay frequency or any multiple (harmonic) thereof. Stays with an oscillation amplitude of several metres can occur, although these more

pronounced effects are usually when the deck is poorly streamlined, such as when twin I-girders are adopted. The deck anchorages are usually the main driver for such oscillations but it is possible for an unbraced pylon to vibrate under wind loading. The Øresund Bridge, for example, is reported to have experienced stay oscillations, owing to this phenomenon. The new Stonecutters Bridge in Hong Kong has 300 m high pylons, which were originally conceived as tapered tubular steel members above the deck level. However, the transverse frequency of the pylons was found to be close to the cable frequencies and consequently vulnerable to parametric oscillation under longitudinal wind conditions. The pylons were modified to concrete construction, with a composite steel skin, in order to increase the pylon mass and hence their frequency. It is therefore possible to set limits on the wind-induced motions and frequencies of the girder and pylon so as to limit or preclude objectionable parametric excitation of the stays, by ensuring separation between the deck or pylon frequencies and the stay frequencies.

Rattling

A stay that is made up of a bundle of sheathed strands experiences aerodynamic interaction, whereby the outer strands will move in and out of the bundle and slap against the inner strands, eventually initiating a general motion of the whole cable. The solution is to encase the strand bundle within an HDPE pipe; this forms an integral component within all modern stay cable designs.

Methods of damping stay oscillations

There are three methods of damping stay oscillations:

- incorporating internal and external damping mechanisms
- texturing the external surface of the cable cover
- installing stabilising cables.

Dampers

The damping ratio of a cable-stay is the sum of its intrinsic damping, which will vary according to the type of stay, and the aerodynamic damping, which increases with increasing wind speed. The intrinsic damping of a cable-stay is low, with a typical range of 0.1%–0.3% (a logarithmic decrement of 0.6%–1.8%). Wind–rain instability can be avoided if the total stay damping exceeds 0.5% (a logarithmic decrement of 3.0%).

Various devices are available to supplement the cable damping. External dampers are hydraulic devices, which apply a transverse damping force to the stay and are mounted on structures that are fixed to the deck close to the stay anchorages. Internal dampers are ring-shaped and placed between the stay and the steel anchorage tube, which is built into the structure.

Internal dampers use the distortion of a dissipating material (specially formulated neoprene) or viscous friction or dry friction. The object of a damper is to minimise the amplitude of any cable-stay vibration. However, unlike a guide, which would completely inhibit cable displacement, a damper must permit some displacement if it is to effectively dissipate energy.

Texture of the stay pipe

Applying a texture to the external surface of the stay is also effective in preventing wind–rain instability as it helps prevent rivulets forming long continuous lengths. Initially, on Japanese bridges, longitudinal fins were adopted but when this type of profile is adopted, the drag coefficient increases dramatically to 1.35. Later helical ribs, as illustrated in Figure 14.14, or a series of random dimples were used and tests have confirmed that the recommended drag coefficient of 0.7 is still appropriate. Tests have shown that the helical ribs are more effective in preventing wind–rain instability than the random dimples.



Figure 14.14 Outer sheath for parallel strand stay with helical ribs (courtesy of Nigel Hewson)

Stabilising cables

Stabilising or secondary cables stiffen the stay cable array and can also provide additional damping. They are, however, of limited effectiveness in preventing wind–rain instability but can successfully inhibit vertical vibration modes of the stays, arising from parametric excitation. They are less effective in reducing transverse vibration modes. Stabilising cables must be sufficiently prestressed, so that they do not de-tension under extreme variations in structure loading. Failures occurred in early designs, with insufficient prestress, where they were subject to repeated shock following unloading. The trajectory of the stay cables will be modified by the stabilising cables, so kinking of the stay cable at the anchorage will occur. The setting of the rigid anchor tubes at deck and pylon needs to be corrected, so that they follow the modified cable geometry.

Stabilising cables cannot therefore be easily used as a remedial measure when cable vibration arises, as only limited prestressing of the stabilising cables is possible without unacceptable modification of the stay geometry of the main cables leading to anchorage kinks.

Summary of preventative measures

Wind and rain instability should be avoided by the inclusion of suitable texturing of the stay cable pipe together with a minimum Scruton number between 5 and 10, achieved by the installation of suitable damping devices.

Long stays (more than 80 m) should have dampers installed, such that the stay has at least 0.5% damping.

Parametric resonance should be assessed at the design stage, by a study of the eigenmodes of both the cables and the structure. When the cable and structure frequency are close (within 20% of each other), the use of stabilising cables should be considered.

To avoid alarming the bridge user, it is recommended in the French CIP (2002) *Recommendations for Cable Stays* that the amplitude of the stays should also be limited. Ratios of amplitude to stay length (L) in the range $L/1000$ to $L/1600$ have previously been adopted.

Pylons

The pylon is the main feature that expresses the visual form of any cable-stayed bridge, giving an opportunity to impart a distinctive style to the design. The design of the pylon must also adapt to the various stay cable layouts, accommodate the topography and geology of the bridge site and carry the forces economically.

The primary function of the pylon is to transmit the forces arising from anchoring the stays; these forces will dominate the design of the pylon. The pylon should ideally carry these forces by axial compression where possible, such that any eccentricity of loading is minimised.

Steel pylons

Early cable-stay pylon designs were predominantly constructed as steel boxes, and bridges such as the Strömsund Bridge (Wenk, 1954) took the form of a steel portal frame, which was intended to provide transverse restraint to the stay system. However, this restraint is largely unnecessary, as sufficient transverse restraint can be provided within the stay system itself. When a single mast supports each stay plane, any lateral displacement at the top of the mast is accompanied by a rotation of the stay plane. This rotation of the stay plane ensures, for the simple fan layout of stays attached to the top of the mast, that the resultant reaction from the main-span and back-span stay cables will pass through the pylon foot. The weight of the pylon will remain vertical but the reaction from the stays

will nevertheless be dominant. Thus, the effective length of the mast in buckling will not be that of a simple cantilever, twice times the height ($2H$), but equal to the height (H). This effect is illustrated in Figure 14.15. With the harp layout of stays, the loads will be applied at various levels along the pylon, but similar principles apply. An example of the harp layout is the Theodor Heuss Bridge (Beyer and Tussing, 1955), where one slender strut supports each cable plane. When considering the buckling behaviour of such a pylon, allowance must be made for constructional inaccuracies; typically an eccentricity of 100 mm of the stay reaction is adopted.

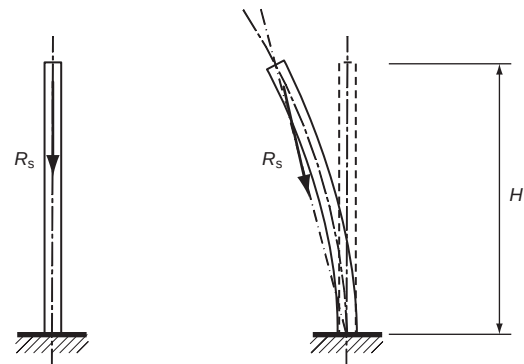


Figure 14.15 Transverse deflection of a simple pylon

In the longitudinal direction, the main and back stay cables will restrain the pylon against buckling, providing the deck, to which the stays are anchored, is adequately restrained against longitudinal movement. The pylon behaviour when the deck is allowed to float is illustrated in Figure 14.16. When the deck is unrestrained, any disturbing forces can only be resisted by the pylon acting as a cantilever, with maximum bending at the base. Thus, the effective length of the mast in buckling will be twice the height ($2H$). When the deck is effectively restrained, at either an abutment or at one of the pylons, the top of the mast is held in position by the stays and the effective length of the mast in buckling will be approximately 0.7 multiplied by the height. Connecting the back stays to an independent gravity anchorage is an equally effective solution. Early cable-stay pylon designs, such as for the Strömsund Bridge (Wenk, 1954), incorporated a pin at the pylon foot, to ensure that the mast did not have to be designed for large bending moments. Later designs have adopted a fixed-end cantilever mast, which is simpler and also more stable during erection. Nevertheless, the effect of a frictionless bearing can still be achieved with a fixed-end mast providing the member is sufficiently slender, such that the maximum axial load approaches the buckling capacity of the mast in free cantilever. As the axial load increases to one-eighth of the buckling capacity, the location of the maximum moment will move up from the base and the base moment will tend to zero. In this condition, the mast will offer no resistance against a longitudinal displacement at the top. In practice, the adoption of a particular slenderness may be limited by the need to maintain stability of the pylon during construction, when restraint from the stays is not available.

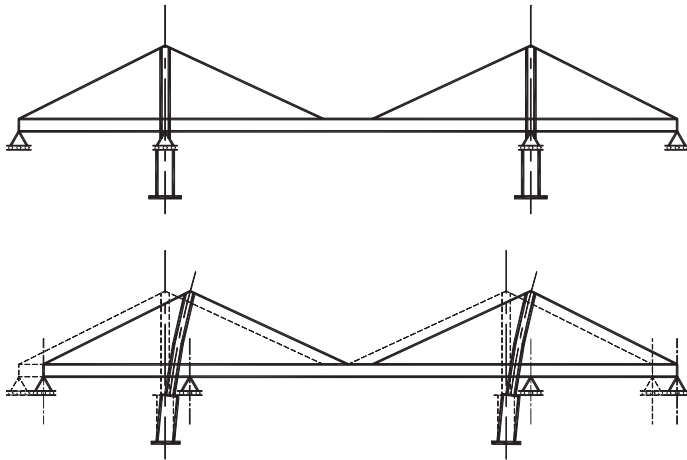


Figure 14.16 Longitudinal restraint of the pylon by the anchor stays

For the single mast pylon supporting a single plane of stays, two methods have been used to connect the mast at its base.

- It can be constructed encasté into a transverse girder forming part of the deck. In this case, a bearing is required on top of the pylon foundation immediately beneath the mast, with two further bearings at each end of the transverse girder so as to provide the necessary torsional restraint to the deck and pylon forces.
- Alternatively, the mast can pass directly through the deck to sit on the pylon foundation. In this case, only bearings at each end of the transverse girder are required and these only have to provide the resistance to the deck forces.

The second method is the more efficient of the two and has been universally adopted in more recent designs. An example of the use of this method of support is in the design of the Rama IX Bridge (Gregory and Freeman, 1987).

Concrete pylons

Concrete is very efficient when supporting loads in axial compression. Advances in concrete construction and modern form-work technology have made the use of concrete increasingly competitive for pylon construction, despite the much greater self-weight when compared with a steel alternative. Concrete has proved particularly adaptable to the more complex forms of pylon. Many varied types of pylon have been developed to support both the vertical and inclined stay layouts. These include H-frame, A-frame and inverted Y-frame pylons, as illustrated in Figure 14.17.

With the H-frame pylon, the stay anchors are normally located above the level of a cross-beam; with the modified fan arrangement of stays, this cross-beam location would be between midheight and two-thirds of the pylon height above the deck. When the harp arrangement of stays is adopted, the anchors are distributed over the full height of the pylon above the deck. Therefore, as in the case of the Øresund Bridge

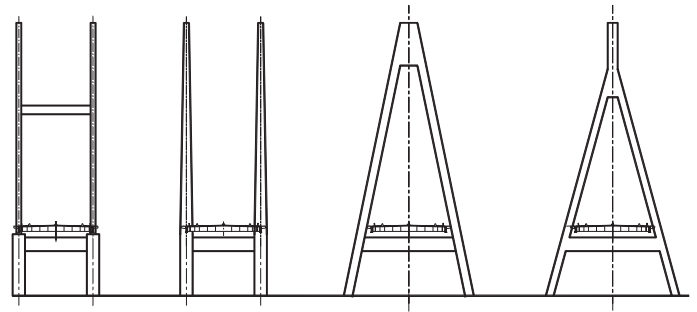


Figure 14.17 H-frame, A-frame and inverted Y-frame pylons

between Denmark and Sweden, a cross-beam can only be provided, in practice, below the deck level.

The deck section located at the pylon is usually the most highly stressed section, combining maximum negative moment and maximum axial load. When the stay is connected directly between the pylon leg and an edge stiffening girder within the deck section, it is necessary to inset the pylon legs into the deck. In addition to this practical detailing problem, a notch is created, giving a zone of concentrated stress at an already highly stressed section.

There are several geometrical configurations that overcome this problem: the pylon can be widened and the stays connected to the deck by means of an outstand bracket on the deck; or the pylon leg can be sloped outwards at its base. The leg may be inclined over the entire height of the pylon, in which case the pylon must be designed for a small eccentricity arising from the stay cable reactions. Alternatively, the upper section of the leg can be maintained in a vertical plane and the pylon inclined only from below the level of the bottom anchorage. Locating the cross-beam at this change of direction conveniently ensures that the stay force reaction is transmitted as a direct thrust. Examples of this pylon geometry can be seen in the Alex Fraser Bridge over the Fraser River, Canada, and the Vasco da Gama Bridge (Capra and Leveille, 1998) over the Tagus River, Portugal, as illustrated in Figure 14.18.



Figure 14.18 Vasco da Gama Bridge, Portugal (courtesy of Yee Associates)

Pylon geometry

The A-frame pylon is suitable for inclined stay arrangements and was first adopted for the Severin Bridge (Fischer, 1960). A variation of the A-frame is the inverted Y-frame, where the vertical leg, containing the stay anchors, extends above the bifurcation point. Further examples of the inverted Y-frame are the pylons for the Pont de Normandie over the River Seine, France, and the Rama VIII Bridge, Bangkok, Thailand. Excessive land take, owing to the wide pylon footprint, can occur with this form of pylon, when a high navigation clearance to the deck is required. This has been overcome by breaking the pylon legs at or just below the deck to produce inward-leaning legs to the foundation to form a diamond configuration, as shown in Figure 14.19. However, this modified arrangement is considerably less stiff when resisting transverse wind or seismic forces and this can result in a significant increase in the deflection of the pylon. This deflection can be mitigated only with a considerable increase in the stiffness of the lower section of the pylon leg below the deck. Nevertheless, this arrangement was favoured for the pylons of the Tatara Bridge in Japan, at 890 m, and the Industrial Ring Road Bridges in Bangkok (Figure 14.20). This configuration was also adopted, in a distinctive tandem configuration, for the twin cable-stayed crossing of the Houston Ship Channel (Svensson, 1999). By connecting the twin diamonds and tying them together at deck level, a strong truss was created, which transmits the transverse wind loads to the foundations (see Figure 14.21).

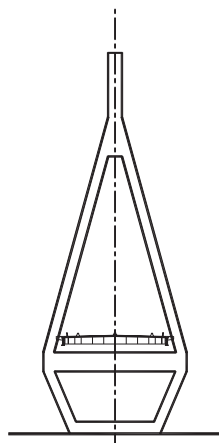


Figure 14.19 Diamond pylon

It is also possible to incline the pylon in the longitudinal direction; many visually exciting structures have incorporated such a pylon as an architectural feature. However, the resultant inclined thrust from the pylon must be carried by the foundation and a significant horizontal component will be developed. When the structure is founded in rock, these horizontal reactions can be easily resisted with only a small displacement of the foundation. However, in typical estuarine soil conditions, the foundation costs may represent a significant proportion of the overall project cost.



Figure 14.20 Industrial Ring Road Bridges, Thailand (courtesy of Ian Martin)

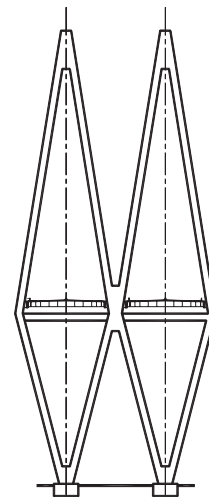


Figure 14.21 Twin diamond pylon, Houston Ship Channel, USA

Stay connection

In early designs, the connection between the stays and the pylon was formed in the same manner as for suspension bridges, where the cables are laid in a deviator saddle and carried through the pylon. This form of connection had limitations, as discussed in the section on fan cable systems. Alternative designs, such as the modified fan and harp arrangements with stays anchored over the upper section of the pylon leg, led to the use of separate stays for the main and back spans.

The most direct form of anchoring is to attach the stay socket or anchorage plate to the wall of the pylon. In this layout, the hollow pylon shaft gives access to the stay anchors for stressing during erection and inspection or replacement in service. In the case of the concrete pylon, the horizontal component of the cable forces will tend to split the shaft vertically and transverse prestressing is required to resist these forces. A typical layout of the prestressing, as adopted for the Helgeland

Bridge (Svensson and Jordet, 1996) in Norway is shown in Figure 14.22.

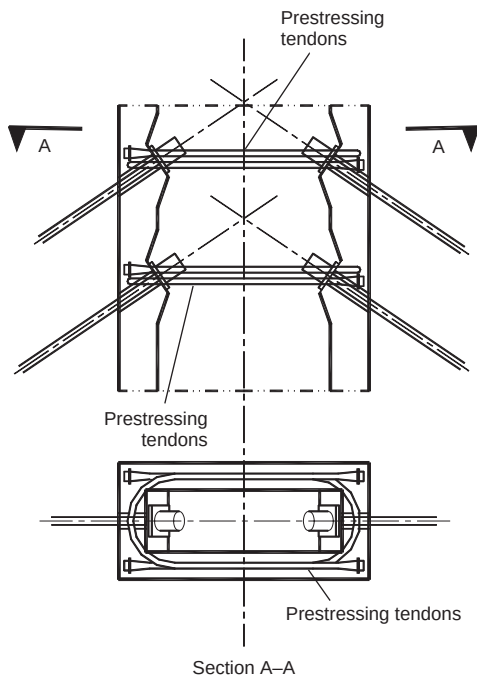


Figure 14.22 Prestressing layout for stay connection to concrete pylon

An alternative arrangement, producing a more slender pylon, allows the main and back span stays to cross so that they are anchored in rebates on the reverse sides of the pylon, as illustrated in Figure 14.23. The horizontal component of the cable forces will place the pylon in compression. However, the two stays cannot be in the same plane and the pylon must be

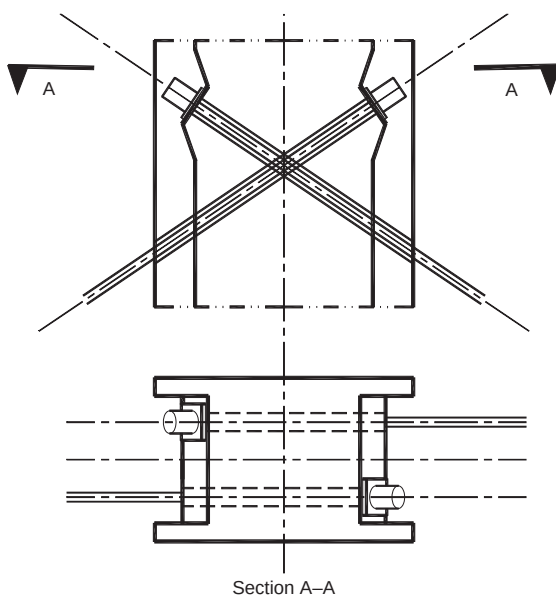


Figure 14.23 Alternative layout for stay connection to concrete pylon

designed for the resulting torsion arising from this eccentric loading. This is a detailing problem that can be overcome by dividing the stay on one side of the pylon into a pair of stays and creating a balanced anchor layout, as shown in Figure 14.24.

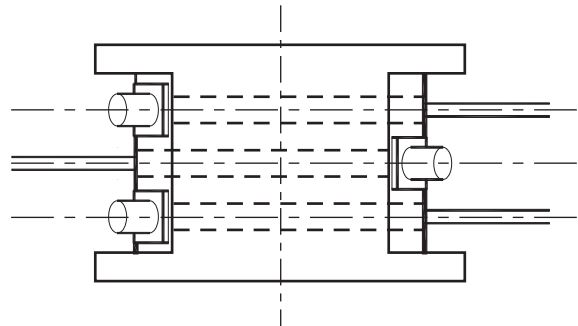


Figure 14.24 Alternative connection with balanced stay layout

For the design of the Alex Fraser Bridge over the Fraser River, a different approach was employed, where a steel fabrication was placed within the hollow pylon shaft at the level of each stay anchor to transfer the horizontal component of the stay force. The vertical component and any differential horizontal component were transferred through a corbel cast on the inside of the shaft wall, as shown in Figure 14.25.

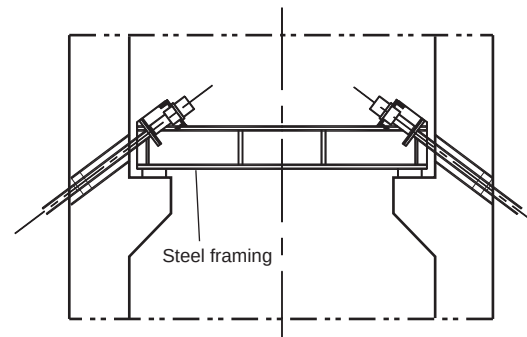


Figure 14.25 Stay connection with steel frame in concrete pylon

All these methods of connecting the stay to the pylon rely on the accurate placement of the steel formers and any anchor prestress and reinforcement within the concrete walls if the stay geometry and the strength of the connection intended in the design are to be realised. The complexity of the required details will often slow the progress of the erection throughout this critical zone of the pylon construction. To mitigate these problems, steel fabricated anchorage modules have been manufactured such that the required stay anchorage geometry is completely defined. This module can then be incorporated in the concrete shaft during its construction. Adequate shear connection, usually in the form of shear studs, is provided so that the concentrated anchorage forces in the fabrication can be transferred to the concrete shaft. Examples of this form of pylon construction, where the fabricated anchorage module is located centrally within the concrete shaft, are the Pont de

Normandie and the Stonecutters Bridge. A similar concept was incorporated in the pylon of the Ting Kau Bridge, Hong Kong. However, here the fabricated anchor modules were connected on the outside of the concrete core, as shown in Figure 14.26.



Figure 14.26 Fabricated anchor modules, Ting Kau Bridge, Hong Kong (contains photographs and information with the permission of the Highways Department of the Government of the Hong Kong Special Administrative Region; all rights reserved)

It is essential that any eccentricity of the stay anchor within the pylon is accurately modelled as part of the analysis of the structure. When the inclinations of the back and main stay cables are identical, with both anchors at the same level, the axes of the stay, and hence the stay forces, will intersect on the pylon centre line. However the inclinations of the back- and main-span stays are rarely identical; hence, the anchors must be located at different levels if the same intersection line is to be maintained. Alternatively, when the levels of the two anchors are maintained at the same level, the vertical result-

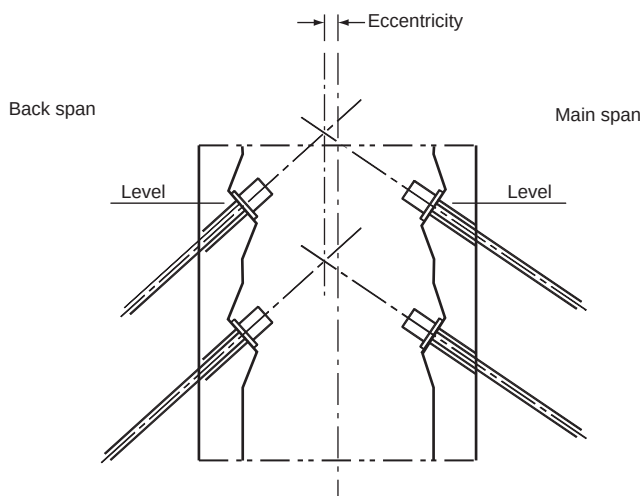


Figure 14.27 Stay anchor pylon geometry

ant of the stay forces will be slightly eccentric to the pylon. This approach is usually preferable as it will simplify the detailing of the anchor zone but the pylon moments arising from this small eccentricity need to be considered in the design, as shown in Figure 14.27.

Deck

Rather than being merely supported by the cable, as in the suspension bridge form, the deck of the cable-stayed bridge is an integral part of the structure, resisting both bending and an axial force derived from the horizontal component of the stay force.

The economic solution for the suspension bridge is a minimum-weight deck section. With the cable-stayed bridge, the greater participation of the deck in the overall structural behaviour gives the opportunity to consider alternative deck forms, particularly with concrete being an efficient material when used for compressive members. As such, cable-stayed bridge deck forms have been developed as

- a steel section incorporating an orthotropic road deck
- a concrete section
- a composite steel and concrete section.

Steel deck section

The early designs for cable-stayed bridges had relatively few stays, requiring the use of a light yet rigid deck capable of spanning the wide stay anchor spacing. This structural arrangement, incorporating an orthotropic road deck, ideally suited the all-steel construction. With the introduction of the multi-stay cable systems, the steel deck solution, for moderate spans, has become less economic. Nevertheless, for the longest spans, where the reduction in deck weight is an economic imperative, steel construction has been retained for structures such as the Pont de Normandie, Stonecutters Bridge and the Tatar Bridge.

The orthotropic road deck consists of a thin surfacing material laid on either a 12 mm or 13 mm steel plate, stiffened longitudinally. The first designs incorporated longitudinal stiffeners, of an open bulb flat section, at approximately 300 mm spacing. Later designs have adopted the closed trough stiffener, fabricated from folded plate, giving greater torsional rigidity to the deck plate system. The deck stiffeners are supported by transverse floor beams at 3–5 m spacing.

The design of the deck cross-section is dominated by the arrangement of the stays. Where a single central plane of stays is adopted, the torsional resistance of the deck section is the only means of carrying any eccentric loading; therefore, a strong torsion box must be provided. The Erskine Bridge (Kerensky *et al.*, 1972) over the River Clyde in Scotland is an example of a steel box with only one central stay. In this

case, the girder has to span in excess of 100m between the cable-stayed anchor points. A common arrangement, originated by German designers, is to divide the cross-section of the torsion box into three or five cells. The central cell is the same width as the pylon, so that the cable stays may be readily anchored within it. Examples of this arrangement are the Rama IX Bridge, Bangkok, which is divided into three cells, and the Oberkasseler Bridge over the River Rhine, Germany, which is divided into five cells. The cross-section of the deck of the Rama IX Bridge is given in Figure 14.28.

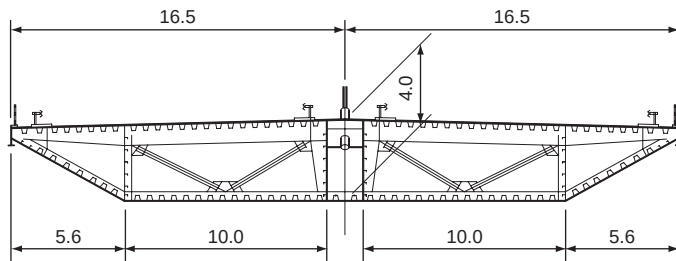


Figure 14.28 Multi-cell steel torsion box deck – Rama IX Bridge, Bangkok (all dimensions in metres)

With highway structures of moderate span and where two planes of stay cable are used, the rigidity of the torsion box deck cross-section is unnecessary and it is possible to simplify the section to that of twin longitudinal girders. Early designs, such as the Knie Bridge over the River Rhine at Düsseldorf, Germany, with a main span of 320m, combined longitudinal steel plate girders with an orthotropic deck. This bridge is illustrated in Figure 14.29.

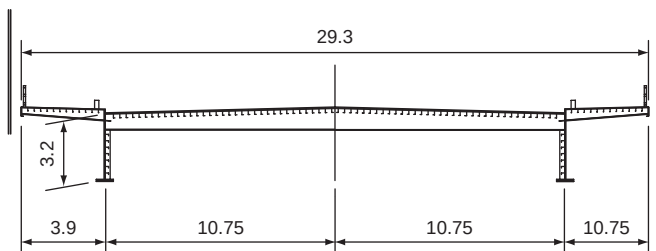


Figure 14.29 Twin girder steel deck – Knie Bridge, Germany (all dimensions in metres)

When two planes of stays are connected to each edge of the deck, eccentric loading of the deck can be carried by the cable system. However, the strong torsion box can still be beneficial when a highly eccentric loading is carried, such as where both road and rail are running on the deck and the heavy rail loading has to be located eccentric to the deck section. Such a deck will improve the distribution of the loads between the two cable planes. The torsion box can also be more easily adapted into a streamlined shape, essential for reducing wind drag and for improving aerodynamic stability in very long spans.

Composite deck section

The orthotropic deck, with its high manufacturing costs, has a cost penalty when compared with an equivalent concrete slab road deck. The cost advantage of the composite concrete slab over the steel deck is, to a limited extent, offset by the larger area of cable stays and the additional pylon and foundation costs required to support the heavier dead loads. German codes, at the time of the early cable-stayed designs, did not permit tensile stresses in a concrete road slab and the provision of the necessary post-tensioning near the middle of the span was not considered economic. In most cases, the tensile stresses within the concrete deck slab are small. When the deck is spanning transversely between two planes of cables at the deck edge, the slab will act as the top flange of a simply supported beam and will remain in compression. Longitudinally, any tensile stresses due to bending are small because the neutral axis of the section is located close to the underside of the slab and will, for the majority of the span, be smaller than the compressive component from the stays. Even at the centre of the span, where the compressive component reduces to zero, it is possible to introduce a sagging moment by adjusting the forces in the stays, so maintaining compression in the composite slab.

When considering the cantilever erection of the composite deck, two sequences are possible.

- The relatively light steel framing can be erected first, allowing the stay to be attached and partially stressed before forming the concrete deck, either as in situ concrete or as precast panel units. An early use of an in situ concrete deck was in the design of the Vidyasagar Setu, India. The use of precast deck panels minimises the amount of work during the critical erection phase and reduces the effects of shrinkage and creep that must be allowed for in the design. The manufacture of the precast panels can be programmed for 2 to 3 months in advance of the deck erection. The precast deck panels sit on a neoprene strip on the edge of each beam and are made structurally continuous by overlapping hairpin reinforcement within an in situ concrete strip over the top flange of the beams. This method of construction was first adopted for the deck of the Alex Fraser Bridge over the Fraser River, Canada. More recently, it was the method preferred by the contractor for the construction of the Houston Ship Channel Bridge and the Rama VIII Bridge, Bangkok.
- Alternatively the concrete deck may be cast in the assembly yard with each deck module using a matchcasting process alongside the neighbouring module. This ensures that lapping reinforcement is accurately aligned across the joint between each module. By programming the works in advance of the deck erection, the procedure also reduces the effect of shrinkage and creep within the design and confines the use of in situ concrete construction to the transverse stitch joint. Examples of structures incorporating this method of construction include the Kap Shui Mun Bridge, which is part of the Lantau Link in Hong Kong, and the Prince of Wales Bridge over the River Severn, UK (Mizon *et al.*, 1997). A further example of a composite deck section, formed by the matchcasting process, is that of the Industrial Ring Road Bridges over the Chao Phraya River, Bangkok, illustrated in Figure 14.30.

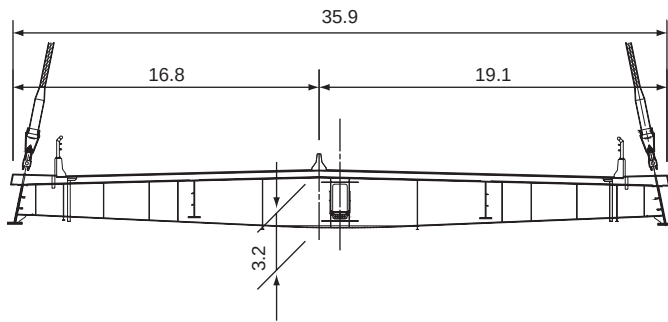


Figure 14.30 Twin girder composite deck – Industrial Ring Road Bridges, Bangkok (all dimensions in metres)

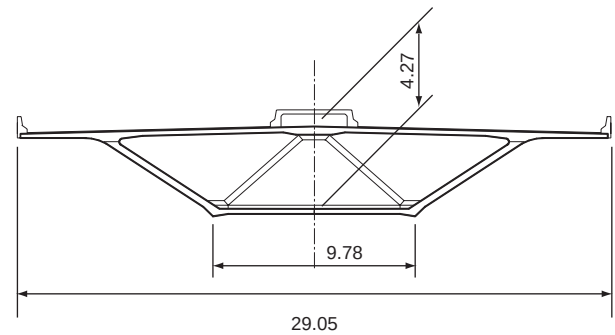


Figure 14.31 Concrete torsion box deck – Sunshine Skyway Bridge, USA (all dimensions in metres)

Concrete deck section

Concrete deck sections were subject to developments similar to those for steel deck sections. The torsion box deck sections are commonly used in conjunction with a single central plane of stays. The first such design was the Brotonne Bridge crossing the River Seine near Rouen, France. A similar design, using precast segmental units for the deck, was adopted for the Sunshine Skyway Bridge, USA, as illustrated in Figure 14.31. In common with a number of major bridge projects in the USA, this design was selected after a process of competitive pricing between alternative steel and concrete designs.

Concrete designs, in common with the evolution of the composite deck section, have developed a simplified deck form. Examples of this deck construction are the Dames Point Bridge over the St. Johns River in Florida, USA, which is illustrated in Figure 14.32, and the Helgeland Bridge, Norway (Svenssen

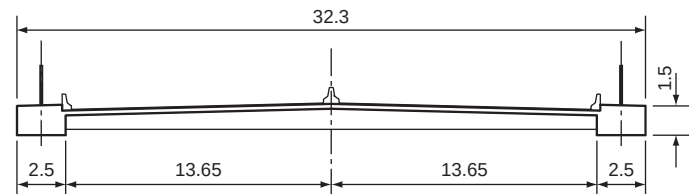


Figure 14.32 Twin beam concrete deck – Dames Point Bridge, USA (all dimensions in metres)

and Jordet, 1996). In a typical arrangement, the transverse floor beams are at 3–5 m centres supporting an in situ concrete road deck. The transverse beams on the Dames Point Bridge took the form of precast T-beams. The longitudinal beams are located at each edge of the deck, centrally beneath the cable planes, and incorporate the stay anchors. Erection is by casting the deck in segments as a free cantilever using a form traveller. The stay is initially stressed against the form sufficiently to minimise

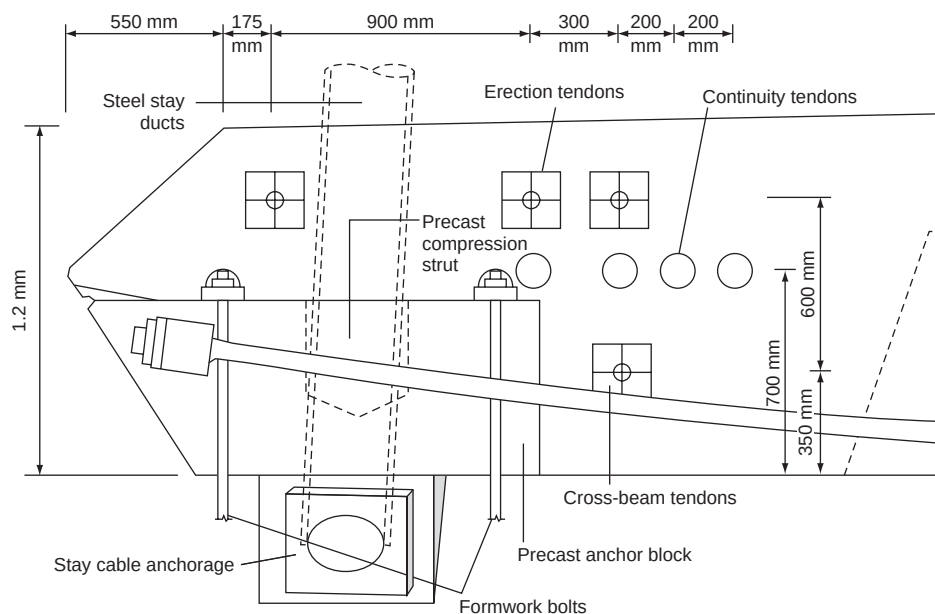


Figure 14.33 Precast anchorage and edge beam – Helgeland Bridge

deflection during the placing of the concrete. The design of the Helgeland Bridge allowed for the precasting of a portion of the longitudinal edge girder, including the stay anchor tube, and then incorporating this element within the deck concrete (Figure 14.33). This system improves the accuracy of the stay anchor geometry and allows stressing of the stay anchor at an earlier stage in the erection cycle. Thus, the stay anchor for this bridge is stressed in three stages: prior to concreting, immediately after concreting and when the transfer strength of the in situ concrete was achieved.

The Helgeland Bridge, which is illustrated in Figure 14.34, has an exceptionally narrow deck at 11.95 m; therefore, the design of the in situ concrete cross-beams will not be a critical part of the design. However, as the width of the deck increases, this form of concrete deck construction becomes less economic. In the case of bridge deck widths suitable for dual three-lane highways – that is, at least 30 m wide – the weight of the transverse floor beams becomes dominant and significant additional stay area must be provided to carry the extra dead load. One method of mitigating this problem, while still maintaining the simplicity of the concrete edge beam with a cast-in stay anchor, is to provide a lighter composite plate girder as the transverse floor beam. This is the design philosophy adopted for the deck of the Vasco da Gama Bridge (Capra and Leveille, 1998) over the Tagus River, Portugal. The deck section for this bridge is illustrated in Figure 14.35.



Figure 14.34 Helgeland Bridge, Norway (courtesy of Leonhardt, Andra und Partners)

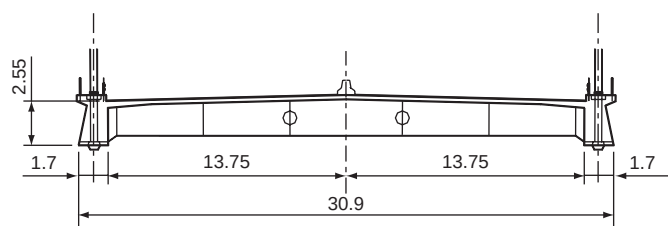


Figure 14.35 Combined concrete and steel deck section – Vasco da Gama Bridge, Portugal (all dimensions in metres)

Design principles

As can be seen, there are a number of possible deck forms; when used in combination with an appropriate stay arrangement, they can provide an economic design within their respective span ranges.

The deck is required to perform a number of particular functions within the overall structural system.

- Distribute the applied loading to the stay anchor points. With dead load and uniform live load, this will take the form of the transverse distribution of load to each stay plane and between each stay anchor. For a deck supported by a multi-stay system, concentrated live loads will be distributed between a number of stays as a beam with elastic supports. It should be noted that the moments in the deck will decrease as the longitudinal stiffness of the deck decreases.
- Act as part of the global system with the stays and pylon. The primary force is the axial force component from the stay force and the deck must be sufficiently stiff to resist buckling from this compression. The capacity of the deck must be checked using a non-linear, second-order analysis. In addition, the deck may be required to resist bending moments induced by unbalanced loading within a harp cable-stay system, as described in the section on harp cable systems.
- Resist transverse wind loading and transmit the reactions to the substructure.

Thus, where the stays are widely spaced, the design of the deck is dominated by longitudinal bending; where there is a single central stay plane, torsion effects may dominate. For multi-stay systems, the design of the deck is dominated by transverse bending, combined with thrust and longitudinal bending derived from the global system.

The requirements for the deck design vary between the main and back spans. The main span must be designed to give an acceptable aerodynamic performance; for economy, the self-weight of the deck section should be a minimum consistent with the choice of material. Table 14.1 gives comparative estimates of deck weight.

Deck type	Weight: kN/m ²
Steel deck	2.5–3.5 kN/m ²
Composite deck	6.5–8.5 kN/m ²
Concrete deck	14.0–15.0 kN/m ²

Table 14.1 Estimates of deck weight for various materials

The back span, through the stays, must stabilise the pylon when unbalanced by live loading within the main span and transmit any resulting uplift to the approach span piers. For any bridge constructed with an all-steel deck section for the main and back spans, a means of transferring significant uplift must be provided. This has conventionally been achieved by the use of a pendel linkage that transfers the uplift force but

still allows longitudinal translation to take place in response to temperature changes. An example of this arrangement is shown in Figure 14.36 which is similar to that used within the back spans of the Queen Elizabeth II Bridge over the River Thames at Dartford, UK.

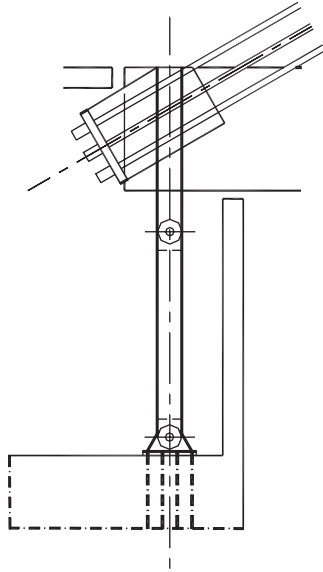


Figure 14.36 Pendel linkage over anchor pier

It is clear that the back span deck does not have to comply with the criterion of minimum self-weight. Provided that separate land access is available for the erection of the back span, such that the main and back spans do not have to be erected in balanced cantilever, there is no reason why the back span should not be constructed using a different material from that in the main span. This arrangement was fully exploited for the construction of the Pont de Normandie over the River Seine. In this case, the central 624 m of the 856 m main span was an all-steel construction, while the remainder of the main span and the back spans were of concrete construction. The Normandie deck was encastred with both the pylons; thus, any strains due to temperature variation were accommodated by flexure within the lower section of the pylons and by variation in curvature of the deck profile.

Where the back span is over land, it can be constructed in advance of the main span, possibly as a continuation of an approach viaduct. This can provide both programme and construction advantages during erection. The back spans can be constructed at the same time as the pylon so that it forms a high-level access platform at the time the cantilever erection within the main span starts.

Preliminary design

The cable-stayed bridge, incorporating a number of stays, is a highly redundant structure where the deck acts as a continuous beam with a number of elastic supports with varying stiff-

ness. The deck and pylon of the cable-stayed bridge are both in compression; therefore, bending moments in these elements will be increased, owing to second-order effects, arising from the deflection of the structure (the *P*-delta effect). With most cable-stay structures, these secondary moments will not exceed 10% but the application of these moments will be non-linear. This means that influence lines, which rely on the principles of linear superposition, can only be used as an approximate method of determining the stay loads.

Non-linear material properties will also influence the design. Apart from the behaviour of the stays under load, discussed separately in the section on stay behaviour, all concrete and concrete-steel composite decks will be subject to the effects of creep and shrinkage during both construction and the service life of the completed structure. It can therefore be seen that a preliminary design by manual calculation should be considered as the first stage in an interactive design process, providing a basis for a more rigorous analysis.

Ratio of back span to main span

When establishing the conceptual arrangement of the bridge, it is important that the ratio between the back span and the main span is less than 0.5 in order to give a clear visual emphasis to the main span. This ratio is equally as important structurally, as it influences the uplift forces at the anchor pier and the range of load within the back stay cables supporting the top of the pylon. The back stay cables have the largest stress amplitude and may therefore be critical when considering the fatigue endurance of the stays. Live load located within the main span will increase the anchor forces within the back stays, while live load within the back span will decrease the anchor forces. Where there are no intermediate piers supporting the back span and no physical constraints imposed by the terrain, the foundations or any other requirements dictating the location of the abutment pier, this ratio can be determined by the balance of the live load moments in the main span. Leonhardt and Zellner (1980) have determined the ratio of the back span to the main span with respect to these parameters. For a highway structure where the live loading is typically 0.25 of the dead load, the theoretical ratio is 0.38. However, this calculation ignores the bending stiffness of the deck. When this stiffness is taken into consideration, the optimum length of the back span is more likely to be between 0.4 and 0.45 of the main span.

Conversely, the optimum ratio for a structure carrying a heavy railway loading, where the live loading may be equivalent to the dead load, would be 0.18. Again, when the bending stiffness of the deck is taken into consideration, the length of the back span is more likely to be between 0.2 and 0.25 of the main span. Live load bending moments in the deck are likely to be a maximum near the end of the side span. Significant rotations are therefore possible at any expansion joint that is located at the anchor pier. While this may be acceptable for a highway bridge, a train crossing such an expansion joint at speed could experience an unacceptable level of acceleration

caused by the sudden change in gradient across the joint. It is preferable in this case to provide continuity over the anchor pier by extending the deck into a small approach span sufficient to attenuate the live load rotations within the suspended back span.

To summarise, it can be seen that the optimum ratio of main span to side span is sensitive to the proportion of live load to dead load. Of course, this will vary depending on whether a concrete, steel or composite deck is being proposed. For a bridge with a steel superstructure, the length of the back span will be smaller than for a similar span bridge with a concrete superstructure.

Where there is ready access to the land beneath the back span, it may be possible to construct this part of the superstructure as an extension of any approach viaduct rather than as a balanced cantilever from the pylon. The back span will then be supported by intermediate piers that provide a direct anchorage for the back stays. This is of particular benefit in bridges with a harp arrangement of stays, as the additional stiffness will reduce the moment in the pylon normally associated with this stay arrangement. By constructing the back spans early in the construction schedule, independently of the pylon construction, early direct access to the pylon is possible and a platform is available from which the fabrication and erection of the stays and cantilever erection of the main span can proceed. This will usually be of benefit to the overall construction schedule.

Stay spacing

The spacing of the stay anchors along the deck should be compatible with the capacity of the longitudinal girders and limit the stay size so that the breaking load is less than 25–30 MN. The capacity of the longitudinal girders is likely to be critical when considering the case of an accidental severance of a stay (stay out condition). The spacing should also be small enough so that the deck may be erected by the free cantilevering method without the need for auxiliary stays or supports. These requirements will effectively limit the spacing to within the range 5–15 m. The heavier concrete construction will require the smaller stay spacing, while the larger stay spacing is more suitable for steel or steel composite construction.

Deck stiffness

The deflection of the longitudinal girders is primarily determined by the stay layout. It is reasonable, therefore, that the depth of the longitudinal girders should be kept to a minimum, subject to sufficient area and stiffness being provided to carry the large compressive forces without buckling. In checking the longitudinal girders for the stay out condition, the PTI (2001) recommendations stipulate that the structure should provide for the replacement of any individual stay with a controlled reduction of the live load during any stay exchange. The structure must also be capable of withstanding the accidental loss of any individual stay without structural instability occurring.

Pylon height

The height of the pylon will determine the overall stiffness of the structure. As the stay angle (α) increases, the required stay size will decrease and the height of the pylon will increase. However, the deflection of the deck will increase as each stay becomes longer. Both the weight of the stay and the deflection of the deck become a minimum when the expression $1/(\sin\alpha \times \cos\alpha)$ is also a minimum. Therefore, the most efficient stay is that with a stay inclination of 45° . In practice, the efficiency of the stay is not significantly impaired when the stay inclination is varied within reasonable limits, which may be taken as 25° – 65° . The stay inclined at 25° will be the outer stay connecting the anchor pier and the deck panel adjacent to the centre of the main span to the top of the pylon. The stay inclined at 65° will be that located nearest the pylon. This implies an optimum ratio of pylon height above the deck (H) to main span (L) of between 0.2 and 0.25, as illustrated in Figure 14.37.

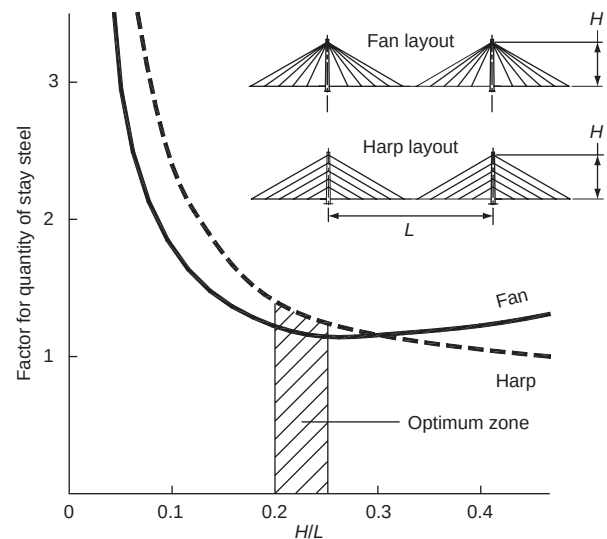


Figure 37 Optimum pylon height

Preliminary stay forces

The main-span stay forces resist the dead loads such that there is no deflection of the deck or pylon; the vertical components due to these loads are therefore known. By assuming that the live loads act in a similar manner, an initial approximation of the stay force can be determined by considering the structure as a simple truss, ignoring the bending stiffness of both the pylon and the deck. Thus, the main-span stay forces (P_{mi}) at an angle (α_i), as shown in Figure 14.38, can be determined as

$$P_{mi} = (W_{DL} + W_{LL}) / \sin \alpha_i \quad (14.6)$$

and the horizontal stay component (F_h) is given by

$$F_h = (W_{DL} + W_{LL}) / \tan \alpha_i \quad (14.7)$$

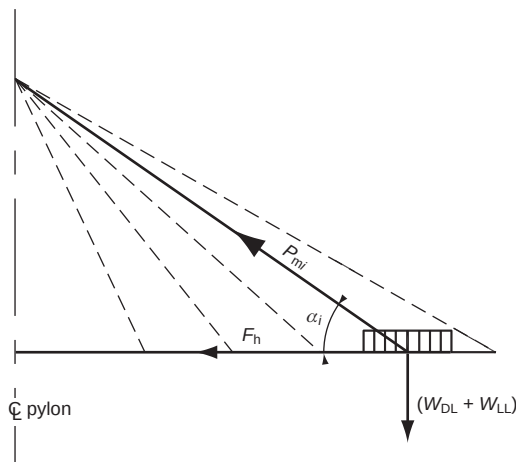


Figure 14.38 Main span stay force diagram

The back stay anchoring forces can be calculated by assuming that the horizontal component of the stay force is balanced at the pylon and ignoring the bending stiffness of the pylon. In most cases, this will be a valid assumption as the bending stiffness of the pylon is usually small compared with the axial stiffness of the stays. The back stay forces (P_{bi}) at an angle (β_i), as shown in Figure 14.39, can therefore be determined as

$$P_{bi} = F_h / \cos \beta_i \quad (14.8)$$

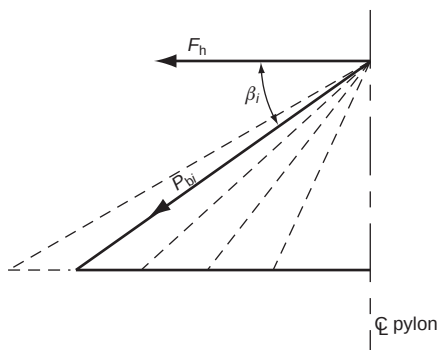


Figure 14.39 Back span – stay force diagram

This linear model is an important first step in the initial sizing of the stays but does not consider the effect of the self-weight of the stay. As the weight of the cable is a gravity effect, the horizontal component of the stay force F_h remains constant along the length of the stay and it is the vertical component that varies. From the equations of equilibrium, the modified stay force at a point along the stay P_{mod} can be calculated as

$$P_{mod} = P + q(z - z_A) \quad (14.9)$$

where q is the linear weight of the stay and $(z - z_A)$ is the difference in elevation between the height of the lower anchorage and the point being considered.

Deck form

The selection of the deck form will usually be based on an economic evaluation of the possible alternatives. The primary factors influencing the choice of deck will be the length of the main span and the deck width. Other factors, such as the cost of foundations, the local availability of materials or labour skills and the competitive conditions at the time of tendering might also influence the costs. Svensson (1995) has made an economic comparison of the various types of deck sections within the span range 200–1000 m. The study concluded that a concrete deck section is the most economic deck section within the span range 200–400 m, and a composite deck the most economic for spans above 400 m. However, the difference in cost is marginal close to the division between the two deck forms, and local factors are often decisive in the final choice. The study also does not consider the influence of any variation in the width of deck. The use of concrete construction in wide decks, where there are six or more traffic lanes, requires substantial cross-beams; the additional weight of these will penalise those spans near the upper end of the economic range. One solution, as used for the Vasco da Gama Bridge, is to design the cross-beams as steel plate girders, which are composite with the deck slab. Additional economy can be achieved, within the span range 350–600 m, by using a hybrid combination of concrete back span and composite main span. Above 600 m, a hybrid combination is still economic, but with the back span as concrete and the main span in an all-steel construction. In the case of the Pont de Normandie, the back span concrete deck was extended more than 100 m into the main span, while for the Stonecutters Bridge the welded steel main span extends into the first back span.

Deck design

For the design of the deck, it is possible, by tuning the loads in the stays, to reduce the moments in the deck, under the applied dead load, to the small local moments between stays. For the design of the steel deck, this dead load condition is applied following the structural completion of the deck and the application of all superimposed dead load. In the case of a concrete or composite deck, the time-dependent properties of the concrete must also be considered. Usually, structural completion is assumed to be at time infinity ($t = \infty$), when all creep and shrinkage related strains have ceased. However, reducing the dead load moments in the deck to purely local effects will not provide the optimum solution. This is because the balance between positive (sagging) and negative (hogging) live load moments at any section along the girder will not be equal. As noted previously, the magnitude of the moments will depend on the ratio of span lengths in the main and back spans. The limiting stresses at the top and bottom of the section will not be the same, as an allowance must be made for the effect of local bending within the road deck. The differences in material

properties, as in the case of composite construction, will also affect the location of the neutral axis of the section. In most cases, the properties of the deck section will be more favourable when resisting positive moments. The composite road slab can be kept in compression near the centre of the main span, where the normal forces are small, by inducing a positive dead load moment. Taking these factors into account, an optimum distribution of dead load moments can be determined. The design of the deck is then undertaken using an iterative process. For initial design purposes, a minimum deck section can be assumed and used to calculate the dead weights and section properties. The structural system is then analysed, incorporating a preliminary distribution of dead load moment and the sections checked against the distribution of total moments and normal forces. The sections where stresses exceed the permissible limits are then modified, the dead weights and section properties are recalculated and the distribution of dead load moment is adjusted for the revised section. The structural system is then reanalysed and the cycle repeated until convergence is achieved.

Deck erection calculations

The common method of deck erection is the successive cantilever method. With this method, it is necessary to determine the stresses in the structure at each stage during the cantilever erection to ensure that they do not exceed the design limits. Erection using the successive cantilever method will give a quite different distribution of stay loads than when the entire bridge is loaded instantaneously with the weight of the superstructure. An instantaneous application of the deck load would also cause considerable extension of the stays, giving rise to severe changes in geometry within the structure. In reality this does not occur, as the stay extension is either absorbed within the anchor head at the time of jacking or is deducted from the stay length during manufacture.

If the deck is of either concrete or composite construction, the stay load distribution will be further modified by the strains, owing to the effect of creep and shrinkage. It is therefore necessary, for calculation purposes, to also define the elapsed time from the initial casting of the various concrete pours at each construction stage.

The stay forces that are compatible with the final distribution of dead load moment and the defined structure geometry, at time infinity ($t=\infty$), are known; however, the initial stay forces introduced at each stage of the erection are not. Two methods are commonly used to determine these initial stay forces. The first method requires that the completed structure be dismantled stage by stage, as follows.

1. Using a model that represents the completed structure, remove, where applicable, any element of the creep and

shrinkage arising from time infinity to the time at the end of construction.

2. Remove the superimposed dead load, such as road surfacing or parapets.
3. Apply any forces representing the formwork or other equipment required for connecting the two cantilevers at the centre of the main span.
4. Using a model representing the completed cantilever, import the cable forces from step 3 and apply a moment at the tip of the cantilever that will give the optimum distribution of dead load moments.
5. Remove half the forces representing the formwork or other equipment necessary to connect the two cantilevers at the centre of the main span.
6. Apply the weight of the lifting gantry or travelling formwork (bridge builder) to the end of the cantilever (a typical arrangement is shown in Figure 14.40).
7. Remove the last stay.
8. Remove the weight of the last segment.
9. The lifting gantry or bridge builder is moved back along the cantilever so as to be in position for constructing the previous segment. This stay and segment are then removed in sequence.
10. Step 9 is then repeated until the entire cantilever has been dismantled.

At each stage prior to the stay removal, the initial stay force can be determined. The vertical component of the initial stay forces should be limited to less than the weight of the segment plus the weight of the lifting gantry.

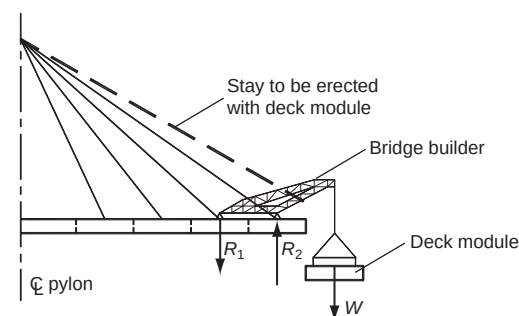


Figure 14.40 Typical cantilever erection using bridge builder

The alternative method of erection is to balance the vertical component of the stay force supporting each segment against the weight of each segment. Following the establishment of structural continuity at the centre of the main span, each stay is then adjusted to ensure that the optimum distribution of dead load moment is achieved. This method does entail additional stressing of the stays but ensures that variations in the creep and shrinkage strains arising during the cantilever erection are discounted.

Wind loading on stays

The drag force from the wind per unit length of stay is

$$F_D = \frac{1}{2} \rho U^2 D C_D \quad (14.10)$$

where ρ is the density of air (1.23 kg/m³ for standard temperature and pressure), U is the wind velocity, D is the diameter of the stay and C_D is the drag coefficient. For any circular section, the drag factor is sensitive to the Reynolds number (Re), which in turn depends on the wind velocity and the pipe roughness. For low values of Re , in the subcritical range, C_D is relatively high, up to about 1.2, while in the supercritical range C_D falls to 0.6. However, for practical purposes, most bridges operate at their maximum wind velocity in the supercritical range and C_D values of 0.7 and 0.8 are typically adopted; the French CIP (2002) *Recommendations for Cable Stays* suggest a value of 0.7, which allows for some increase in surface roughness of the stay as the stay cover ages. The drag factor adopted must also account for any protuberances incorporated on the stay cover as a stabilisation measure against stay oscillations. Cable-stayed bridges with streamlined deck shapes will commonly have a drag coefficient for the deck in the range 0.10–0.15. It is therefore common, for longer spans, that the wind action on the plane of cables will dominate. Indeed, this has provided a justification for adopting the more compact cross-section of PWS cables on the Stonecutters Bridge in Hong Kong, with a main span of 1018 m; the Tatara Bridge in Japan, with a main span of 890 m; and the Incheon Bridge in South Korea, with a main span of 800 m. All these bridges are subject to severe typhoon winds.

Static analysis

For the final analysis, the most common approach is to model either half or the entire structure as a space frame. The pylon, deck and stays will usually be represented within the space frame model as 'bar' elements. The stays can be represented

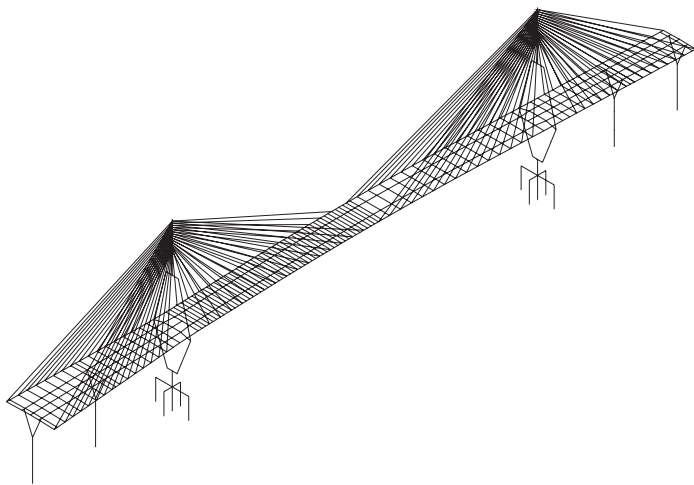


Figure 14.41 Typical space frame model

with a small inertia and a modified modulus of elasticity that will mimic the sag behaviour of the stay (see the section on stay behaviour). In addition to the analysis of the completed structure, the model can be used in the stage-by-stage erection analysis. An example of a typical space frame model is illustrated in Figure 14.41. The use of such a space frame model will ensure that the interaction between the deck, stays, pylon and piers and their foundations is accurately represented, in both the longitudinal and the transverse directions. Several commercially available computer packages incorporate the facility to consider the non-linear behaviour of a structure and are suitable for analysing cable-stayed bridges.

Dynamic analysis

Dynamic analysis is the determination of the frequencies and the modes of vibration of the structure. This information is utilised for the following aspects of the design:

- the seismic analysis of the structure
- the response of the structure in turbulent steady-flow wind
- the physiological effect of vibrations.

For a detailed review of the dynamic analysis of structures, reference should be made to Chapters 4, 5 and 6.

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Chapter 15

Suspension bridges

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This chapter provides a summary of the principal requirements for the design and construction of gravity-anchored suspension bridges with a classical three-span layout. After a brief historical introduction, available types of structural cable are described. The analysis and characteristics of suspended cables are then considered, both when acting in isolation and in conjunction with the suspended structure and towers of a complete structure. The requirements governing the overall layout and detailed design of the main cables, suspended structure, hangers, main towers, cable saddles and anchorages are then described. After consideration of the critical role of aerodynamics in the design of these bridges, construction methods are discussed, with particular reference to those for the main cables (spinning, compaction and wrapping for corrosion protection) that are unique to suspension bridges.

Introduction

Scope

This chapter describes the design and construction of the classical three-span suspension bridge configuration (Figure 15.1), with a stiffened load-carrying deck structure supported by earth-anchored cables. Unusual span or cable configurations, including bridges with more than one main span, self-anchored bridges and hybrid part suspension or cable-stayed structures are only briefly considered. The principal structural elements of these classical suspension bridges are as follows.

- Two flexible main cables formed from high-strength steel wires positioned at or close to the deck edges, which transfer the self-weight and live loads of the suspended deck structure to the supporting towers and anchorages.
- A traffic-carrying deck structure and stiffening girder supported from the main cables by steel wire rope or strand hangers. The flexural stiffness of this structure distributes concentrated traffic loadings to the cable, minimising and controlling local deflections,

and provides additional torsional stiffness to assist in preventing aerodynamically induced oscillations or instability of the deck. An economical overall design requires the lightest practicable deck structure, as savings in deck weight will lead to further reductions in the weight and cost of the cables, towers and anchorages. Modern practice, therefore, is to use a slender steel box girder deck structure and an orthotropic steel deck plate with thin asphalt surfacing. The low drag of a such a box girder has the additional advantage of minimising lateral wind loading.

- Towers of either steel or reinforced concrete to support the main cables at a level determined by the main-span cable sag, combined with the clearance required above the waterway or other obstacle being crossed.
- Anchorages to secure the ends of the main cables against movement. These structures are usually massive concrete structures that resist the horizontal component of the cable forces, principally by base friction; good ground conditions are therefore important, as otherwise very deep and costly foundations are required. If the anchorages can be positioned where there is strong unfaulted rock, tunnelled anchorages can be used to transfer the cable load directly into the surrounding rock mass.

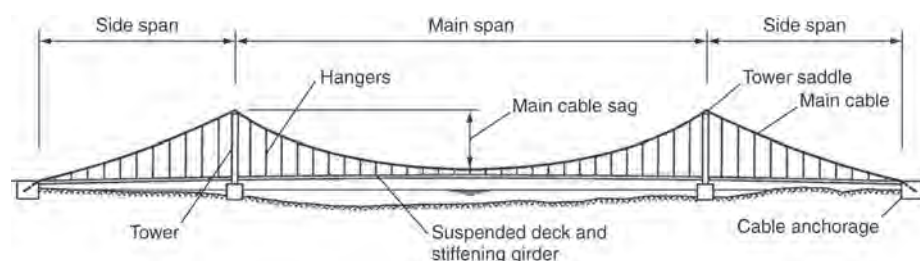


Figure 15.1 Typical three-span bridge

The advantage of cables as the primary load-carrying element is the structural efficiency derived from carrying applied loading by pure tension forces acting on the whole cross-section, so that the much higher tensile strength of cable wire relative to conventional weldable structural steel can be fully utilised. The quantity of material required for the supporting cables is therefore substantially smaller than would be required for a beam-type structure of

comparable span and loading. This advantage is, however, somewhat reduced by the following.

- A separate deck structure is needed to carry traffic.
- Owing to the cable sag, the end points of the main-span cable are at a high level, requiring tall towers above deck level.
- The cables must continue beyond the main supporting towers to transfer the cable tension down to a level at which its horizontal component can be efficiently resisted.
- Large and heavy anchorage foundations are required, to resist the cable forces and transfer these forces to the ground.

As the advantage in the relatively smaller quantity of primary load-carrying material will become more significant as span length is increased, it can be expected that cable-supported bridges are most likely to be the preferred structural form only for longer spans. Although suspension bridges were once the predominant long-span bridge type for spans exceeding 500 m, cable-stayed structures are now generally a more economical solution up to spans of around 1000 m, with suspension bridges remaining as the unchallenged choice only for spans significantly above this.

However, the natural curve of the supporting cables combined with a slender suspended deck and vertical towers produces a visually attractive structure that can also be an appropriate choice, even for relatively short-span footbridges.

The evolution of the modern suspension bridge

Although the idea of crossing a gap using a suspended flexible rope as both a support and a platform to carry pedestrian traffic appears to have originated in ancient times and was used by early civilisations in Central and South America and the Far East, the evolution of the modern form of suspension bridge with a suspended level traffic-carrying deck commenced during the Industrial Revolution and has required a series of developments in cable construction, understanding of the structural behaviour and the effects of wind, and the design of deck structures to become the most efficient bridge structure for spans in excess of 1000 m.

Cables

The first development was the availability of wrought iron that could be formed into relatively long flat bars, which when linked with pin connections, could be formed into a main cable. Although these relatively low-strength wrought iron links limited the spans that could be achieved, the durability of these chain-type cables made them the preferred choice for many early long-span bridges of the Industrial Revolution, a typical example being the 176 m span Menai bridge, completed in 1826.

The first major bridge supported by wire cables was the 273 m span Grand Pont Suspendu across the Sarine valley at Fribourg in Switzerland, completed in 1834. However, the construction of parallel wire cables compacted into a wrapped circular shape dates from Roebling's development of aerial in situ spinning, which, with the availability of wire with improved strength and quality, enabled him to construct in the USA a series of bridges with increasing span lengths, culminating in 1883 with the 486 m span Brooklyn Bridge. With progressive development and refinement, aerial spinning continues to be one of the preferred methods for main cable construction.

The desirability of eliminating, as far as practicable, labour-intensive and weather-sensitive site operations for cable construction led to the development in the 1960s, initially in the USA and subsequently in Japan and China, of an alternative to spinning. In this system, the cable is assembled at the bridge site from a large number of factory-manufactured prefabricated parallel wire strands (PPWSs) and has now been used on many major bridges, including the current longest-span 1991 m Akashi Kaikyo bridge in Japan.

Analysis of the cable/deck system

During the nineteenth century, the only available methods for the analysis of suspension bridges were the Rankine and elastic theories, neither of which correctly represented the structural interaction between the cable system and the suspended deck. At the beginning of the twentieth century, the 'deflection' theory – originally published by Melan (1888) – which accurately represented the cable–deck interaction began to be adopted, with its first use being in the design by Moisseiff of the 450 m span Manhattan bridge, completed in 1909. The deflection theory enabled bridges to be designed with increasingly slender deck structures, and by 1931 the 1006 m span George Washington bridge in New York was completed, almost doubling the previous largest main-span length. The culmination of this trend to increasing slenderness was the ill-fated 853 m span Tacoma Narrows Bridge, with a depth-to-span ratio of 1:350, completed in 1940.

The importance of aerodynamics

Although many early suspension bridges had been severely damaged, or in some cases collapsed, owing to the dynamic effects of wind, this problem appeared to have been eliminated in the bridges of the later nineteenth and early twentieth centuries, even though the reasons for these failures was not then understood. However, owing to its extreme slenderness, combined with the unfavourable aerodynamic properties of its plate girder stiffened deck, the Tacoma Narrows Bridge was destroyed soon after completion in only moderate wind conditions by a divergent flutter-type torsional oscillation, alerting engineers to the sensitivity of suspension bridges to dynamic wind effects. Thereafter, the verification of aerodynamic stability has become an essential element of the design process.

Suspended deck design

As a result of the Tacoma Narrows Bridge experience, the suspended decks of subsequently constructed long-span suspension bridges used relatively deep truss stiffening girders, continuing until the construction of the 988 m span Severn Bridge, designed by Freeman Fox and completed in 1966. The design of this bridge, with a suspended deck structure consisting of a shallow box girder combining the traffic deck and the stiffening girder, was a radical departure from previous practice. The clear advantages of the light weight and low drag of a shallow box girder deck has resulted in this becoming the generally preferred choice for the suspended decks of most subsequently suspended structures, although in Japan the need to resist very high typhoon wind speeds resulted in the retention of truss-stiffened decks for many of the Honshu-Shikoku bridges.

Span lengths

The 1991 m span of the Akashi Kaikyo bridge currently remains the longest but will be overtaken by the proposed 2223 m span Dardanelles crossing when this is completed, and a 3300 m main-span bridge for the Messina strait crossing has been proposed and shown to be technically feasible. Kawada (2011) provides a comprehensive description of the history and development of modern suspension bridges.

Structural cables

General

The preferred main cable type for long-span suspension bridges is an assembly of a number of small-diameter high-strength wires laid parallel and constructed by in situ assembly. For the hangers connecting the deck to the main cables, either locked coil or spiral strands are generally used; these can also be suitable for the main cables of moderate spans, where their relatively lower structural efficiency is of less importance.

The basic element for all these cables is high-strength steel wire, which, for parallel wire cables, will usually be between 5 and 5.5 mm in diameter. Wire of this size has generally been produced with a tensile strength of 1550 N/mm^2 , and a 0.2% proof stress of around 1200 N/mm^2 but wire with a minimum tensile strength of 1800 N/mm^2 was first used for the cables of the Akashi Kaikyo bridge and is now widely available.

This wire is produced from steel rods with a much higher carbon content (around 0.8%) than the normally used weldable structural steels. The high tensile strength is obtained by cold drawing through a series of dies to reduce the initial rod diameter to the final wire size, producing wire with consistent accurate dimensions, a smooth flaw-free surface but relatively low ductility, the elongation at fracture being typically about 4%. For corrosion protection, the wire is galvanised, with the weight of zinc coating being typically 300 g/m^2 .

Spiral bridge strands

Spiral strands are manufactured by the helical winding of a number of layers of round steel wires, with each layer of wires laid in an opposite helix to the preceding layer. The helical twist of the wires results in a decrease of the order of 15% to 25% in the stiffness of the strand relative to that of straight wire, and also reduces the strand strength to around 90% of the sum of the individual wire breaking strengths. Figure 15.2 shows a typical arrangement of a spiral strand. For attachment to anchor points, spiral bridge strands are terminated by zinc or epoxy-filled sockets.

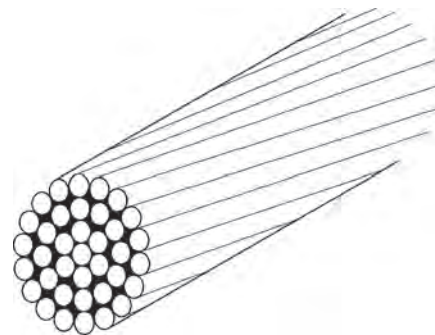


Figure 15.2 Spiral bridge strand

The twisting of the wires causes spiral strands to self-compact under axial loading, resulting in non-elastic stretch when the strand is first loaded. It is therefore essential to remove this as far as practicable during manufacture so that the strand behaves elastically when installed in a cable. This is achieved by cyclic prestretching of the completed strand by applying an axial load between 10% and 50% of the breaking load until a stable modulus value has been reached. However, despite this prestretching, some in-service long-term inelastic extension of spiral strand cables may occur, causing permanent downward deflection of the bridge deck. Typical bridges using spiral strands for their main cables are the 608 m main-span Tancarville Bridge, completed in 1959, and the 600 m main-span Lillebælt bridge in Denmark, completed in 1969.

Locked coil strands

Locked coil strands are also factory-produced by the helical winding of a number of layers of round steel wires onto a straight centre core wire. They differ from spiral bridge strands in that the final layers of wires are made up of interlocking Z-shaped wires, giving a larger proportion of wire area to strand area and, more importantly, a smooth easily protected exterior surface. Their stiffness and other characteristics are generally similar to those of spiral strands.

Typical bridges using locked coil strands using compact closed formation for their main cables are the 335 m main-span Tamar Bridge, completed in 1961, the 378 m Rodenkirchen

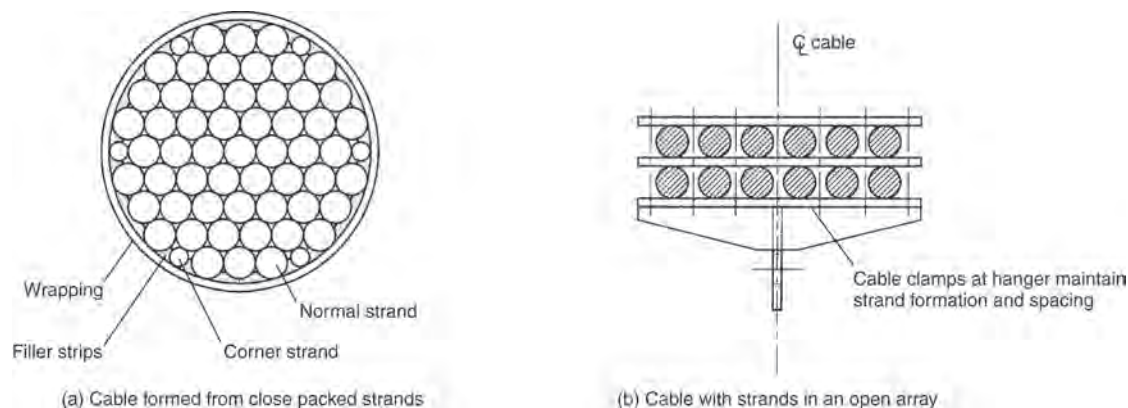


Figure 15.3 Spiral or locked coil strand arrangements

Bridge in Germany, completed in 1954, but reconstructed and widened in 1994, and the 850m main-span Askøy Bridge in Norway, completed in 1993, with an open strand cable arrangement (Figure 15.3(b)).

Locked coil or spiral strands can be arranged in either a close-packed hexagonal formation (Figure 15.3(a)), or in an open rectangular array (Figure 15.3(b)). The first of these has the advantage that, through the addition of aluminium or plastic edge spacers, the cross-section can be circularised and wrapped for corrosion protection. The open strand arrangement does not require the time-consuming wrapping operation, and the cable bands can be simple fabricated structures, but has the disadvantage that the wind load on the cable is increased.

Parallel wire cables

General

Because of their higher structural efficiency, cables of this type, consisting of the required number of individual wires, laid straight and parallel throughout the complete cable length from anchorage to anchorage, are the only realistic choice for very long spans. Parallel wire cables are constructed by either aerial in situ spinning of the wires or the assembly of preformed parallel wire strands. After completion of wire placement, both cable types are compacted into a tightly packed circular mass of parallel wires.

In situ spun cables

Until the development of PPWSs, aerial spinning was the only method used for parallel wire cable construction. In conventional aerial spinning, either two or four loops of wire are pulled by a travelling 'spinning' wheel from one anchorage to the other. After each trip of the wheel, the wires are adjusted to the required sag, this process continuing until all the wires in the cable have been assembled. The wires are anchored at both ends of the bridge by placing each loop of wire around a

semicircular strand shoe (Figure 15.4). A further development of 'controlled tension spinning' avoids the need for individual wire adjustments, greatly simplifying site work.



Figure 15.4 Anchorage termination for spun cable

For convenience in construction, the individual wires are divided into groups, referred to as strands, each containing between 200 and around 550 wires. The number of wires in each strand must be a multiple of $2 \times 2 = 4$, the first factor of two arising from the looping of the wire round the strand anchors, and the second from the need for symmetry about the strand shoe centre line; this will generally result in a few wires more than the theoretical minimum requirement.

Strands produced by spinning consist of two continuous wires looped around the anchorage strand shoes and comprising wire from many coils joined together using swaged compression splices, a typical example being shown in Figure 15.5.

During spinning, the strands are usually arranged in a hexagonal formation with vertical sides, and cables are therefore generally made up from 19, 37, 61 or 91 strands, arranged as shown in Figure 15.6.

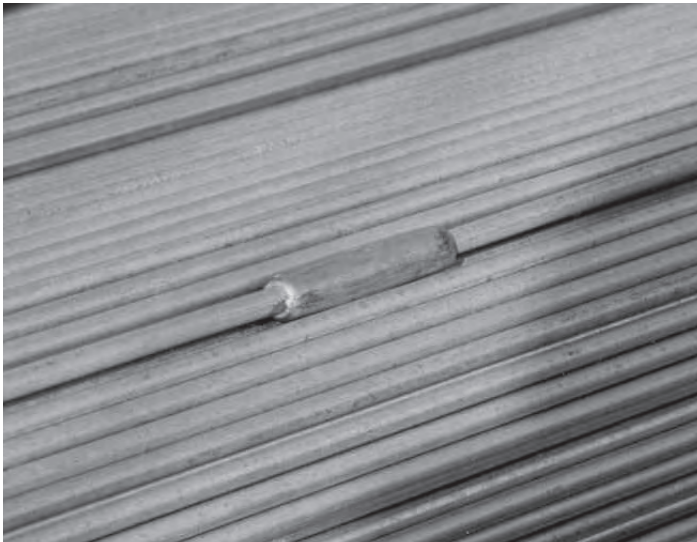


Figure 15.5 Typical wire splice

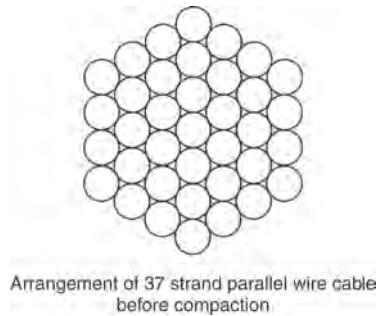
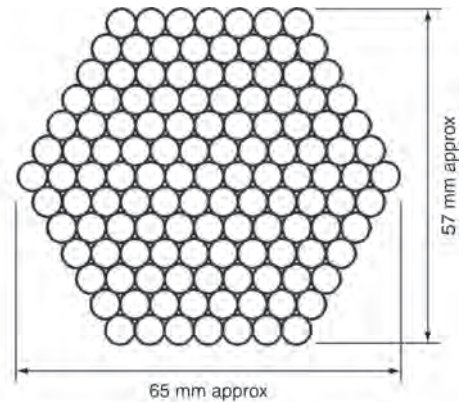


Figure 15.6 Hexagonal strand arrangement

The arrangement of strands in a vertical hexagon has the advantage that, during spinning, temporary spacers can be placed at intervals between the columns of strands. These spacers keep the strands of a part-completed cable under control in storm wind conditions, and enable air to circulate between the strands, minimising strand temperature differences and resultant sag variations.

Prefabricated parallel wire strand (PPWSs)

This alternative to aerial spinning was originally developed in the USA in the 1960s, but the method has been subsequently developed and used in Japan and China. The PPWSs consist of a number of wires factory-assembled into hexagonal strands, bound together at intervals by plastic tape, fitted with sockets at each end and assembled onto large-diameter reels for delivery to the bridge site. To generate the hexagonal shape of the strand, the number of wires in each PPWS must be one of the series of 'centred hexagonal' numbers, that is, 37, 61, 91, 127, and so on. Early PPWSs used 61 or 91 wires, but 127 wires per strand, arranged as in Figure 15.7, is now the generally adopted compromise between excessive reel weight and the erection of an unduly large number of strands.



Arrangement of 127 wire preformed parallel wire strand

Figure 15.7 PPWS make-up

The use of PPWSs requires the transport and handling of large-diameter heavy reels, with the longest PPWSs so far used being those of the Akashi Kaikyo bridge, with an average strand length of 4073 m, with each reeled strand weighing 94 t. Cables constructed using PPWSs have a much larger number of small strands than in an equivalent-size spun cable. This has the disadvantage that it impacts the design of the anchorages, as the size of the anchorage chamber might have to be increased to accommodate the much larger number of strand terminations. The advantage of the PPWS system is that it transfers much of the cable production process off-site, making main cable assembly more rapid and much less weather-sensitive than aerial spinning.

Compaction of parallel wire cables

After completion of spinning or PPWS erection, the individual strands are squeezed together into a compact circular cross-section. The degree of compaction achieved in this operation is specified by the void ratio, defined as the area of voids in the cable divided by the area enclosed by the circumscribing circle. The compacted voids ratio achievable in actual cables varies between 18% and 22%. The increase from the theoretically minimum possible value arises mainly from wire crosses, caused by the impossibility of achieving perfectly parallel wire alignments.

Corrosion protection of parallel wire cables

As the main cables are the primary structural elements of the bridge, it is important to provide them with a level of protection against corrosion that, with reasonable care and maintenance, will enable the design life of the bridge to be achieved. Parallel wire cables are an assembly of many small-diameter wires, and consequently have a large exposed surface area relative to their overall cross-sectional area, increasing their vulnerability to corrosion damage.

From the earliest days of parallel wire cable construction, the preferred method for corrosion protection has been to employ a multi-barrier system, consisting of

- a galvanised zinc coating on the wires
- coating of the exterior surface of the compacted cable by a red lead paste covered by circumferential wrapping with tightly spaced galvanised steel wire wrapping (The use of lead-based products is no longer environmentally acceptable, but wrapping pastes based on metallic zinc powder are an effective alternative.)
- a final series of paint layers applied over the wrapping wire.

Such a wrapping system cannot completely prevent water and air ingress into the cable as the red lead paste can harden and crack and sealing adjacent to the cable clamps is difficult. Furthermore, direct observation of the interior wires is not possible so any deterioration because of corrosion can potentially progress to a serious level before becoming detectable. Nevertheless, it was believed that this system could be relied on to provide an adequate level of corrosion protection.

The first indications that this system cannot be relied on to provide adequate protection came from intrusive examination of the cables of early American suspension bridges (Stahl and Gagnon, 1995). It became clear from these inspections that although this system is partially effective against internal wire corrosion it does not completely prevent it, and inspections of many other more recently constructed bridges in both the UK and elsewhere have confirmed this. An improvement to the conventional wrapping system for cable protection is therefore essential for future bridges and to extend the life of existing structures.

As experience has shown that the corrosion of steel can be reduced to a negligible level when the ambient humidity is below around 40%, an active approach to protection by dehumidifying the interior of the cable is now accepted as the most effective method of enhancing cable protection. The first application of a dehumidification system was on the Akashi Kaikyo bridge in 1998, and similar systems have been installed on many major suspension bridges as the need for enhanced protection and the clear benefits of dehumidification to enhance protection have become apparent. It is likely that all new major bridges will be constructed with cable dehumidification provided as their primary corrosion protection.

Cable dehumidification systems consist of the following components.

- The attachment of a series of manifolds along the cable from which dry air can be injected into the cable, with these alternating with similar manifolds from which exhaust air can escape.
- An impermeable wrapping to seal the cable, containing the dried air and ideally permitting a small overpressure so that even if there are minor imperfections, water or moisture does not enter the cable. The wrapping typically consists of several layers of a polymer-based elastomeric material laid circumferentially with generous overlaps and heat shrunk on to ensure tight contact with the cable wires.

- Dehumidification plant to produce and distribute dried air to the injection points.

Corrosion protection of spiral strand and locked coil cables

During the manufacture of these cables, a flexible corrosion-inhibiting filler is introduced to fill any voids between the wires. After erection, further protection is provided by the application of an external flexible paint or similar coating. An alternative is the manufacture of strands with a long wire lay length so that a tightly fitting high-density polyethylene external sheath can be incorporated.

Static analysis of loaded cables

General

The main cables are the primary load-carrying members of suspension bridges and the overall behaviour of these structures is largely determined by how these cables respond to applied loading. Whether comprising a single small-diameter wire or an assembly of wires laid parallel or wound in a helical formation, all cables have a very low resistance to bending moments, shear forces and compressive forces, and can therefore be treated as being perfectly flexible members capable of resisting only direct tensile forces.

A suspended cable can therefore only support transverse loading by adopting a geometry in which, at all points, the transverse component of the net tensile force in the cable is in equilibrium with any applied load; to achieve this equilibrium, angular changes must occur at every such location. Where the load is continuous and uniform (for example, self-weight loading), these changes result in the familiar vertical curvature of suspended cables. The tensile forces resulting from vertical loading are transmitted along the cable to its end points, which must therefore provide not only vertical support but also resistance to the horizontal component of cable tension. For cables in most situations, this resultant horizontal end reaction will generally be significantly larger than the vertical reaction.

Where only transverse loads are applied to the cable, equilibrium requires that the horizontal end reactions are equal and opposite; consequently, the horizontal component of cable tension must be constant throughout the whole length of the cable.

Cables with uniform loading

General case

The simplest forms of suspended cables occur where the vertical loading is applied uniformly along either the span or the cable length. In Figure 15.8, a cable with a self-weight of w_c per unit length supports a suspended load with a constant distributed weight of w_d per unit span length.

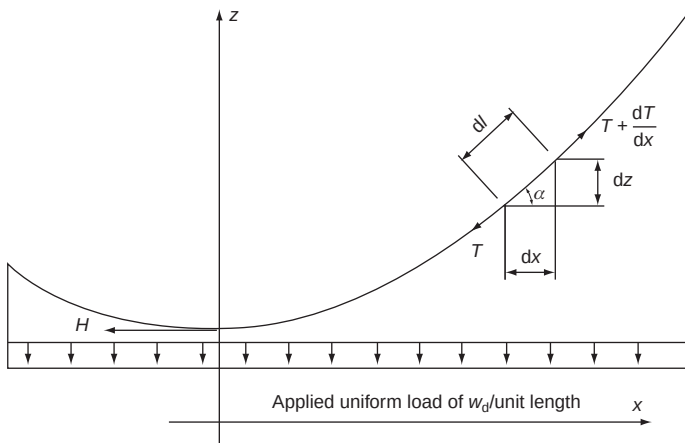


Figure 15.8 Suspended cable

Considering a small element of the cable at a distance x from its lowest point, vertical equilibrium requires that

$$[T \sin \alpha + \frac{dT}{dx}(T \sin \alpha) dx] - T \sin \alpha = w_c dl + w_d dx \quad (15.1)$$

which simplifies to

$$\frac{dT}{dx}(T \sin \alpha) = w_c \frac{dl}{dx} + w_d \quad (15.2)$$

Similarly, horizontal equilibrium of the element requires that

$$[T \cos \alpha + \frac{dT}{dx}(\cos \alpha) dx] - T \cos \alpha = 0 \quad (15.3)$$

which simplifies to

$$\frac{dT}{dx}(\cos \alpha) dx = 0 \quad (15.4)$$

Integration of this equation gives $T \cos \alpha = Q$, where Q is a constant of integration. At the lowest point of the cable, α is zero, so that $\cos \alpha = 1$ and $T = H$ (the constant horizontal component of cable tension) and therefore

$$T \cos \alpha = H \quad (15.5)$$

For a small element of the cable, by similar triangles

$$\frac{T \sin \alpha}{H} = \frac{dz}{dx}, \quad \text{i.e.} \quad T \sin \alpha = H \frac{dz}{dx} \quad (15.6)$$

and, as $(dl)^2 = (dx)^2 + (dz)^2$

$$\frac{dl}{dx} = \sqrt{1 + \left(\frac{dz}{dx}\right)^2} \quad (15.7)$$

The vertical equilibrium equation can therefore be written as

$$\frac{d}{dx}\left(H \frac{dz}{dx}\right) = w_c \sqrt{1 + \left(\frac{dz}{dx}\right)^2} + w_d \quad (15.8)$$

or

$$H \frac{d^2 z}{dx^2} = w_c \sqrt{1 + \left(\frac{dz}{dx}\right)^2} + w_d \quad (15.9)$$

Even though this second-order differential equation relates only to uniform loading, a general solution cannot be obtained. Solutions can, however, be derived for two useful simple loading conditions:

- a cable of weight w_c with no applied loading ($w_d = 0$)
- a weightless ($w_c = 0$) cable carrying a uniform distributed loading w_d .

Catenary cables

The free-hanging shape adopted by an inextensible flexible cable of uniform cross-section is called a catenary, the governing equations of which were derived by Bernoulli in the late seventeenth century. For a cable hanging under its self-weight, only w_d is zero, and Equation 15.9 can be integrated by writing $dz/dx = \psi$ and also $H/w_c = C$, so that

$$C \frac{d\psi}{dx} = \sqrt{1 + \psi^2} \quad \text{or} \quad C \frac{d\psi}{\sqrt{1 + \psi^2}} = dx \quad (15.10)$$

This integrates to give $C \sinh^{-1} \psi = x + A_1$, where A_1 is a constant of integration, and after substituting for ψ

$$\frac{dz}{dx} = \sinh\left(\frac{x}{C} + A_1\right) \quad (15.11)$$

At its lowest point, the cable is horizontal (i.e. $dz/dx = 0$) and $x = 0$, so that the constant of integration is zero.

Integrating a second time

$$z = C \cosh \frac{x}{C} + A_2 \quad (15.12)$$

where A_2 is a constant. By choosing the z -coordinate origin to be at a distance C below the lowest point of the cable, so that at $x = 0$, $z = C$, this second constant of integration becomes zero and the Cartesian equation of the cable becomes

$$z = C \cosh \frac{x}{C} \quad (15.13)$$

where x and z are the horizontal and vertical coordinates and C is the catenary constant of the cable. The length l of any segment of the cable measured from its lowest point can be derived as any point at that distance, starting from the equilibrium conditions

$$T \cos \alpha = H \quad (15.14)$$

$$T \sin \alpha = l w_c \quad (15.15)$$

Eliminating T gives

$$\tan \alpha = \frac{l w_c}{H} = \frac{dz}{dx} \quad (15.16)$$

and as

$$\frac{H}{w_c} = C \quad \text{and} \quad \frac{dz}{dx} = \sinh \frac{x}{C} \quad (15.17)$$

the cable length measured from its lowest point is given by

$$l = C \sinh \frac{x}{C} \quad (15.18)$$

The tension in the cable can be determined by squaring and adding Equations 15.14 and 15.15, giving

$$T^2 = (w_c C)^2 \cosh^2 \left(\frac{x}{C}\right) \quad \text{or} \quad T = w_c z \quad (15.19)$$

The cable tension at any point is therefore given by the product of the cable weight per unit length and the distance of that point above the origin of coordinates (chosen to be at distance C below the lowest point), so that the maximum tension in the cable occurs at the higher end support. For a cable hanging between two points at the same level, solutions to these

equations can be obtained directly if either the overall length or the central sag of the cable is known.

In the more general case, where the end points are at differing levels, a solution can be derived as follows. If the length of the cable is L , the span length is K , and the difference in height between end points is B , then from Equation 15.13

$$B = C \left[\cosh \frac{x+K}{C} - \cosh \frac{x}{C} \right] \quad (15.20)$$

and from Equation 15.18

$$L = C \left[\sinh \frac{x+L}{C} - \sinh \frac{x}{C} \right] \quad (15.21)$$

Squaring and adding these equations gives

$$L^2 - B^2 = 4C^2 \sinh^2 \frac{K}{2C} \quad (15.22)$$

For a cable of given length, Equation 15.22 relates the catenary parameter C to the known dimensions L , B and K . An explicit solution of the equation is not possible, and an iterative numerical solution is required to derive the value of C .

As main cables are constructed as a free-hanging group of wires, they have an initial 'free cable' catenary profile, the geometry of which is determined so that, after the addition of the deck and other dead and permanent loads, the specified final dead load cable geometry is achieved.

Parabolic cables

An inextensible weightless cable subjected to a load distribution that is uniform with span length takes up a parabolic shape with, in this case, $w_c = 0$ and Equation 15.9 becomes

$$H \frac{d^2 z}{dx^2} = w_d \quad (15.23)$$

which on integration twice gives

$$\frac{dz}{dx} = \frac{w_d}{H}x + A \quad \text{and} \quad z = \frac{w_d}{2H}x^2 + Ax + B \quad (15.24)$$

where A and B are constants of integration. By choosing the origin of coordinates to be coincident with the lowest point of the cable, where both $z=0$ and $dz/dx=0$, both of these constants become zero, and

$$z = \frac{w_d}{2H}x^2 \quad (15.25)$$

For a parabolic cable with span K and ends at the same level, the difference in elevation between the midpoint and the ends is the sag f of the cable and from Equation 15.25 the relationship between the sag and H is obtained as

$$H = \frac{w_d K^2}{8f} \quad (15.26)$$

The length L of the cable for small (f/K) ratios can be derived as

$$L = K \left[1 + \frac{8}{3} \left(\frac{f}{K} \right)^2 - \frac{32}{5} \left(\frac{f}{K} \right)^4 \right] \quad (15.27)$$

and the extension of the cable is given to an acceptable accuracy level by

$$\Delta L = \frac{HL^2}{KAE} \quad (15.28)$$

where A and E are, respectively, the cable area and elastic modulus.

A uniform distribution of dead load across the span is a good approximation to the actual distribution for most suspension bridge decks; since for most long-span bridges the deck weight per unit span length is significantly greater than that of the cables, the resultant cable profile will therefore be close to a parabola. It can be shown (Pugsley, 1968) that, for the span/sag proportions typical of suspension bridge cables, there is little difference between catenary and parabolic cable shapes and, because of the simplicity of the governing equations, the latter shape is generally assumed for suspension bridge analysis.

General case of vertical loading on cables

The assumption of zero bending stiffness implies that the bending moment at any point must also be zero; this condition can be used to derive a general equation for the shape of a cable loaded by any combination of concentrated and distributed vertical loads, whether or not its end supports are at the same level. In Figure 15.9, a suspended cable is subjected to an arbitrary distribution of both concentrated and distributed applied vertical loads. As there are no horizontal loads, H has a constant value and is accompanied by vertical end reactions of $\pm HB/K$. After defining $\sum M_{A-P}$ as the moment about P due to the applied vertical forces to the left of that point, and R_a as the vertical reaction at end A due to these loads, application of the zero bending moment condition gives

$$M_P = 0 = \sum M_{A-P} + H z_1 - x_P \left(R_a - \frac{HB}{K} \right) \quad (15.29)$$

that is

$$0 = [\sum M_{A-P} - R_a x_P] + H z_1 + \frac{HB}{K} x_P \quad (15.30)$$

As $x_P = z_u(K/B)$ and $z_P = z_u + z_1$ (the total distance from the chord line to the cable at P), the equation can finally be written as

$$H z_P = R_a - \sum M_{A-P} \quad (15.31)$$

or

$$z_P = \frac{M_P}{H} \quad (15.32)$$

where M_P is the bending moment that would be caused by the applied vertical loading applied to a simply supported beam of the same span. The vertical ordinate of any point on the cable below the chord line joining its end support points is therefore equal to the moment that would exist at that point in a simply supported beam of the same span as the cable, divided by the horizontal component of the cable tension H . Where only concentrated loads are present in a weightless cable, the resultant shape will be a polygon consisting of a series of straight sections with a sharp angular change at each loaded point. Alternatively, where only distributed loading is applied, the cable profile will become a continuous curve.

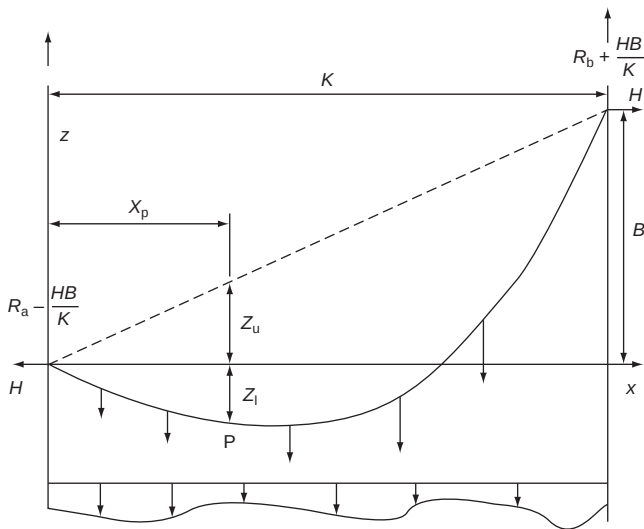


Figure 15.9 Cable diagram

From Equation 15.32, if the initial vertical (e.g. self-weight) loading would produce a moment of M_i at a point on the cable, then this would be equal to H_z , where z is the ordinate at that point relative to the chord line linking its end supports. The application of an additional vertical loading causing a moment M_A at that point will result in an increase ΔH in the horizontal component of cable tension, together with a change in the cable geometry, causing a deflection v from the initial cable position. The total moment at any point due to the initial plus additional loading will be ($M_i = M_A$); therefore, for equilibrium

$$(H + \Delta H)(z + v) = M_i + M_A \quad (15.33)$$

$$\text{As } M_i = H_z$$

$$z + v = \frac{H_z + M_A}{H + \Delta H} \quad (15.34)$$

and the deflection v is given by

$$v = \frac{M_A}{H + \Delta H} - \frac{\Delta H}{H + \Delta H}(z) \quad (15.35)$$

The usefulness of Equation 15.35 to derive the deflected shape of a suspended cable depends on knowledge of $H + \Delta H$; the value of this is determined by the condition that the deflections of the additionally loaded cable must be compatible with the initial length of the cable together with any displacements of its end supports, and for an elastic cable, any elongation produced by the increment ΔH in the horizontal tension component. For a cable of span K between rigid end supports, the equation relating deflections v to the change in horizontal tension component ΔH can be derived as

$$\int_0^K v \frac{d^2 z}{dx^2} dx + \frac{\Delta H}{AE} L_c = 0 \quad (15.36)$$

where L_c is the cable length, A is the cable area and E its elastic modulus. For a parabolic cable

$$\frac{d^2 z}{dx^2} = \frac{w}{H} \quad (15.37)$$

so that the condition becomes

$$\int_0^K v \frac{w}{H} dx \quad (15.38)$$

and as w and H are constants

$$\int_0^K v dx = 0 \quad (15.39)$$

Cable with a single concentrated load

The effect of a concentrated load is shown in Figure 15.10. The cable will initially hang in the funicular curve determined by its dead load; the effect of the concentrated load is to deflect the cable, as shown in the figure. Because H must be constant, the tension is increased everywhere; to preserve vertical equilibrium and length compatibility, the profile flattens, with the cable tending to be deflected upwards in areas remote from the load. Additional deflection will occur owing to cable extension but, for a concentrated or short distributed load, this will generally be small relative to that due to the geometrical shape adjustment.

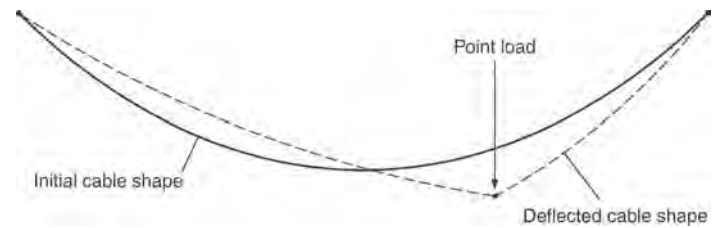


Figure 15.10 Shape of a deflected cable

The resistance of a cable to displacement by additional loading is therefore largely determined by its self-weight and referred to as the gravity stiffness.

The response of a suspended cable to applied loading is most easily investigated by initially examining the effect of a single concentrated load on an inextensible cable. In Figure 15.11, an inextensible cable carrying an initial uniform load of w per unit span is suspended between level supports with a span K and is subjected to an additional concentrated load P that is small relative to the total distributed load wK , and positioned at a distance αK from the centre point of the span.

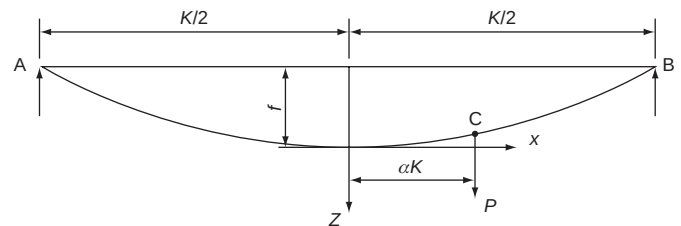


Figure 15.11 Cable with a single concentrated load

The cable shape with the initial uniform loading only will be a parabola and, taking the centre point of the cable as the origin, is given by

$$y = \frac{1}{2} \left(\frac{wx^2}{2H} \right) \quad (15.40)$$

and in addition

$$H = \frac{wK^2}{8f} \quad (15.41)$$

When an additional load P is applied, the horizontal component of the cable tension increases to $H + \Delta H$, and the cable shape now consists of two discontinuous parabolic arcs intersecting at the loaded position, with the vertical components of the cable tension on either side of the loaded point in equilibrium with the applied load P . Ignoring any small longitudinal movement of the load P , the bending moment due to this load on the cable between support A and the loaded point C (i.e. for $0 < x < (0.5 + \alpha K)$) will be

$$M_p = P(0.5 - \alpha)\left(\frac{K}{2} + x\right) \quad (15.42)$$

and between C and the other support B (i.e. for $(0.5 + \alpha K) < x < K$)

$$M_p = P(0.5 + \alpha)\left(\frac{K}{2} - x\right) \quad (15.43)$$

The increase in the horizontal component of cable tension ($H + \Delta H$) is determined from consideration of the geometric compatibility of the cable. Inserting in Equation 15.39 the value of v from Equation 15.35 gives

$$\int_{-K/2}^{K/2} \frac{M_p}{H + \Delta H} dx - \int_{-K/2}^{K/2} \frac{\Delta H}{H + \Delta H} z dx = 0 \quad (15.44)$$

that is

$$\Delta H = \frac{\int_{-K/2}^{K/2} M_p dx}{\int_{-K/2}^{K/2} z dx} \quad (15.45)$$

From Equations 15.42 and 15.43, the span-wise distribution of M_p is triangular and has a maximum value of $(PK/4)(1 - 4\alpha^2)$ at the load position, so that the integral

$$\int_{-K/2}^{K/2} M_p dx \text{ is equal to } \frac{PK^2}{8}(1 - 4\alpha^2) \quad (15.46)$$

At the centre line, the original parabolic cable shape has a maximum sag f and the integral

$$\int_{-K/2}^{K/2} z dx \text{ is equal to } \frac{2}{3}fK \quad (15.47)$$

Inserting these values into Equation 15.45 gives the value of ΔH as

$$\Delta H = \frac{3}{16}\left(\frac{PK}{f}\right)(1 - 4\alpha^2) \quad (15.48)$$

and since

$$f = \frac{wK^2}{8f} \quad (15.49)$$

this becomes

$$\Delta H = \frac{3}{2}\left(\frac{P}{wK}\right)H(1 - 4\alpha^2) \quad (15.50)$$

and the final cable horizontal tension H_f is

$$H_f = H + \Delta H = H\left[1 + \frac{3}{2}\left(\frac{P}{wK}\right)(1 - 4\alpha^2)\right] \quad (15.51)$$

The deflection v at any point relative to the original shape can then be determined using Equation 15.35, and that of most interest is at the point of application of the additional load P . At this point

$$M_p = \frac{PK}{4}(1 - 4\alpha^2) \quad (15.52)$$

and (for the initially unloaded cable) $z = f(1 - 4\alpha^2)$.

Using Equation 15.35 then gives

$$v_p = \left(\frac{1}{H + \Delta H}\right)\left[\frac{PK}{4}(1 - 4\alpha^2) - \frac{3}{2}H\left(\frac{P}{wK}\right)f(1 - 4\alpha^2)^2\right] \quad (15.53)$$

After substituting $H = wK^2/8f$, this gives the cable vertical deflection as

$$v_p = \frac{f}{K}\left(\frac{P}{w}\right)\frac{(1 - 4\alpha^2)(1 + 12\alpha^2)}{2\left[1 + \frac{3}{2}\left(\frac{P}{wK}\right)(1 - 4\alpha^2)\right]} \quad (15.54)$$

The relationship of deflection to the position of the concentrated load P and its ratio to the initial distributed wK load is given in Figure 15.12, which shows, for a range of increasing load ratios P/wK , the variation in v/f at the load position as the load moves across the span. Although, from Equation 15.51, the maximum increase in cable tension occurs with the applied load positioned centrally (i.e. with $\alpha = 0$), Figure 15.12 shows that this is not the case for deflection, with maximum deflection occurring when the concentrated load is only around 20% of the span length.

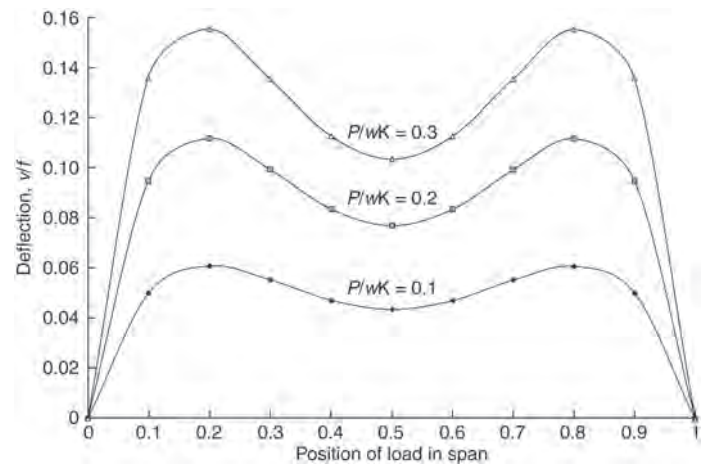


Figure 15.12 Deflection variation for a single load

Equation 15.54 and Figure 15.12 show the primary factors governing the gravity stiffness and response of a single suspended cable to applied loading.

- The deflection under the applied load is proportional to the sag/span ratio (f/K). A reduced sag/span ratio (with a consequent increase in H) reduces the deflection under the load. Cable stiffness increases with decrease in sag/span ratio, a result in accordance with what would be expected intuitively.
- As the ratio of the deflection under the applied load is proportional to the ratio of applied load to cable self-weight (P/w), an increase in cable weight reduces this ratio and produces a corresponding reduction in deflection under the load. A relatively heavier cable is more difficult to displace and has a higher vertical stiffness than a cable with similar geometry but a lower self-weight loading.
- As a result of the presence of the P/wK term in the denominator of the equation, the relationship between the applied load and the resultant deflection is non-linear, with the effective cable stiffness progressively increasing with an increase in load.

- As a result of this non-linearity, deflections resulting from a combination of additional loads cannot be obtained by superposition of the results obtained from separate analyses of each loading condition.
- The cable vertical stiffness varies according to the location of the load, with the cable having lower stiffness for loads applied non-centrally.

Cables with a number of applied loads

The previous method can be extended to derive solutions for cases involving both a number of concentrated loads and part-span distributed loading, but the complexity of the resulting expressions rapidly increases, and the general case of an arbitrary distribution of distributed and concentrated loads is not amenable to direct analytical solution.

Approximate numerical solution procedures (Jennings, 1962; Michaelos and Birnstiel, 1960; O'Brien, 1967; O'Brien and Francis, 1964) are therefore more realistic methods in which the cable is divided into a series of short straight segments modelling the real cable geometry to the necessary degree of accuracy. All loads (including the cable self-weight) are distributed to the segment nodal points.

By making the cable statically determinate with an assumed value of H , the resultant profile, including the effects of segment extension, can be calculated by working progressively from one end support. In general, this assumed value of H will not produce a cable profile that terminates at the other support position, but an improved value of H can be calculated from the magnitude and direction of the lack of fit, and by a process of iteration the deflected cable profile can be determined to any desired degree of accuracy.

The effect of cable elasticity and non-uniform loading

Actual cables are composed of elastic material; consequently, when transverse loading is applied, deflection occurs as the result of a combination of geometrical change (i.e. gravity stiffness) and elastic extension, with the relative importance of these depending largely on the distribution of the applied loading.

When additional load is applied to only part of the span, geometrical change is the predominant effect on deflections, with the extent of this being dictated by the extent of loading non-uniformity. Figure 15.13 illustrates this by showing the relative contributions of gravity stiffness and extension to the deflection for a steel cable with a sag/span ratio of 1:10, a cross-sectional area of 0.5 m² and a span of 1000 m. The cable is assumed to have an initial UDL (self-weight plus suspended structure) of 200 kN/m, and a symmetrical centrally applied

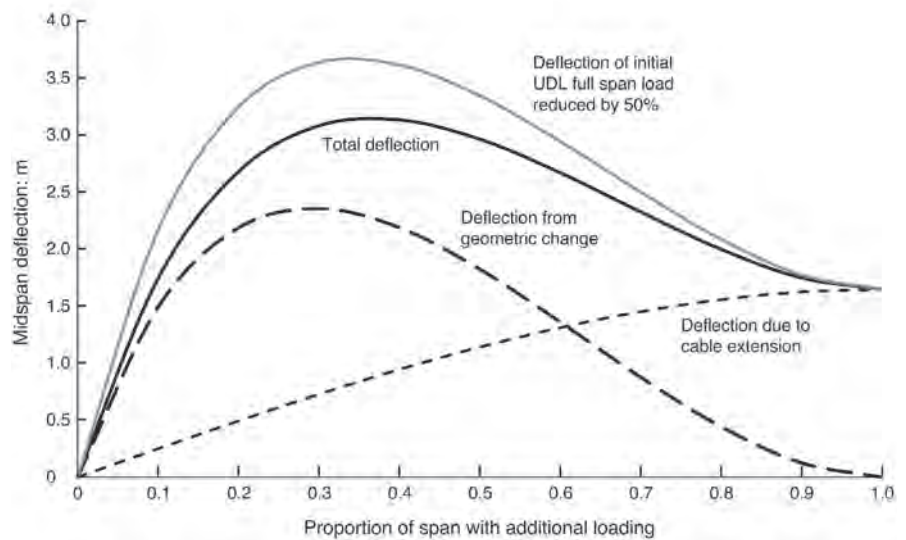


Figure 15.13 Cable elasticity

uniform (e.g. live) load of 80 kN/m is applied over increasing proportions of the span.

The maximum deflection occurs with only about 40% of the span loaded, is about 90% greater than if the additional load were applied over the whole span length and is largely due to cable displacement. As the loaded length increases, the effect of cable displacement steadily decreases, with elongation becoming more significant until, with the additional load acting over the whole span, the deflection is entirely due to cable elongation caused by the increase in cable tension.

The figure also illustrates the influence of dead load on cable stiffness by including the increase in deflection that results from a reduction of 50% in the initial UDL.

Flexibility of cable end supports

The cable end supports have thus far been assumed to be rigid, with no change in span occurring as a result of changes in loading. The effect of cable end support flexibility can be quantified using the equation for the effect of a change in span (dK) on the cable sag (df) of a parabolic cable. This can be obtained by partial differentiation of Equation 15.27 to give, for a cable with end supports at the same level

$$df = -\frac{15 - 40\left(\frac{f}{K}\right)^2 + 288\left(\frac{f}{K}\right)^4}{16\left(\frac{f}{K}\right)[5 - 24\left(\frac{f}{K}\right)]} \times dK \quad (15.55)$$

For a typical main-span cable with a value of $f/K=0.10$, Equation 15.55 gives

$$df = -1.84(dK) \quad (15.56)$$

The extent of tower saddle movement caused by live loading therefore has a significant effect on the overall vertical stiffness of the main span, and as the towers provide relatively little bending resistance to changes in horizontal cable forces, the longitudinal stiffness of the side span cables is a significant factor in the control of bridge main-span deflections.

The stiffened suspension bridge

Bridge configuration

The need to control longitudinal displacements of the supporting towers of a suspended cable span leads naturally to the conventional three-span layout of Figure 15.1, with a main span flanked by shorter side spans, in which the cables are deflected downwards to ground-based anchorages. Depending on the site topography, the side spans can be supported by the main cables or be independent ground-supported viaduct structures, in which case the side span cables act simply as backstays to the towers.

Main-span length

The most frequent requirement for a long-span suspension bridge is the crossing of a wide river or estuary; as the main cable load (and hence its size and the quantity of cable material) is proportional to the square of the main-span length, the shortest practicable main-span length is desirable, subject to the requirement for sufficient clear width to ensure safe navigation of shipping using the waterway.

The foundations of the main towers may be sufficiently massive to resist accidental ship impacts, but if this is not the case then the provision of protective works, such as ship protection islands, will be required. A longer main span may then be the most economical overall structure if this positions the towers in sufficiently shallow water to prevent the approach of large vessels; this in turn reduces the size or removes the need altogether for any ship protection. An increase in span length may also be appropriate to enable more rapid and economical construction of the tower foundations by locating the towers on land or in shallow water.

Cable dead load geometry

Suspension bridges are normally constructed so that, under permanent loading at design temperature, the main cables carry the entire bridge self-weight, with the suspended deck structure carrying no global bending moments. In most bridges, the dead load per unit span of the suspended structure is almost constant; therefore, a parabolic curve is a sufficiently close approximation to the cable geometry. For the three-span bridge of Figure 15.14, ignoring the relatively small contribution of the hangers, the horizontal component of cable tension in the main span will be

$$H = \frac{(w_c + w_d) K_M^2}{8 f_M} \quad (15.57)$$

where w_c is the cable weight per metre of span, w_d is the suspended structure weight per metre of span, K_M is the length of the main span and f_M is the midspan sag in the main span.

As a result of the uniformity of H throughout the bridge, and if the cable cross-section and the weight/metre of the

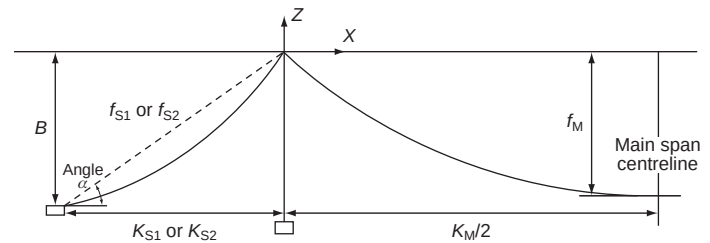


Figure 15.14 Three-span bridge dead load cable geometry

suspended structure are constant throughout the bridge, the corresponding relationships in the general case of unequal side span lengths are

$$H = \frac{(w_c + w_d) K_{S1}^2}{8 f_{S1}} \quad \text{and} \quad H = \frac{(w_c + w_d) K_{S2}^2}{8 f_{S2}} \quad (15.58)$$

where K_{S1} and K_{S2} are the side span lengths and f_{S1} and f_{S2} are the corresponding midspan sags. The cable sags in the side spans are therefore related to that of the main span, as

$$f_{S1} = \left(\frac{K_{S1}}{K_M}\right)^2 f_M \quad \text{and} \quad f_{S2} = \left(\frac{K_{S2}}{K_M}\right)^2 f_M \quad (15.59)$$

For an unsuspended side span, the sag is reduced to

$$f_s = \left(\frac{w_c}{w_c + w_d}\right) \times \left(\frac{K_s}{K_M}\right)^2 K_M \quad (15.60)$$

Side span length

Suspension bridges with side spans of equal length have a visually attractive overall appearance, owing to the symmetry of the cable system about the towers. However, unequal side span lengths required to enable optimum positioning of the cable anchorages can also be visually acceptable, provided the difference in side span lengths is not too great.

An upper limit on side span length of half that of the main span is determined by the need for the main cables to remain above deck level as the anchorage splay saddle is approached, so that tensile hangers can be used throughout the span. At this upper limit, the main and side span cable angles and hence tensions at the tower will be equal, but for shorter side span lengths, the cable angle will be steepest on the side span side of the tower; the maximum cable tension will occur there and determine the main cable size.

The length of the side span has a significant effect in obtaining favourable deformational behaviour of the bridge main span under live loading. An increase in main-span cable H from main-span live loading causes the tower tops to move into that span, with this movement being resisted by a reduction in the side span sag and an increase in side span H until equilibrium is reached.

The reduced sag of a short side span cable increases its resistance to tower top movement and therefore produces an improvement in the vertical stiffness of the main span. A maximum side span of no more than 30% to 40% of main-span length is therefore usually preferable.

Where the side span cables act simply as backstays, this produces a further improvement in stiffness, so that, where ground conditions are suitable, an approach viaduct rather than a suspended side span may provide the most economical bridge configuration.

Main-span cable sag

For a bridge with a defined main-span geometry, applied loading and cable physical properties (strength and density), the cable size and material quantity are determined by the sag/span ratio of the main-span cable. Figure 15.15 shows, for a three-span bridge with suspended side spans with a typical length ratio of K_S/K_M of 0.35, the variation in cable material quantity with change in the main-span (span/sag) ratio. The required cable material increases steadily with an increase in the cable span/sag ratio so that, to minimise cable quantity, a relatively large main-span sag is beneficial.

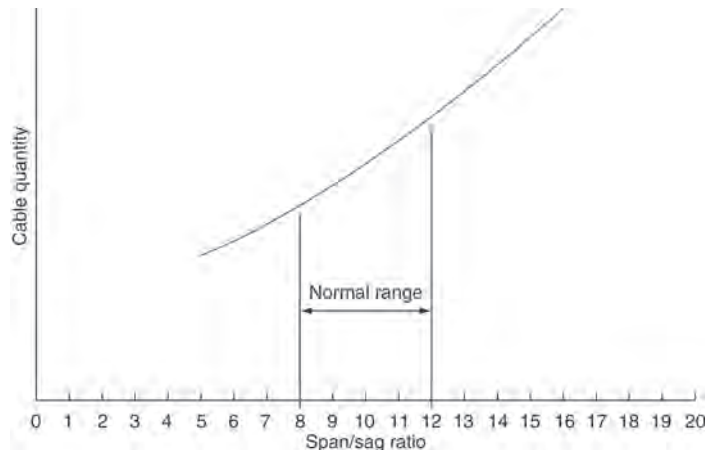


Figure 15.15 Cable quantity v sag ratio

Although beneficial in reducing the quantity and cost of the cable material required, an increase in main-span sag (i.e. a low span/sag ratio) has the following effects on other major components of the bridge:

- an adverse effect on the cost of the main towers from their additional height and material quantity, along with more difficult construction
- a beneficial effect on the anchorage, as the associated reduction in cable H decreases the required size and cost of anchorages
- an adverse effect on the vertical stiffness of the cable system: the reduction in stiffness may result in unacceptable deflections and a reduced margin against aerodynamic instability, together with increases in the longitudinal and angular changes at the movement joints and bearings.

The span/sag ratio producing the most economical and satisfactory design overall must, therefore, be a compromise

between these (and other) conflicting requirements and can only be determined by a comparative parametric study, varying this ratio to determine its impact on all aspects of the proposed design. The experience of previous successful designs is a useful guide and Figure 15.16 shows this for representative bridges. Although there is considerable scatter, with no clear trend for variation in span/sag ratio with span length, all of the bridges considered have ratios within the range 8:1 to 12:1.

It can be concluded that, unless the proposed design has a very unusual configuration, the span/sag ratio most likely to produce an acceptable design will lie within this range, and a value within this range will be the best starting point for a parametric study to determine the optimum value to suit the particular conditions of the proposed design.

Analysis of interaction between main cables and deck structure

In long-span suspension bridges, the vertical flexural stiffness of the suspended deck is generally relatively small, compared with the gravitational stiffness of the main cables. The deck flexural stiffness does not, therefore, contribute significantly to the global structural behaviour of the bridge and an initial estimate of the main cable tension and maximum main-span deflections can therefore be calculated to a reasonable level of accuracy by assuming the deck to be completely flexible. This is not, however, the case for side span deflections, as the relatively higher flexural stiffness owing to the shorter span length is more influential for these.

Such an approximate analysis cannot, however, determine deck girder bending moments, shear forces and support reactions, as these are determined by the interaction between the cable deformations and deck bending stiffness and an accurate analysis must take this into account. Early attempts at analysis of the complete suspension bridge system by Rankine and the elastic theory of Napier did not properly account for this interaction but a fully comprehensive 'deflection theory' was developed by Melan (1888), which combined the differential equations governing the cable and stiffening girder into the following single equation:

$$(H + \Delta H) \frac{d^2}{dx^2} (z + v) = EI \frac{d^4 v}{dx^4} - p + H \frac{d^2 z}{dx^2} \quad (15.61)$$

where p is the live loading.

This is the basic non-linear differential equation governing the behaviour of a stiffened suspension bridge as a function of two unknowns, the increase in cable horizontal tension (ΔH) and the deflection of the cable (v). To complete the solution, a further equation relating the increase in cable tension and deflections is required; this is derived from the expression governing compatibility between the change in cable deflections and extended length due to the applied loading

$$0 = \frac{w}{H} \int_0^K v \cdot dx + \frac{\Delta H}{AE} L_c \quad (15.62)$$

where L_c is the length of the cable.

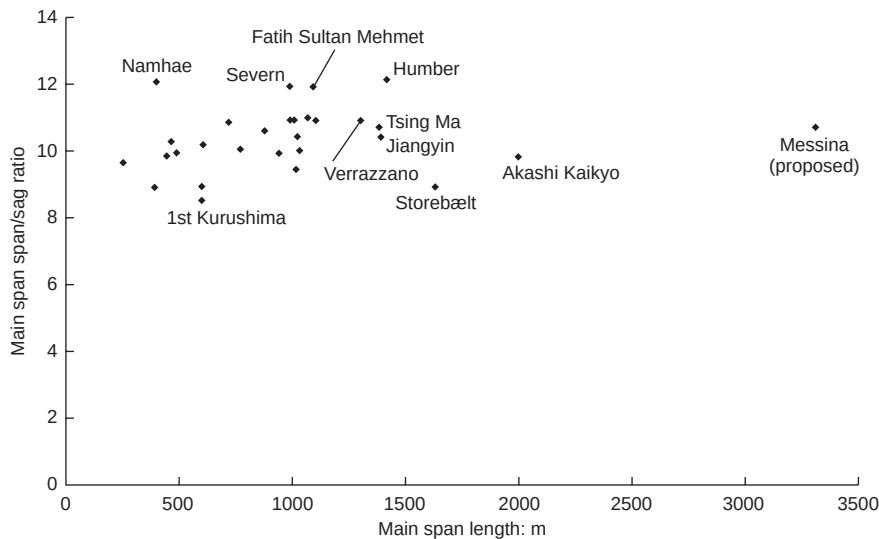


Figure 15.16 Main-span span/sag ratios for previous bridges

These equations can be solved simultaneously for the three spans, although requiring lengthy and laborious calculations. However, the universal availability of finite-element software for the three-dimensional large-displacement analysis of geometrically non-linear structures has now rendered this method obsolete.

For parametric studies to determine the most economical overall bridge configuration, initially only a two-dimensional global model of the structure is required to determine the structure geometry for permanent loads, and to determine the deck girder global bending moments, shear forces and deflections from traffic loads and temperature variation. After initial sizing of members, the model can be expanded into a full three-dimensional representation of the structure to analyse wind loading and differential temperature effects, torsional moments in the deck, hanger and bearing loads from asymmetrical traffic loading, and the structure natural frequencies that are required for calculating stability under wind effects.

Critical loaded lengths for traffic loading must be derived by an iterative procedure, as conventional influence lines will usually produce an underestimate of the required loaded length, owing to the non-linear response of the structure. The design of critical areas of the structure, such as the tower and splay saddles, cable clamps, hanger to deck connections and deck bearing local structure will require additional local fine mesh finite-element models of these areas or elements.

Lateral wind loading

Lateral loads on the suspended deck are resisted by both deck flexure and transverse deflection of the main cables. The effect on a suspended cable of a uniformly distributed horizontal wind load combined with a similarly distributed vertical (gravity) load, is to produce a rotation of the cable plane so that the resultant force lies within the plane of the cable, with its

angle to the vertical being given by

$$\phi = \tan^{-1}\left(\frac{q}{w}\right) \quad (15.63)$$

where q represents the lateral wind load and w the deck and cable dead load. The lateral deflection at the centre of a main-span cable is therefore approximately

$$\delta_c = f\left(\frac{q}{w}\right) \quad (15.64)$$

If the suspended deck were free to move laterally relative to the cable, its lateral deflection δ_D at the centre of the main span, owing to a uniform wind load of q would be

$$\delta_D = \frac{5}{384} \left(\frac{q K_T^4}{EI_T} \right) \quad (15.65)$$

where EI_T is the transverse bending stiffness of the suspended deck.

However, as the ratio (q/w) will generally be very low, it is clear that, in the central area of the span, where the short hangers constrain the deck and cable to deflect together, the cable provides a more effective mechanism for the transfer of lateral wind load to the towers. The effectiveness of the hangers in transmitting deck wind load to the cable is proportional to their length and reduces progressively as the towers are approached. Nevertheless, the resistance of the cables to deck lateral deflection in the central part of the main span is the predominant effect for long spans and causes the majority of the wind load to be transmitted to the tower tops so that deck lateral bending moments are significantly reduced. Because the restraining effect of the cable is greatest in the midspan area, the maximum lateral moment in the suspended deck is not at midspan but occurs nearer to the ends of the span.

Aerodynamics

Although many early nineteenth century suspension bridges were damaged or destroyed by instability under the action of storm winds, the American bridges of the late nineteenth and early twentieth centuries did not experience such problems, owing to the use of relatively heavy and deep stiffening trusses. However, the use of the deflection theory enabled the design of bridges with progressively more slender deck structures, culminating in the construction of the 853 m span Tacoma Narrows Bridge, completed in 1940, which collapsed shortly after construction under relatively low wind speeds, owing to aerodynamic instability. The prevention of aerodynamic instability behaviour has since then been an essential part of the design process.

Suspended deck structure

The relatively high flexibility of suspension bridge structures makes them particularly susceptible to the effects of structure–wind interaction, including

- divergent aerodynamic instability, such as static divergence or flutter, which must be prevented to avoid destruction of the bridge
- buffeting caused by the random variations (turbulence) in the natural wind that might cause unacceptable vibratory motions and possibly cause fatigue damage
- periodic vortex shedding from the flow separation that occurs from shapes and which, although self-limiting, can produce vibrations that may also be unacceptable to bridge users or cause fatigue damage.

Bridge decks with open truss stiffening girders do not generate and periodically shed large vortices, and hence the possibility of producing resonant structure oscillations is reduced, and do not generate significant lift and pitching moment forces that might give rise to static divergence or flutter-type instability. Such forces can be produced by the deck plate structure, but the use of deep stiffening trusses with top and bottom lateral bracings generally provides sufficient torsional stiffness to produce an adequate margin against these instabilities. However, when bridges were constructed with much shallower plate girders deck structures, using plated stiffening girders with low torsional stiffness and susceptibility to strong vortex shedding, the aerodynamic problems reappeared.

As open truss structures produce relatively high drag forces that, for long-span structures, have a significant adverse effect on the design of the suspended deck, cables and towers, shallow box deck girders are now the most frequently chosen suspended deck structure. These substantially reduce the lateral wind load but increase the risk of limited amplitude vibrations from vortex shedding from the entire cross-section. However, the most important aerodynamic effect that must be prevented is the onset of divergent flutter-type instability.

This arises from a complex interaction between the aerodynamic forces produced by a coupling of the vertical and torsional movements of the structure, producing strong sustaining forces, even though the cross-section might have positive aerodynamic damping for individual vertical and torsional movements. It can be inferred that, as coupling between vertical and torsional motions is a prerequisite for such instability, the extent of any separation of the torsional and vertical frequencies will have a significant effect on the onset of instability. With a low depth-to-width ratio, streamlined box girder deck structures can achieve critical flutter speeds not significantly different from that of a flat plate aerofoil, for which an approximate solution, originally given by Selberg (1961) for the critical flutter speed, is

$$V_f = 3.7(bf_T) \sqrt{\frac{m}{\rho b^2}} \left(\frac{f_b}{f_T} \right) \left[1 - \left(\frac{f_b}{f_T} \right)^2 \right] \quad (15.66)$$

where V_f is the critical speed for the onset of flutter, b is the width of the plate, m is the mass per unit length, r is the polar moment of inertia per unit length, f_b is the vertical natural frequency, f_T is the torsional natural frequency and ρ is the density of air. The vertical stiffness, and hence natural frequency, of the bridge is largely governed by the cable size and geometry, so the principal way in which the flutter speed can be influenced

is by an increase in the torsional stiffness of the deck, which is most effectively achieved for box girders by enlarging the cross-sectional area.

A rigid tie (Figure 15.17) between the cables and the deck structure at the midpoint of the main span is an effective way of improving aerodynamic stability, as it increases the torsional stiffness of the cable system by preventing relative longitudinal movements of the cables associated with the lowest antisymmetrical vibration mode.

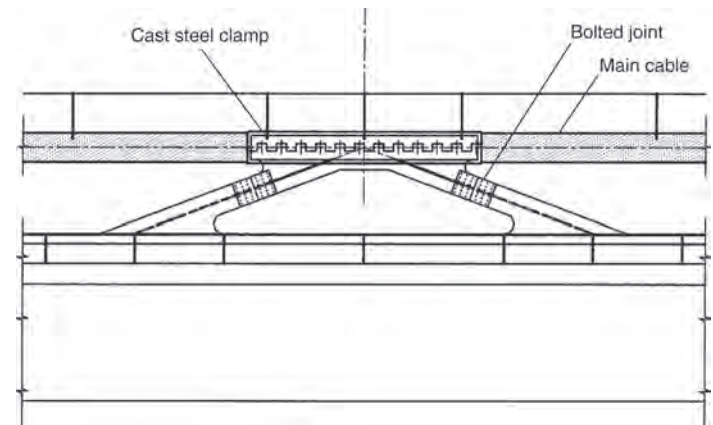


Figure 15.17 Rigid hanger at midspan

At very long-span lengths, the increase in box girder depth necessary to produce the required cross-sectional area in a single box girder may be excessive and for the 1991 m span Akashi Kaikyo Bridge the designers therefore chose an open truss stiffening girder. A more elegant solution is the division of the deck structure into two or more separate box girders linked by cross-girders, proposed by Richardson (1981) and adopted for the 1650 m main-span Xihoumen Bridge and the proposed Messina Straits Bridge.

Wind tunnel testing

Although great improvements have been made in the theoretical and computational analysis of the aerodynamic performance, using computational fluid dynamics (CFD), the complexity of the structure–flow interactions for practical bridge configurations is such that wind tunnel testing remains essential to validate proposed designs by testing small-scale models to simulate the wind conditions to which the bridge will be exposed. This testing is required to provide

- aerodynamic force coefficients for drag, lift and moment, from which the quasistatic wind loads on the structure can be calculated, and static stability (i.e. divergence) of the suspended deck evaluated
- verification of the response of the bridge to fluctuating wind (gusts) and vortex shedding and dynamic stability (flutter).

There are two complementary ways of carrying out these investigations, with the simpler being a sectional model test in

which an accurately scaled two-dimensional cross-section of the suspended structure is mounted in a wind tunnel and spring supported to give the required modal natural frequencies. As only a short length of deck (equal to the width of the tunnel cross-section) is required, a relatively large model scale can be used, enabling accurate representation of deck details, including parapets, and so on, with benefits to the accuracy of the simulation of aerodynamic forces. A further advantage is that, as the models are simple, rapidly produced and inexpensive, they can be easily modified to investigate the effect of changes to the proposed cross-section.

Wind tunnel testing of an aeroelastic model of the complete structure, including cables and towers, is also generally required, as changes in structural properties along the span can be included and three-dimensional effects, such as variations in the ambient wind conditions along the span, including the degree of turbulence can be simulated. Part-completed erection conditions, which are inherently three-dimensional, can also be fully investigated in this way. However, even in specially designed wind tunnels with very wide working sections, the model scale is inevitably much reduced from that possible with sectional models, so that the accuracy of cross-sectional details is reduced. The construction and testing of a full aeroelastic model is significantly more lengthy and expensive than for sectional model testing and is therefore usually appropriate only to confirm the aerodynamic behaviour of an essentially finalised design.

Hangers

Hangers are particularly prone to vibrations induced by periodic vortex shedding, as their dead load tension, size and construction type do not vary significantly along the span, but they have lengths that can vary from a few metres at midspan to as much as several hundred metres near the towers, resulting in a large range of natural frequencies, with a high probability that at least some hangers will be susceptible to this form of induced vibration, including the effects of rain–wind interaction. Possible measures to deal with this vortex shedding induced vibration include the attachment of Stockbridge dampers, linking the hangers with secondary stabilising cables, or the inclusion of dampers at the hanger–deck connection. These measures either prevent lock-on or modify the natural frequency, which in turn prevents lock-on.

Towers

In the permanent condition, the towers are longitudinally supported at saddle level by the main cables and are not likely to be susceptible to wind-induced vibrations. However, during erection, the towers are longitudinally free-standing cantilevers and therefore much more flexible longitudinally so that aero-

dynamic effects are more important.

Concrete towers have a high structure mass and, although generally rectangular, usually have rounded corners that decrease, to some extent, the intensity of any shed vortices and so are therefore unlikely to suffer from vortex-induced vibrations. By contrast, steel towers made up from welded stiffened plates have a much lower mass (of the order of one-fifth to one-seventh of a concrete structure) and low internal damping, and the sharp corners of the cross-section can produce strong shed vortices. It is possible, therefore, that a steel tower, either part-completed or at full height, will have a natural frequency coinciding with that of vortex shedding at wind speeds likely to occur during the construction period. If analysis and wind tunnel model testing predicts this, additional damping must be provided, either by an external friction damper or an internally mounted tuned mass damper.

Seismic design

The flexibility and relatively low natural frequencies of long-span suspension bridges make them relatively insensitive to the effects of low to moderate magnitude earthquakes without special provision having to be made in the design. However, in areas where significant magnitude earthquakes might be expected to occur during the design life of the bridge, the structure must be designed to resist the effects of seismically originating ground accelerations; for some elements, such as the deck bearings, towers and tower foundations, these may comprise the critical design loading. Although the bridge must be designed to resist the predicted maximum magnitude event without total or even partial collapse, some localised repairable damage will probably be inevitable and acceptable, with the elements most likely to be damaged being the deck bearings and movement joints. The extent of acceptable damage will largely depend on the extent to which the bridge is a vital transport link that must remain usable after an earthquake by, at least, emergency response vehicles.

As seismically induced loads are proportional to structural mass, the lightest practicable deck structure is desirable, to minimise seismic loads transferred to the towers by way of the deck bearings.

In addition to seismic loads transferred from the deck and cables, the towers and their foundations must resist those generated by the self-weight of the tower; for this reason, steel towers have generally been preferred for bridges in seismically active areas. A particular problem for long-span suspension bridge structures is that, as seismic ground disturbances travel through the ground at a finite velocity, seismic inputs to the foundations might not be simultaneous but might be separated by a delay of up to several seconds.

Design of bridge elements

Cable design

Number of cables

The natural transverse cable arrangement for a suspension bridge is with two main cables hanging in vertical planes, with each cable positioned at or close to the edges of the suspended deck, optimising torsional support to the suspended deck and providing a maximum usable width for traffic. However, for very long spans or a heavy suspended deck structure, the use of two main cables could result in a cable diameter larger than the practicable maximum size for construction; in this case, the preferred arrangement is to provide a pair of cables at each edge of the deck, with these cables being positioned at the closest practicable spacing permitted by the requirements of the construction, compacting and wrapping machinery. Two major bridges, namely the George Washington and Verrazzano-Narrows bridges in New York, utilise four main cables arranged in pairs. Improvements in deck design and higher cable wire strength have enabled subsequent long-span bridges, including the Akashi Kaikyo bridge to be constructed with a two main cable layout, although a four main cable layout was proposed for the very long span of the proposed Messina bridge.

Location of maximum tension

The maximum tension in the cable will generally arise from live load in all spans and occur at the point where cable slope is steepest. Except for bridges with very long side spans, this will be on the side span side of the tower; the tension at this point, therefore, also defines the cable size, including that of the main span. An alternative arrangement is to optimise the cable for the main span only, providing additional wires in the side spans only, anchoring these at the tower top, as shown in Figure 15.18.

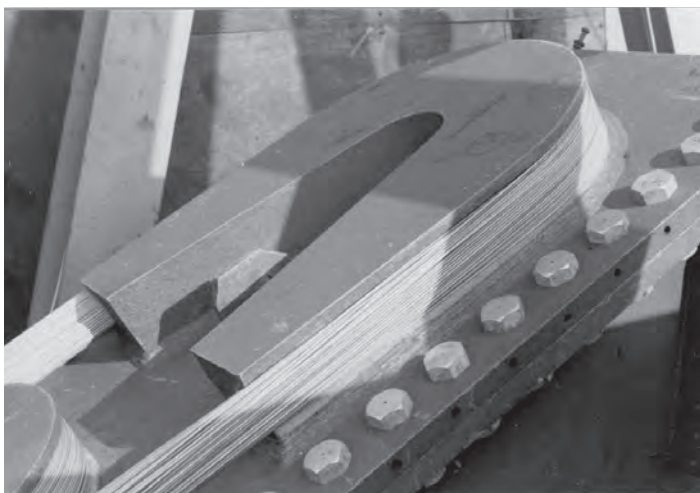


Figure 18: Anchorage for additional side span cable wires

Design stress level

Main cables are primarily subject to axial tensile stress only, with additional bending and transverse compression stresses principally occurring only in the areas of the tower and splay saddles, the cable clamps and the anchorage strand shoes. The predominant loading for long-span suspension bridge cables is the dead load of the cables and deck structure and can be as much as 80% to 90% of the total vertical loading.

The historic approach to permissible tensile stress level in the main cable has been to use a working stress design philosophy, determining this on a project-specific basis and limiting it to the order of 40% to 45% of the cable wire minimum guaranteed ultimate tensile stress. Limit state principles are now accepted as providing a more rational approach to the determination of design safety margins, and the partial factors selected for strength (γ_m) and loading (γ_F) need to produce an overall safety margin comparable to that which previous experience has shown to be acceptable.

Bending stresses

Parallel wire cables constructed by aerial spinning contain inbuilt bending stresses in each wire caused by the construction process, arising from the following effects:

- at the tower and splay saddles, bending of individual wires around the curved surface of the saddle trough
- at the anchorages, bending of individual wires around the curved surface of the strand shoe
- where additional strands are used in the side spans, bending of individual wires around the curved surface of the tower top anchor shoe.

For cables constructed from PPWSs, only the first of these is applicable, as the attachment of strands at the anchorage (and, if required, for additional side span strands at the tower) is by a socketed connection. With the cables hanging in free catenary at the completion of cable construction, and prior to the installation of the cable clamps, the only sources of non-uniformity in cable stress are due to wire or strand bending at saddles, wire bending at the strand anchor shoes and differences in wire lengths caused by erection tolerances. At this stage, the only significant transverse compressive forces in the cable are within the tower and splay saddles but, as there is no change in cable shape at this stage, any shear and bending resistance owing to inter-wire frictional forces is not brought into action.

However, once the cable clamps have been fitted and tensioned, these produce transverse compressive forces and the resultant inter-wire friction develops resistance to relative movement between individual wires and hence bending stiffness in the cable. The cable wrapping wire also produces a small transverse compression, but this effect can be ignored, so that relative wire movement is prevented only at the cable

clamps. When the addition of the suspended deck structure causes the cable to deflect from its free cable configuration to the final dead load geometry, the angular changes at the saddles and cable clamps develop bending stresses throughout the depth of the cable. The subsequent application of live loading and the associated deflections of the cable also causes additional transient bending stresses.

Suspended deck structure

General

The primary function of the suspended deck is to provide an aerodynamically stable and level traffic platform with acceptable deflection characteristics; if the combined main cable and suspended structure weight is sufficiently large, the cable gravity stiffness may be sufficient to achieve this. The George Washington bridge (as originally constructed) is an example. This is not, however, an efficient solution, and the inclusion of longitudinal bending stiffness in the suspended structure is required to distribute applied traffic loadings to hangers away from the point of application of the load and hence distribute the load more evenly to the main cables and prevent the development of unacceptable local deflections and deck slopes at concentrated load positions. The longitudinal bending stiffness also contributes to improving aerodynamic stability.

This stiffness requirement is effectively related to the hanger spacing; as this is a small proportion of the span length, the contribution of the deck bending stiffness to the overall vertical stiffness of the structure is generally insignificant, with that of the cable system dominating. This is not, however, the case for torsion, where the torsional stiffness of the deck can significantly improve stiffness, increasing the torsional natural frequency and improving aerodynamic stability. A high torsional stiffness is, therefore, an important requirement of the deck structure. It is a characteristic of all bridges that the contribution of dead load to total bridge loading rapidly increases

with increase in span length; for suspension bridges, this effect is illustrated in Figure 15.19, showing, for a single span with a given allowable cable stress and span/sag ratio, the rapid increase in the ratio of cable weight to suspended deck weight as span length is increased.

An efficient design, therefore, requires a suspended structure with the minimum self-weight consistent with the achievement of other design requirements; thus, structural steel is invariably used for the deck and stiffening girder. An important contribution to minimising dead load is the use of an orthotropically stiffened steel deck plate, combined with the thinnest surfacing layer that will provide an adequate deck fatigue life. Reduction in deck weight produces savings in the other elements of the cable system (hangers, towers and anchorages), with the benefit becoming increasingly significant as the span length increases. However, the effect of reduced dead load on the deformational behaviour of the bridge must also be considered, as the overall gravitational stiffness of the structure is reduced and, as a result, deflection under live load will be increased.

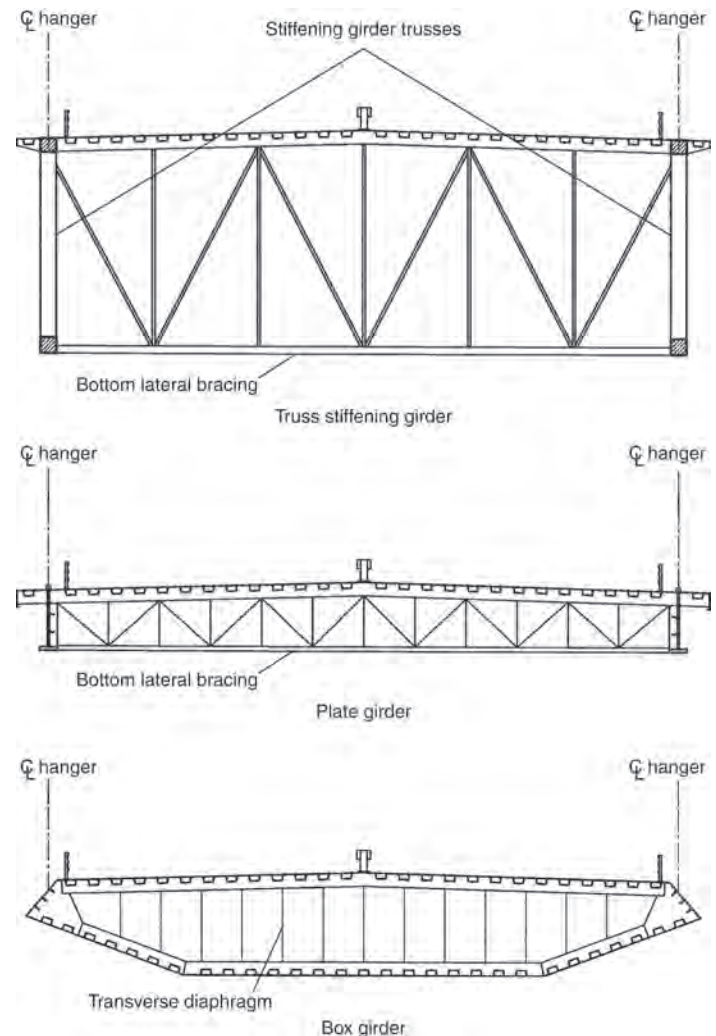


Figure 15.20 Deck configurations

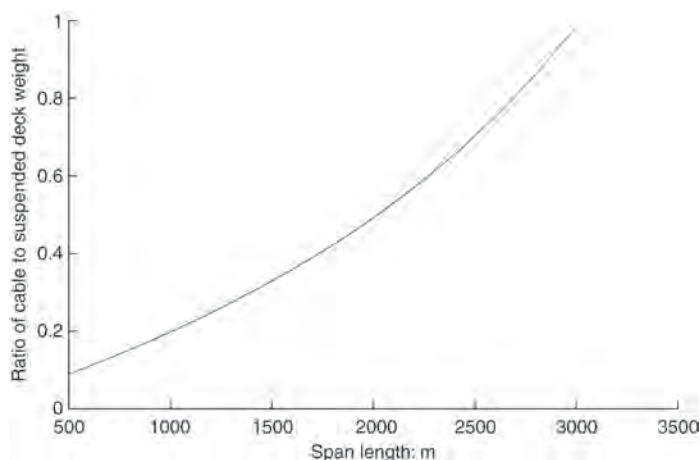


Figure 15.19 Ratio of cable to deck dead weight

The options for the suspended deck stiffening girder structure (see Figure 15.20) are

- an open truss stiffening girder with a separate traffic deck
- a solid web stiffening girder with a separate traffic deck
- a box girder structure combining the traffic deck and stiffening functions.

Prior to the introduction of slender box girder deck structures, the preferred arrangement to provide flexural and torsional stiffness consisted of two vertical truss girders positioned at or near the deck edges, connected by upper and lower plan bracing systems to provide torsional stiffness with transverse sway bracings at regular intervals to maintain the cross-section shape. The positioning of the stiffening structure below deck level enables the traffic deck structure to provide the upper bracing function by acting as a horizontal web plate between the truss upper chords.

Where a combination of road and rail traffic is required, a truss-stiffened deck can be the preferred stiffening structure, as rail traffic can be carried on a naturally ventilated lower deck level and the high flexural stiffness required by the more deflection-sensitive rail traffic is more easily obtained. Suspension bridges have been constructed with their decks stiffened by twin plate girders, but because of the poor aerodynamic characteristics of this arrangement (as exhibited by the Tacoma bridge), this is suitable only for shorter span lengths.

Deck articulation and bearings

Most suspension bridges have separate deck structures in the main and side spans, with the ends of the main and side span stiffening girders supported by bearings fixed to the tower, and with the main movement joints for the bridge located there. The bearings must accommodate large longitudinal movements and might also have to resist uplift forces in some loading conditions. Rocker bearings have, therefore, been a natural choice. Sliding spherical bearings can be used, provided that the load distribution between the cable and the stiffening girder is adjusted to ensure that the bearing reaction is positive for all load conditions, or an alternative means of resisting uplift forces is incorporated.

As the maintenance of bearings and movement joints constitutes a substantial part of the total maintenance costs, suspension bridges can be designed with their deck structures continuous between anchorages, with only lateral bearings at the towers. Deck continuity through the towers improves the vertical and horizontal stiffness of the structure but the absence of vertical support at the towers removes the beneficial effect on aerodynamic stability of torsional fixity at the towers.

A continuous deck through the tower will experience increased bending moments in this area as it conforms to the angular rotation of the main cable at the saddles, and there will be increased deck longitudinal movements at the anchorages. These increased movements require the provision of large

movement joints at the anchorages, as well as hydraulic damping devices (as on the Storebælt East and Høga Kusten bridges) to provide span-wise location of the deck, and resist traffic braking or wind loads while permitting movement in response to temperature changes.

Tower saddles

Tower saddles support and deflect the main cables and can be either a longitudinally curved cast trough, supported by a grillage of welded steel plates that transmit and diffuse the cable load into the supporting tower structure, or a complete steel casting. The shape of the trough is determined by the type of cable.

- For cables of locked coil or spiral strands, the bottom of the saddle trough will consist of a series of stepped circular grooves, so as to provide full bearing for the bottom layer of strands. The gaps between the strand layers are filled with shaped zinc or aluminium sections.
- For parallel wire cables, the bottom of the trough has a stepped rectangular cable shape, with the number of steps corresponding to the number columns of strands in the cable hexagon.



Figure 15.21 Tower saddle for spiral strand cable

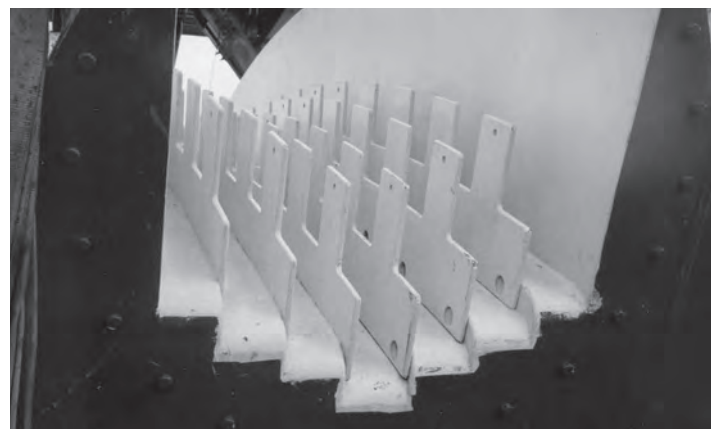


Figure 15.22 Tower saddle for parallel wire cable

Typical arrangements of these saddle types are shown in Figures 15.21 and 15.22. As parallel wire cables are by far the most common type of cable, only saddles for these are considered further.

The trough grooves need to produce a cable shape in the saddle close to that of the compacted cable on either side of the tower; this is achieved by having an approximately square strand arrangement in the saddle, with equal numbers of wire rows and columns.

The groove width is determined to suit this arrangement, assuming alternating rows of N and $(N-1)$ wires, as shown in Figure 15.23, although the top row of wires will generally be incomplete. The height of the saddle side walls is determined by the efficiency of packing of the individual wires in the saddle. Although the wires are packed as closely as practicable during positioning in the saddle groove, edge effects and minor inaccuracies in wire packing cause the achievable voids ratio to be somewhat larger than the theoretical minimum value, and a figure of between 14% and 16% is therefore a realistic value for a spun cable, with the value for PPWS cables probably being slightly higher, being dependent on the care taken in strand reshaping and placement in the saddle. To facilitate spinning, permanent steel separator plates are required to separate the strand columns; a typical arrangement is as shown in Figure 15.22. These support the strands laterally during spinning and separate them for ease of strand movement through the saddle during strand sag adjustment.

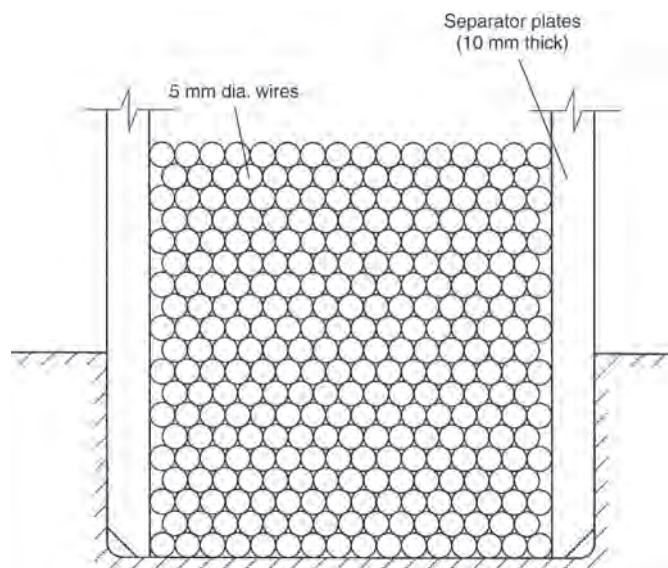


Figure 15.23 Wire arrangement in saddle strands

The wires of parallel cables are laid individually during spinning, or in relatively small numbers for PPWSs, so that the whole cable does not experience significant bending during construction and only a relatively low ratio of saddle curvature to cable diameter is required to minimise compressive forces between wires induced by their bending. These are proportional to the cable tensile stress, saddle radius and cable overall

depth in the saddle, have a maximum in the bottom wires and cause a reduction in the effective yield strength of the wire. A cable centre line radius of curvature of the order of nine to ten times the main cable diameter has previously been found acceptable; representative examples for some modern bridges are given in Table 15.1. The saddle trough length is determined by the maximum cable slopes at each side; it is usual practice to dimension the saddle to the tangent points of the full dead load condition, and provide sufficient saddle length beyond these points to support the cable under the most adverse live load combination, or during erection of the deck.

	Cable diameter: mm	Saddle radius: mm	Ratio
Humber Bridge	684	6 000	8.8
Fatih Sultan Mehmet Bridge	763	7 400	9.7
Storebælt East Bridge	827	7 000	8.5
Tsing Ma Bridge	1095	12 500	11.4

Table 15.1 Examples of tower saddle radii

The distribution of the saddle radial loading into the tower structure produces longitudinal bending stresses and transverse loading on the saddle side walls produced by diagonal contacts between wire layers. These produce lateral forces between the wires that are additive from each wire layer so that the side wall pressure increases throughout the depth of the saddle to a maximum value at trough base level. The lateral forces are related to the efficiency of wire packing; for perfect (60°) hexagonal packing of the wires, ignoring friction between the wires, the ratio of horizontal pressure on the saddle walls to the vertical pressure at that level is about 0.33. However, this quality of packing cannot be generally achieved, and actual pressures will be somewhat greater than this. Inter-wire friction produces a reduction in lateral pressure but as this cannot be assessed accurately it should be ignored for design. Pre-tensioned transverse tie bolts installed above the cable prior to suspended deck erection can be used to reduce the sidewall stresses arising from deck load and live loading.

During cable construction, the saddle must be offset towards the bridge side span to enable the cable to be built to its 'free cable' geometry. This offset can be achieved in two ways.

- The saddle can be permanently fixed to the tower top in its final position, with the required offset obtained by deflecting the tower into the side span using some form of pull-back system. This method is most appropriate for steel towers, but has also been used for tall, relatively flexible concrete towers (e.g. the Storebælt East bridge).
- The saddle can be positioned on a bearing and initially offset to the free cable position. In this method, which is most appropriate for relatively stiff concrete towers, the tower is maintained vertical during cable construction. The saddle is then moved into its permanent position on the tower top during construction of the bridge deck. For some concrete towers, a combination of saddle offset and pull-back of the tower may be necessary.

Anchorage (splay) saddles

The splay saddle separates the cable into its individual strands, deviating them both laterally and vertically to their connection points on the anchorage face; a typical example is shown in Figure 15.24.

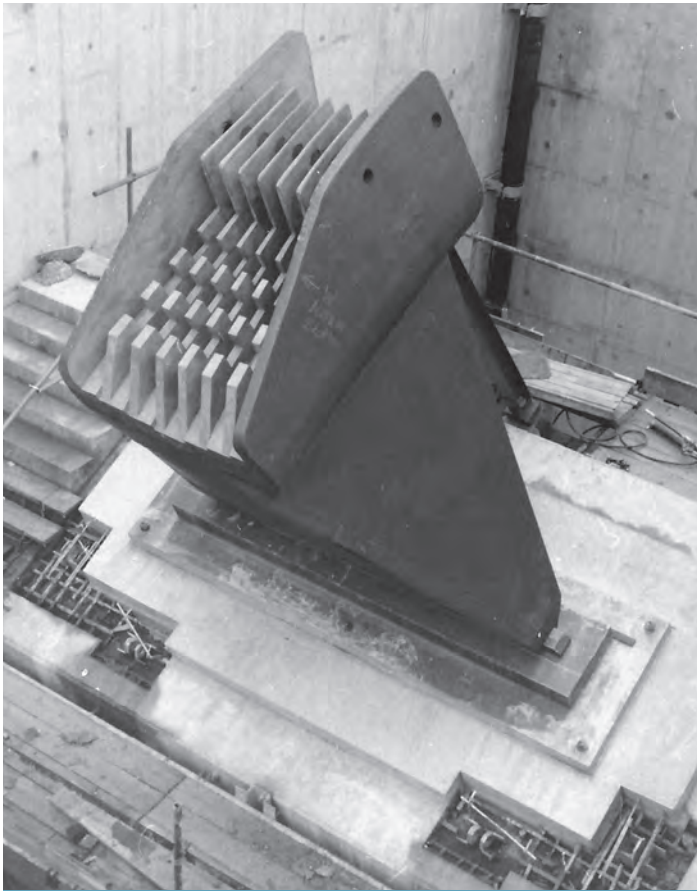


Figure 15.24 Typical splay saddle

In order that strands can be placed layer by layer during cable construction, without the need for temporary restraint, the saddle should deviate all strands downwards relative to the cable centre line at the lower end of the side span, producing an overall downwards deflection of the cable centre line. To achieve the vertical and lateral strand deviations, the saddle trough is curved in both elevation and plan, as shown in Figure 15.25. If strands are deflected horizontally in the absence of vertical pressure, wires tend to drift outwards and pile up on the outside of the trough, so that horizontal curvature of strands should only be carried out when they are also being deflected vertically.

As at the tower saddle, the trough base has a stepped transverse profile to suit the hexagonal arrangement of the cable strands. To produce a resultant force directed through the centre of the saddle, the bearing pressure on the trough must be kept constant, and this can be achieved by increasing the trough longitudinal curvature so that the cable tension divided by the



Figure 15.25 Splay saddle trough separators

bend radius is kept constant as the strands peel off towards the rear of the saddle, as shown in Figure 15.26.

To accommodate length changes in the strands within the anchor chamber, owing to fluctuations in live loading and strand temperature variations, the splay saddle has to be able to move longitudinally, and the saddle must also be initially positioned

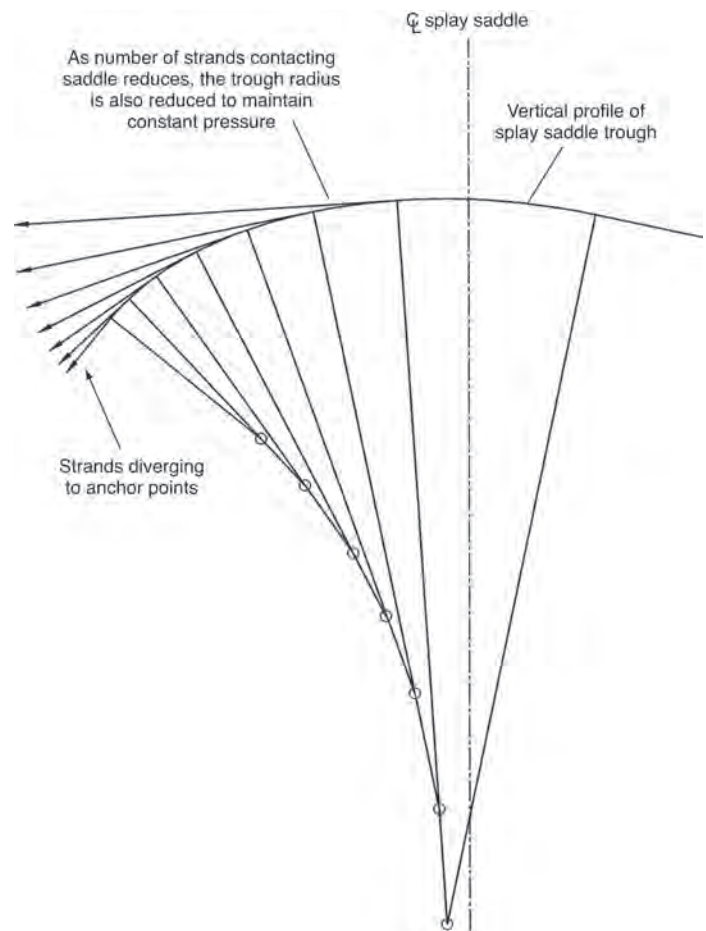


Figure 15.26 Splay saddle geometry

offset towards the strand anchor points so that extension of the strands within the anchor chamber caused by the addition of the suspended structure brings the saddle to its nominal dead load position. These movements can be permitted by mounting the splay saddle on a sliding or rolling bearing, but a better solution is the arrangement shown in Figure 15.24, with the longitudinal movements being generated by saddle rotation on a rocker bearing.

Any water leaking into the side span cable tends to run down to the splay saddle, exiting at the rear of the saddle, where the lateral divergence of the strands produces small horizontal gaps, which are extremely difficult to protect and so particularly susceptible to corrosion. Protection of corrosion in this area involves

- filling the vertical gaps between the strands at the rear of the saddle with solid spacer plates
- providing the anchorage chamber with a dehumidification system to maintain humidity levels below the threshold value for initiation of corrosion.

Cable clamps

The cable clamps that connect the suspended deck structure hangers to the main cables are divided into two half sections to enable them to be fitted to the cables. Apart from at the mid-point of the main span, the main cables are everywhere inclined to the horizontal, and the hanger load has a down-slope component parallel to the main cable axis that must be resisted by friction between the cable clamp and the cable wires. The two half sections are shaped internally to be a close fit to the compacted main cable and are forced into tight contact with it by highly tensioned connecting bolts to generate the required frictional resistance. For economy of manufacture and convenience in tightening, the same diameter bolt is preferably used throughout, with the number in each clamp varied to suit the cable slope and hence the friction force required.



Figure 15.27 Cable clamp for looped hanger

The form of the cable clamps is determined by the type of hanger employed and, for two-part wire rope hangers attached by looping over the main cable, is a constant-diameter cylinder (Figure 15.27) with local enlargements at the bolt positions and a transverse groove to locate the hanger rope, with each half section provided with interlocking and overlapping castellations to transmit longitudinal shear, ensure accurate longitudinal alignment of the two halves, and prevent wires being trapped during installation of the bands and tightening of the clamping bolts. For single-part wire rope or strand hangers, a connection is required on the underside of the clamp, which must therefore be divided horizontally, with the lower half of the clamp being shaped into a vertical gusset plate for a pin connection to the hanger socket; again, each half clamp is provided with interlocking and overlapping castellations, as shown in Figure 15.28.

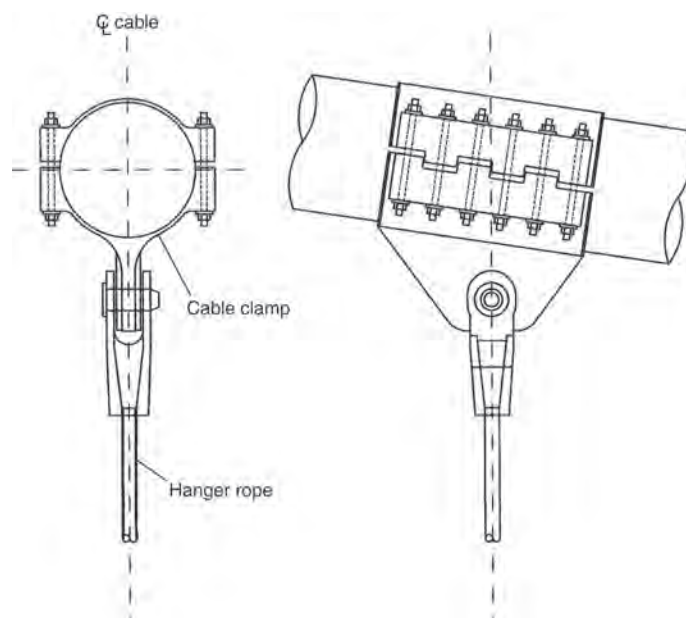


Figure 15.28 Cable clamp for single-part hanger

Tensioning of the clamping bolts causes radial deformation of the clamp, as its bore is forced into tight contact with the cable; therefore, circumferential bending flexibility is desirable, requiring a low wall thickness (normally between one-twentieth and one-thirtieth of the cable diameter) and a material with high ductility. Cable clamps have a complex shape; the only viable manufacturing process is, therefore, production as steel castings. Casting and machining of cable clamps is a lengthy process, and the bore diameter of the cable clamps must be finalised, and manufacturing largely completed prior to cable compaction and determination of the actual compacted cable diameter. Manufacture must therefore be based on a prediction of the compacted diameter based on data from previous cables.

Since a high frictional resistance between the cable wires and the inside surface of the clamp is required, a high standard of surface finish on the bore is undesirable, and slight roughening by machine finishing the bore with shallow closely spaced

circumferential cuts that are just visible to the naked eye is normally specified, with zinc metal spray for corrosion protection, as it has a slightly rough texture and is compatible with the galvanising of the cable wires. The coefficient of friction that can be developed between the cable clamp and the cable wires is usually assumed to be around 0.2, although few reliable full-scale data are available, with the only recent published data being those from tests carried out during construction of the Storebælt East bridge (Gimsing, 1998) that indicated friction coefficients of only around 0.17 at the start of sliding.

The resistance to sliding is a function of the total radial load across the interface between the clamp and the cable wires, and the distribution of this load depends on the relative stiffness of the cable and the clamp. For a thick and relatively rigid clamp, the bearing pressure will vary from a peak at the centre line of each half clamp down to zero at the interface between the two halves, with the actual distribution being determined by the relative cable and clamp stiffness. The total load applied to each half clamp will, however, be equal to $2NP$ where N is the number of bolts on each side of the clamp and P is the tensile load in each clamping bolt. The total frictional resistance to sliding will then be

$$F = 2 \times \frac{2NP}{\gamma} \quad (15.67)$$

where γ is the partial factor of safety

For cable clamps with thinner walls, bending stiffness reduces much more rapidly than does the axial stiffness and the behaviour of the clamp approaches that of a flexible strap surrounding the cable, with the bolt's tensions being reacted by radial pressure and tangential friction forces, and a more realistic assumption for assessment of the frictional resistance is required. With N bolts on each side of the cable clamp, each tensioned to a load of P , the total tension force applied to each quadrant of the band is $(P \times N)$. As the bolts are tensioned, the clamp is pulled into tight contact around the complete periphery of the cable, and frictional loss due to relative circumferential movement between the clamp and the cable wires causes the tension in the clamp to reduce progressively, so that, at an angle θ from the dividing line between the clamp half sections, it will be $NP e^{-\mu\theta}$ with a minimum value of $NP e^{-\mu\pi}$ at the centre line of each half section, where μ is the effective coefficient of friction. The radial contact force on a small element $d\theta$ of the clamp–cable interface at an angle θ is

$$F = 2NP e^{-\mu\theta} \sin\left(\frac{d\theta}{2}\right) \approx NP e^{-\mu\theta} d\theta \quad (15.68)$$

The total radial force on all four quadrants of the interface is

$$F_{\text{TOT}} = 4 \int_0^{\frac{\pi}{2}} NP e^{-\mu\theta} d\theta = 4NP \left[-\frac{e^{-\mu\theta}}{\mu} \right]_0^{\frac{\pi}{2}} \quad (15.69)$$

giving, finally

$$F_{\text{TOT}} = \frac{4NP}{\mu} [1 - e^{-\frac{\mu\pi}{2}}] \quad (15.70)$$

As the two parts of the clamp are mechanically keyed together by the castellations, the total frictional resistance against sliding along the cable is equal to $\mu_L F_{\text{TOT}}$, or

$$\frac{4NP(\mu_L)}{\gamma} [1 - e^{-\frac{\mu\pi}{2}}] \quad (15.71)$$

where μ_L is the effective coefficient of friction for longitudinal sliding and γ is a partial factor of safety.

Experience has shown that the tension applied to the clamping bolts reduces rapidly after initial tightening, owing to a combination of creep in the bolt steel and the zinc coating of the cable wires, combined with continued settling and slight repositioning of the wires under the radial pressures exerted by the cable band bolts, with this latter effect being particularly significant during the increase in main cable tension as the suspended structure is constructed. To compensate, the clamping bolts must be re-tensioned at intervals during erection of the suspended structure, with a final tightening carried out as late as practicable before bridge completion. However, even with these retightening operations, relaxation in bolt tension will continue to occur during the service life of the bridge, and this can result in a significant long-term reduction in clamping force. To allow for this, the resistance to sliding should be calculated by assuming that the effective long-term clamping force will only be a proportion of the initial value, and a value of 70% has been proposed (Iwaya *et al.*, 1993), based on Japanese and other experience.

Cable clamp bolts are specially manufactured high-tensile steel screwed rods, tightened to a very high load; it is advantageous to use waisted rods, in which the central unthreaded section of the rod has a reduced diameter, enabling the rods to be tensioned up to or slightly beyond their proof stress without the risk of thread deformation. Deformation of the clamp half sections during bolt tightening causes rotation of the contact faces between the clamp and the bolts, which can be accommodated by using spherical washers below the nuts.

The vertical load from the hanger induces an angular change in the cable axis that, because of the relative rigidity of the cable clamp, will largely occur in the cable immediately adjacent to its ends. Equalisation of these angular changes will minimise their effect; ideally, this requires that the line of action of the hanger should pass as near as practicable through the intersection of the cable axis and the centre point of the clamp, requiring every clamp to ideally have a slightly different hanger pinhole or groove position. This is not practicable, particularly for grooved clamps for two-part hangers, where a separate casting pattern would be required for every clamp; therefore, normal practice is to adopt the same groove geometry for a series of clamps, allowing for the slight eccentricities that arise. In clamps for single-part hangers, the problem can, to a large extent, be eliminated by the use of an oversize lug, with the pinholes positioned to suit individual hanger locations.

The initial sizing of the cable band can be made by making an assumption for the initial lack of fit and the subsequent distribution of bearing stresses at the bore–cable interface but the complex geometry of most cable bands requires a finite-element analysis, preferably using a range of initial lack of fit and cable radial compressive stiffness. The distribution of bearing pressures at the interface between the cable band and the main cable can then be used to determine the resistance to longitudinal sliding.

Both the exposed central section of the bolts and the lengths in the clearance holes within the cable clamp need to be protected against corrosion by filling the annular space between the bolts and the clamp with an injectable flexible sealant, as well as sealing the central gaps between the castellations with a similar flexible sealant.

Hangers

Hangers are normally aligned vertically and located along the span at equal intervals, with a spacing close enough to distribute the suspended deck dead load as an almost continuously distributed load on the cable. The selected spacing must also be a multiple of the length of the suspended deck erection sections so that, as each section is lifted into position, it can be supported by at least one pair of hangers. Hangers inclined along the axis of the bridges to form a triangulated system between the deck and cable have been used on a few bridges, the first application being on the Severn Bridge (1966), and similar hanger layouts have subsequently been used on the First Bosphorus Bridge (1973) and the Humber Bridge (1981). However, this arrangement causes the hangers to experience a significantly greater range of stress fluctuation due to live loading, and a conventional vertical hanger arrangement is now generally recognised to be preferable. There are two basic forms of hanger.

- *Two-part (looped) hangers.* This type of hanger was used for most early suspension bridges and consists of a single length of wire rope located in a groove in its associated cable clamp to drape over the main cable, with its lower ends attached to the suspended deck. This type of hanger must be sufficiently flexible to conform to the radius of the cable clamp groove and is therefore usually made of steel wire rope. With large-diameter cables, the separation of the two parts of the hanger can be reduced by bringing them together with a spacer positioned just below the cable. Additional spacers can also be fitted at intervals along longer hangers to help in avoiding vibration problems but, as such connections can accumulate dirt and moisture, they are best avoided. The connection of the hanger lower sockets to the deck can be either by end bearing on part of the stiffening girder or by a pin connection.
- *Single-part hangers (Figure 15.28).* These consist of a single length of wire rope, spiral or locked coil strand, positioned on the centre line of the cable, and attached to the underside of the cable clamp by a socket-and-pin connection. The connection of the lower end of the hanger to the stiffening girder or deck structure can be a simple bearing socket but is more usually a socket-and-pin joint. The advantage of this type of hanger is that, as it does not need the flexibility to bend around the cable, parallel wire strands encased in a plastic sheath can also be used, taking advantage of their superior strength, stiffness and durability.

The design of hangers is governed by local loading effects, with the maximum design load being the passage of a heavy concentrated load. However, repeated load fluctuations, owing to traffic loading, and relative longitudinal and lateral movements between the main cables and the deck make hangers susceptible to fatigue damage. The effects of relative move-

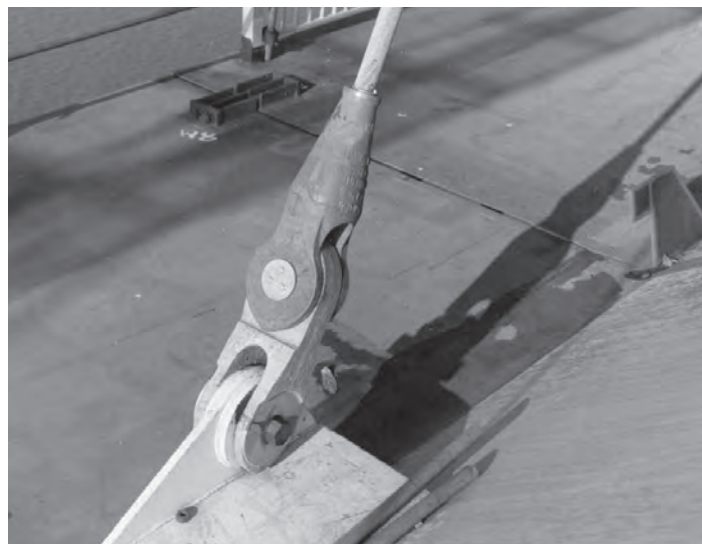


Figure 15.29 Articulated hanger connection to the deck

ment are most severe near the midspan areas and near the anchorages, where the short hangers provide the most restraint against deck deflection from transverse wind, and at the lower end of the hanger, where the environment is most conducive to corrosion damage. Hanger end attachments must therefore be designed to minimise bending actions and preferably avoid these in areas known to be susceptible to fatigue damage, such as the entry point of the hanger into its end sockets.

Articulation (Figure 15.29) of the hanger attachment to the deck or the inclusion of spherical bearings for pin-connected hangers that permit rotation in both axes can alleviate the

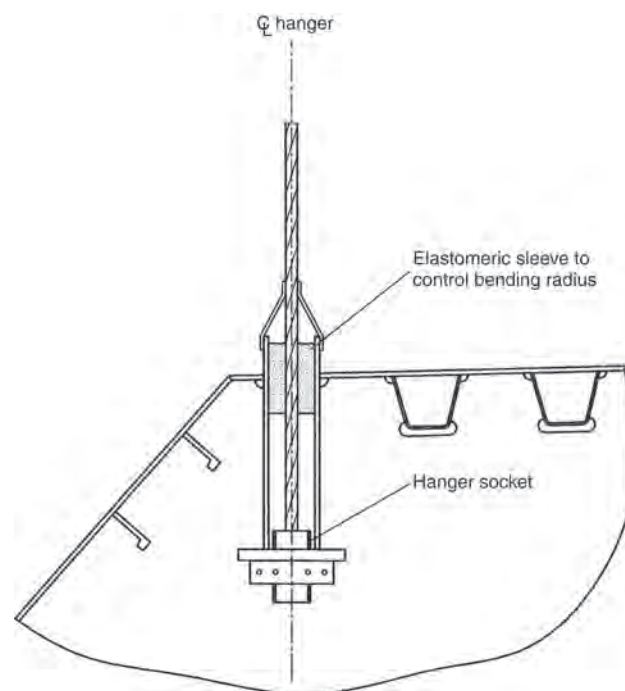


Figure 15.30 Internal hanger attachment

problem but the frictional moments inherent in pin connections inevitably still induce some bending moments and stresses in the hanger.

An alternative solution is to connect the lower end of the hanger with a bearing socket located inside the structure to provide maximum protection from the environment, combining this with control of the hanger curvature away from the socket entry point using an elastic sleeve around the hanger, as shown in Figure 15.30.

Even for well-designed and well-maintained hangers, the effects of fatigue or corrosion will probably cause sufficient deterioration and loss of strength that replacement will be required at some stage during the life of the bridge. The design of the hanger–cable clamp–deck system should therefore permit the temporary removal of a hanger for replacement, without the need for traffic restrictions, other than the closure of the immediately adjacent lane.

In addition to the beneficial effect on aerodynamic stability, the incorporation of a rigid hanger, as shown in Figure 15.17, at the centre of the main span has the additional advantage of protecting the adjacent short midspan hangers from bending fatigue damage by preventing both longitudinal and transverse movements of the deck relative to the cables.

Cable terminations in the anchorage

The way in which the individual strands of the cable are attached depends on the type of cable construction. In parallel wire cables constructed by in situ spinning, the wires are looped around a semicircular strand shoe, as shown in Figure 15.4, with the wires progressively laid into tapered grooves on either side of the anchor rods that attach them to a base plate prestressed to the concrete face of the anchorage. These anchor rods also enable repositioning of the strand shoe after spinning to allow for strand sag adjustment. A radius of about 400 mm has generally been used for the strand shoes of spun cables with 5 mm diameter wires. The bending of the cable wires around the strand shoe induces peak bending stresses in each wire of $Ed/2R$, where E is the wire modulus of elasticity, d is the wire diameter and R is the strand shoe radius. With these dimensions, the resulting wire bending stress is 1281 N/mm², comparable to the 0.2% proof stress of the wire material. However, strand shoes of these proportions have given satisfactory performance on previous bridges, probably because the stress is localised and reduced by some plasticity and redistribution.

For PPWSs, locked coil or spiral strand cables, the individual strands have end sockets attached to the anchorage structure by either screwed rods or end bearing onto anchor beams. In the latter case, adjustment for minor errors in strand length is made using shim plates between the socket and the anchor face.

Towers

Towers are generally configured with individual vertically oriented shafts positioned directly below each cable plane and connected by transverse bracing for stability and to resist

lateral wind loading. The bracing system can be either diagonal or portal members, or a combination of these, as shown in Figure 15.31.

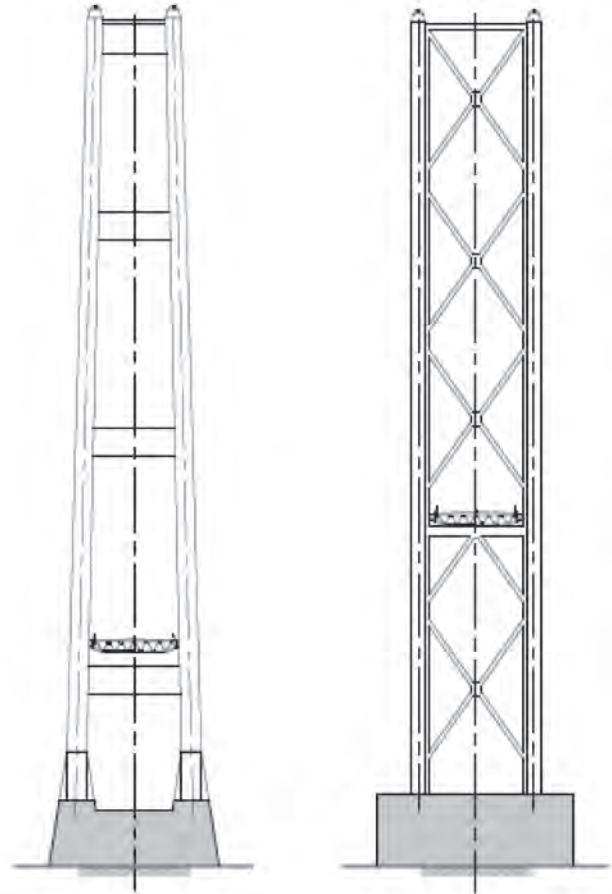


Figure 15.31 Tower transverse bracing types

Both steel and concrete are feasible materials for tower construction; as towers are predominantly loaded in compression, concrete is an obvious first choice of construction material. The increased self-weight over that of an equivalent steel structure may, however, result in an uneconomic foundation if ground conditions are relatively poor, and this is particularly so for bridges in areas of moderate or severe seismicity.

As the predominant force carried by the tower is compressive axial load from the cables, combined with bending in response to the eccentricity of its application, its cross-section must be arranged to produce the most effective column section, with its material placed at the maximum practical distance from the centroid. The natural choice for a steel tower is, therefore, a rectangular cross-section, made up of four stiffened plates, and a similar hollow rectangular cross-section is also appropriate for a concrete tower, with typical cross-sections of steel and concrete towers shown in Figure 15.32.

The principal loading on the tower is the vertical load applied to the tower top by the cables, together with deck lateral wind load transmitted by the cables to the tower top, and the wind

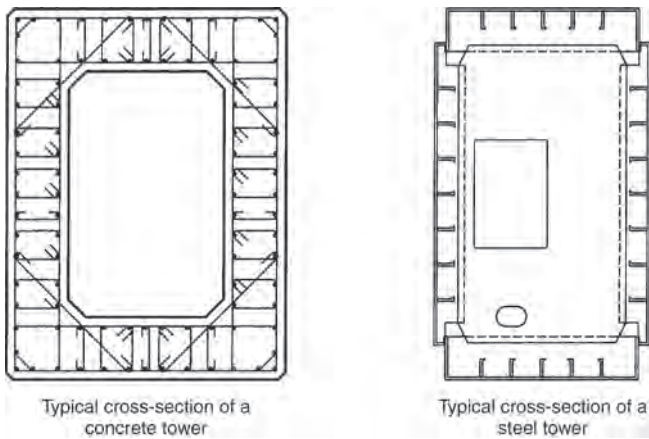


Figure 15.32 Tower cross-sections

loading on the tower itself. Of less importance are the vertical and lateral forces applied to the tower at a lower level by the suspended deck bearings.

An increase in vertical load on one span of the suspended deck causes a corresponding increase in the horizontal component H of the cable tension in that span and, unless the tower saddle is free to move, causes an unbalanced longitudinal load applied by the cable at the tower top. Resisting this imbalance using a very stiff tower would produce very large bending moments on the tower, so the tower design must permit the cable saddles to move until a balanced horizontal force condition is restored.

All modern long-span suspension bridges, therefore, have flexible fixed base towers with fixed saddles, relying on bending of the tower legs to produce the required saddle displacements. For such a tower, the design loading is, therefore, that which produces the most adverse combination of cable vertical load and coincident longitudinal displacement of the tower top, and two extreme conditions can be identified.

- Maximum vertical load applied to the tower, in combination with the tower top displacement corresponding to that load. This is most likely to result from live load applied to all spans of the bridge, with only a relatively small displacement of the tower top.
- Maximum tower top displacement in combination with the vertical load corresponding to that displacement, an example being live load in one side span, which will produce displacement of the tower top into the side span but with a smaller vertical load on the tower.

The most adverse combination might also arise from some condition intermediate between these extremes.

The horizontal displacement of the tower axis from the vertical induces (P -delta) bending stresses in the legs from both the cable vertical load and the tower self-weight, in addition to the direct compressive stresses. With a relatively stiff tower, the extent of the horizontal displacement of the tower top is reduced, and (P -delta) stresses minimised, but at the

expense of an increased horizontal out-of-balance load from the cable saddle.

A slender, more flexible, tower reduces the longitudinal out-of-balance cable force, but deflections and hence P -delta stresses are increased. Although it is difficult to generalise, the most economical design will have a tower of the highest practicable slenderness, compatible with the need for

- an adequate margin against overall buckling of the tower
- sufficient strength and stiffness for the tower to be safely erected as a free-standing vertical cantilever.

As the stresses in the tower leg are a function of the deflected shape, the analysis of towers, prior to the availability of large-displacement analysis software, required an iterative approach, with an initial deflected shape being assumed and the resultant bending moments used to calculate an improved shape, and iteration continuing until convergence was achieved. Overall buckling of the tower is determined by its end support conditions. The base of the tower is fixed in both position and direction, but for the tower top it is necessary to consider whether the cable system provides sufficient stiffness to prevent longitudinal movement of the saddle other than that dictated by the applied loading. It can easily be demonstrated (Pugsley, 1968) that this is, in fact, the case and that, for longitudinal buckling, the tower behaves as a propped cantilever.

Anchorage

The anchorages secure the ends of the main cables and transfer their force into the ground. The direction of the cable forces is determined by the side span geometry but is predominantly horizontal, with a smaller upward component; resisting this predominantly horizontal force is difficult unless reasonably



Figure 15.33 Strand divergence in anchor chamber

good ground conditions exist. The individual strand anchor points are normally arranged in a closely spaced hexagonal or rectangular array on an anchor face positioned sufficiently far from the splay saddle, so that the angular deviation of the outer strands (Figure 15.33) does not exceed about 10° .

The strand forces must be transmitted and diffused into the anchorage block; in early suspension bridges, this was achieved by transmitting the cable force through a chain of embedded eyebars to anchorage girders at the back of the anchorage. The same result is nowadays more efficiently achieved by attaching the strand shoes or end sockets to anchor slabs post-tensioned against the face of the anchor block concrete (see Figures 15.4 and 15.33).

Where sound unfaulted rock exists at or very close to the surface, it will be possible to transfer the cable forces directly to the rock; a tunnel anchorage can be used with a length penetrating sufficiently far into sound unfaulted rock that sufficient mass can be mobilised to resist the cable force, and with the upper part of the tunnel forming the splay chamber. In most cases, however, a 'gravity' anchorage will be required, in which the horizontal loads from the main cables are resisted principally by friction between the underside of the foundation and the supporting strata. The anchorage can be partly or wholly embedded in the ground and a stepped base used to provide improved resistance by utilising the passive resistance of the ground. The development of the necessary resistance against sliding by base friction requires the anchorage to have a very large mass, as can be deduced from consideration of the horizontal equilibrium of the anchor block:

$$C \cos \theta = \mu (M - C \sin \theta) \quad (15.72)$$

where C is the total cable force, M is the dead load of the anchor block, θ is the cable angle to the horizontal and μ is the base coefficient of friction. The required anchor block dead load is therefore

$$M = C (\sin \theta + \frac{\cos \theta}{\mu}) \quad (15.73)$$

The inclination of the cable to the horizontal will generally be around 10° – 15° , and typical values of the base friction could be in the range 0.3 to 0.5. Taking the more adverse of these values, the required value of M would be $3.48C$, that is, the anchorage dead load will need to be of the order of 3.5 times the total cable load, so that gravity anchorages are necessarily very large and heavy structures.

As the cable force is predominantly horizontal, it will produce a moment of approximately $(C \times h)$ about the front edge of the anchor block, where h is the height of application of the cable force above its base level. This produces a pressure distribution with a maximum at the front edge of the anchorage, reducing progressively towards the rear, and it is therefore advantageous to concentrate the mass of the anchorage towards the rear, producing an opposing distribution to that from the cables, with an almost uniform resultant total loading. Consideration must also be given to the construction condition, when the cable force is either absent or much reduced, and the anchorage configured to ensure that allowable bearing pressures are

not exceeded in this condition. This may require a proportion of the anchorage dead load to be added concurrently with the construction of the suspended deck structure. The design must include consideration of all possible soil failure modes, including slip circle failure of the ground initiated by the vertical bearing stresses under its base. As gravity-type anchorages for long-span bridges are necessarily massive structures with large horizontal and vertical dimensions, careful design is required to produce an aesthetically acceptable solution; possible ways this can be achieved are by

- placing as much as possible of the anchor block below ground level
- using a cellular base structure, with the cells filled with a high-density material to maximise the anchorage's mass relative to its plan dimensions
- aligning the structure edges parallel and normal to the cable axis, so that the function of the anchorage in resisting the cable pull is logically expressed
- using suitable architectural features, such as ribbing to break up the monotony of large concrete surfaces
- using an open structure, again with as much as possible of the required mass placed below ground.

Figure 15.34 (Storebælt East Bridge) shows a particularly effective use of the last of these methods in a marine environment, in which the required mass has been concentrated mainly below water level, with an open frame structure used to minimise the visual impact.



Figure 15.34 Storebælt East Bridge anchor block

Construction

General

Although suspension bridges can be erected without the need for any temporary intermediate supports, considerable temporary works are required for their construction; as some of these

are unique to this type of bridge, an understanding and appreciation of them is therefore an essential input to their design.

Tower construction

The base sections of land-based steel towers can be erected using large mobile or crawler cranes, with floating cranes used for towers in offshore locations, but generally the tower height is too high for erection to be completed with such cranes and erection must therefore be continued with specially designed climbing or tower cranes. During erection, the towers are free-standing vertical cantilevers, and the critical load case is longitudinal wind loading, combined with any eccentric vertical loading from erection equipment, and the effect of any deflection of the tower to provide the free cable tower saddle offset.

Concrete towers can be constructed either by continuous slip forming or in a series of lifts, using jump shuttering. Slip forming is the quicker method and will give the shortest construction programme, but requires a continuous supply of concrete, with certainty of delivery in all weather conditions, and is therefore only appropriate for construction of towers that are either on land or close enough to a shore line to be accessible by means of a temporary bridge or causeway.

Temporary footbridge

Irrespective of whether the main cables are constructed by spinning or by the erection of PPWSs, a working platform is required to give access to the complete length of the cable, and subsequently for compaction, cable clamp and hanger erection, deck erection, cable wrapping and painting. This platform is provided by a temporary footbridge positioned below the free cable profile to enable work at a convenient height, while providing enough clearance for the later operation of the compacting and wrapping machines, a typical arrangement being shown in Figure 15.35. The footbridge consists of a number of spiral strands or wire ropes with a floor of galvanised wire mesh, with transverse timber treads to provide a secure foothold, particularly on the steeper areas near the towers and U-frames,



Figure 15.35 Temporary footbridge

fixed to the strands at intervals to maintain the cross-section, and hand strands that support wire mesh sides to provide a safe working environment. The footbridges are stabilised by linking temporary cross-bridges, which also provide crossing points for personnel. Additional stiffness is provided by storm strands positioned in an inverted catenary below the footbridge and linked to it at intervals by vertical rope ties. Support for an overhead endless tramway system to drive the spinning wheel or haul out prefabricated strands is provided by two or more strands positioned above the outer floor strands and linked to them by wire rope ties. As the footbridge is required as a working platform after deck erection, when the main cables have deflected to their full dead load profile, the supporting strands must incorporate adjustment to permit this.

Cable construction

Wire production

Wire for parallel wire spun cables is manufactured and transported to site in coils weighing between 250kg and 1200kg, with an internal diameter of about 1500mm, ensuring that, when uncoiled, the wire is sufficiently straight for subsequent cable spinning operations. An extended production period is needed to produce the large quantity of wire required for cable spinning before commencement of work at site, and a storage facility with a controlled environment for the large quantities of wire coils produced is therefore required.

Conventional aerial spinning

Parallel wire cables are divided into a number of strands for construction by spinning, with each strand containing between 350 and 550 wires, and the cable is assembled by the sequential spinning of these strands.

In each cable, the first strand, positioned at the bottom of the cable hexagon, is spun alone and adjusted to the required free cable sag, using the results of an absolute level survey between the towers and anchorages. All subsequent strands are spun in pairs and positioned by a comparative survey relative to this bottom strand. Developed by Roebling in the nineteenth century, and progressively improved since then, in situ spinning essentially consists of pulling loops of wire wrapped around a 'spinning wheel' (Figure 15.36) across the temporary footbridge using an aerial ropeway (tramway). Two wheels are attached to the tramway rope and positioned so that, as a loaded spinning wheel makes an outward trip across the footbridge, the wheel from the previous trip returns empty to enable the next trip to be commenced with the minimum interruption to operations. Each spinning wheel is grooved to accommodate either two or four loops of wire, with two loops being appropriate for small to medium-sized cables, and four for larger cables. Cable spinning with two loops of wire is described next, but the same principles apply for four-loop spinning for larger cables.

The coils produced by the wire manufacturer contain too short a length of wire for them to be used directly in the

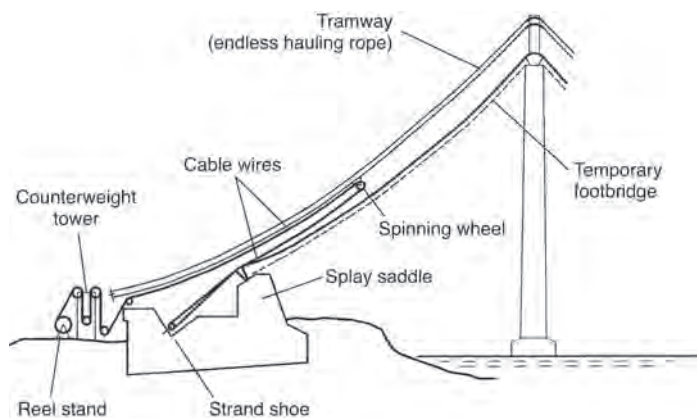


Figure 15.36 Cable spinning

spinning operation; before the wire can be used it is therefore wound onto large-diameter reels containing sufficient wire to enable the spinning wheel to make a reasonable number of trips across the bridge before the need for refilling. These reels are located in unreeling machines that can pay out wire to match the speed of the spinning wheel, and wire is led from them into a 'counterweight' tower, to pass around sheaves that form a loop of wire supporting a free-hanging counterweight, which maintains a constant back-tension and can absorb any variations between the unreeler and tramway speeds. At the start of spinning each strand, the ends of the wires from two reels are led through the counterweight tower, through a deflection sheave near to the appropriate strand shoe location, up the anchorage chamber and the splay saddle to a temporary fastening point on the footbridge. Loops of wire are then placed in the strand shoe grooves, with further loops around the spinning wheel. The tramway is then operated to pull these loops of wire across the footbridge; as the wheel (Figure 15.37) travels across the bridge, the upper (live) wires are released from the unreeler and move at twice the tramway speed, whereas the lower (dead) wires wrapped round the strand shoe do not move. The tramway is normally operated at a speed of up to 6 m/s in the spans, but more slowly as the spinning wheel moves past the tower and splay saddles.

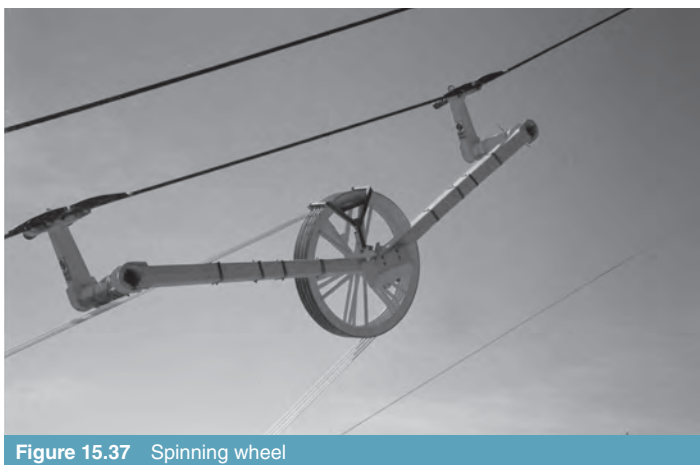


Figure 15.37 Spinning wheel

Temporary sheaves at intervals across the footbridge support the live wires, enabling them to be pulled out without undue friction, with the dead wires being allowed to fall onto the footbridge floor or, as the spinning wheel passes each saddle, placed in their final position in the appropriate groove. When the spinning wheel reaches the far anchor chamber, its loops of wire are removed and placed around the strand shoe at that end and, at the same time, the empty spinning wheel, which has arrived at the spinning anchorage, has new loops of wire formed around it ready for the next trip. As each trip proceeds, the sag of the spun wires is adjusted to conform to a guide wire set slightly above the final required position of the cable. Once adjustment of all the wires of the trip has been completed, the tramway is again operated to pull a new set of wire loops across the bridge, with spinning continuing in this way until all the wires of a strand have been positioned and adjusted to a common sag. The usual practice is to spin two strands at a time, with each spinning wheel placing wires into one strand. When the spinning of each pair of strands has been completed, the wires are freed of any temporary lashings and shaken out to allow them to hang in free catenary, following which any wires falling outside a specified sag tolerance are adjusted by cutting out a section and re-splicing any low wires, or by splicing in an extra length for high wires. The completed strands are then compacted into a circular shape with a small hand press and secured by thin metal straps placed at about 3 m intervals. The final adjustment of the strands to the required free cable sag is made by carrying out a night-time survey (in stable temperature conditions) to determine its as-spun position, followed by adjustment to bring it to the correct sag for the prevailing temperature and saddle positions. The strand movements necessary to produce the required sag are produced by jacking at the strand shoes at each anchorage and, for the main span, moving the strand through the tower saddles with pulling equipment.

Controlled tension spinning

This is a development of the original spinning method in which the wires are spun at a higher tension, and the adjustment of individual wires is eliminated, simplifying the work on the footbridge and at the saddles, with a substantial reduction in labour requirements. The wire is pre-reeled in the same way as for conventional spinning, and the unreeling and counterweight equipment is similar to that for conventional spinning, except that the counterweight is arranged to produce a spinning tension in the range 60% to 80% of the wire's free-hanging tension. The tramway spinning wheels have a variable angle setting, and the outward-going wheel is tilted so that it places the dead wires directly into their final position in cable formers fixed to the footbridge, with the live wires supported by temporary sheaves. Handling and placing the wires in the saddles is similar to that for conventional spinning; on arrival of the spinning wheel at the far anchorage, loops of wire are pulled out and placed around the strand shoe, with the wires remaining on the wheel. The tilt of the spinning wheel is reversed for the return trip, so that as the wheel moves back across the foot-

bridge the live wires placed on the outward trip are lifted from their sheaves and placed into the cable formers. As the spun wires are at between 60% and 80% of their free-hanging tension, the cable formers transfer between 40% and 20% of the weight of the spun wires to the footbridge. As the footbridge is a very flexible structure, this can cause significant deflections and hence sag variation in the spun wires, and a means of compensating for this effect is therefore required. This is only required for the first few strands, as the addition of strands rapidly increases the effective stiffness of the footbridge. Once spinning of a strand has been completed, it is banded and adjusted as for conventional spinning. This method was first used for the cables of the Shimotsui Seto Bridge in 1985–86 and has subsequently been used for other major bridge cables, including the Second Bosphorus (1987) and the Storebælt East (1996) bridges.

Prefabricated parallel wire strands

This method represents a further step in the reduction of cable construction work at the bridge site, achieved by completely prefabricating the cable strands, so that cable construction is reduced to simply unreeling the strands and pulling them across the footbridge using an overhead tramway system similar to that for spinning. This produces a further reduction in site labour requirements and, more importantly, reduced sensitivity to adverse weather conditions. To support the strands as they are pulled out across the footbridge, closely spaced support rollers are required to minimise frictional resistance and prevent abrasion damage. Once pulling out has been completed, the parts of the strand that will be placed in the saddles are reshaped from their hexagonal manufactured shape into a rectangular form before lifting into position. Survey and adjustment of the strands to their final level is carried out using methods and equipment similar to those for spun cable strands.

Compaction

After erection and adjustment of all strands has been completed, compaction is carried out using a machine consisting of a hexagonal steel frame (Figure 15.38) equipped with six hydraulic rams fitted with circular profile steel shoes that are overlapped and interlocked so that operation of the rams squeezes the cable wires into a circular shape. The hydraulic controls are arranged so that the vertical and side rams can be operated either simultaneously or separately to provide control over the compacted shape of the cable. The cable is squeezed at close (around 750–1000 mm) intervals up to as close as practicable to the saddles; as soon as each squeeze has been made, and before the ram pressure is released, the compacted shape of the cable is retained by tensioning temporary strapping (of either stainless or galvanised steel) around the cable, with additional compaction squeezes and strapping at the positions where cable clamps will subsequently be fitted. The capacity

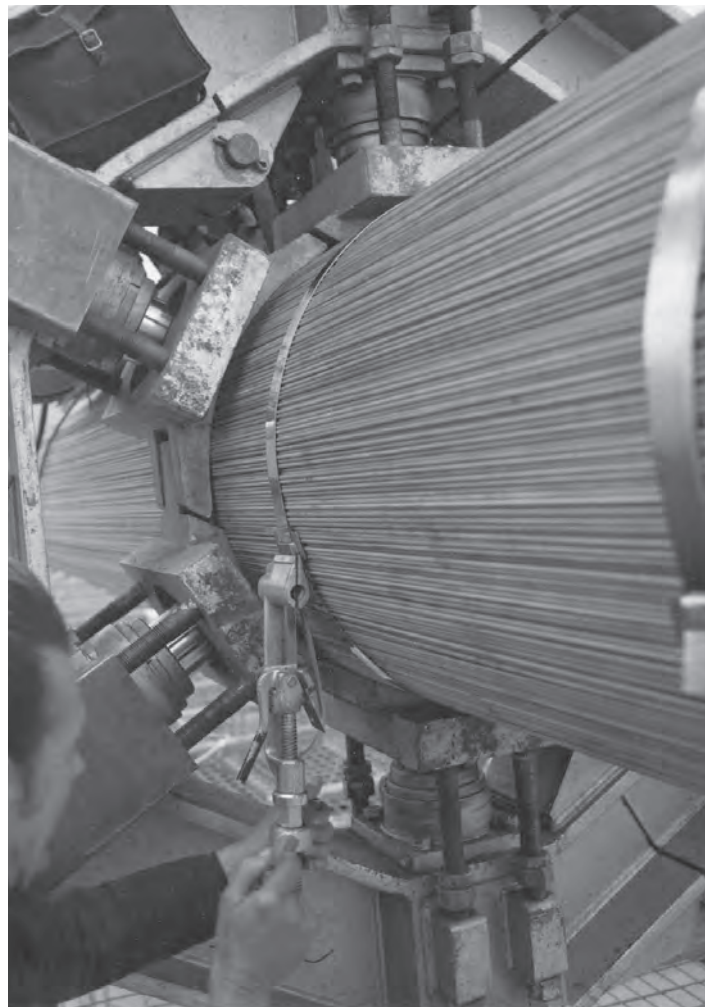


Figure 15.38 Cable compactor

of the hydraulic rams required to achieve the necessary voids ratio is a function of the size of the cable and the quality of the cable construction (variation in individual wire sags and the frequency of wire crosses) and, as it is not amenable to theoretical analysis, must be determined from previous experience.

Cable clamp and hanger erection

Once compaction has been completed, cable clamps and hangers can be erected, starting either at the centre of the main span or from the towers and working progressively outwards from there. After the clamps have been accurately positioned, their bolts are tensioned, preferably by direct application using hydraulically powered bolt tensioning equipment, and the hangers erected, either by carrying out from the towers with a work car, or by direct lifting into position from a supply barge. As previously noted, a series of bolt tensioning operations is subsequently required, with the last of these being made immediately prior to footbridge removal.

Erection of suspended deck

Prefabrication

To minimise the amount of work at the bridge site, the deck structure is usually divided into a series of prefabricated sections, the length of which is the hanger spacing or a multiple of this. The only site work then required is the lifting of these sections, followed by the bolting or welding of the transverse joints between them. Deck sections can be erected much faster than they can be fabricated and assembled, and most, if not all, of the sections must therefore be completed and ready for lifting before erection is commenced, requiring the provision of a large temporary storage area with access to the bridge site. During assembly of the deck sections, the ends of each abutting section must be accurately matched, with the sections set at their required relative vertical alignment so that the transverse joints fit correctly after lifting. Temporary deck connectors (Figure 15.39) are fitted at the transverse joints to ensure that correct alignment is maintained after erection and to resist forces from wind loading; as the transverse joints will initially be open at the bottom of the section, these are only positioned on the deck and upper web plate of the sections.

Deck erection sequence

The options (Figure 15.40) for the erection sequence of the suspended structure are as follows.

- Commence erection at the centre of the main span, working outwards until the towers are reached, with the side spans either being erected concurrently with the main span or, if tower bending strength limitations permit, erected after completion of the main span. In the early stages of this sequence, the partial loading of the cable forces the main-span deck into a sagging profile, and it is not possible to close and connect the bottom flange joints, so that the temporary connections form the only link between sections. This sequence has the advantage that, as erection proceeds towards the towers, the deck profile quite rapidly converges towards the final dead load profile, enabling the connection and bolting or welding of the transverse joints to be begun soon after starting erection. However, it has the disadvantage that the part-erected deck is accessible only by ascending the tower and walking down the footbridge to midspan.
- Commence erection at the towers, working outwards until the centre of the main span is reached, with the side spans erected either working from the towers or the anchorages, again either concurrently with the main span or subsequently. This sequence has the advantage of direct access along the already erected deck to the erection fronts. However, the effect of the part-loading of the cable again causes a pronounced sagging curvature

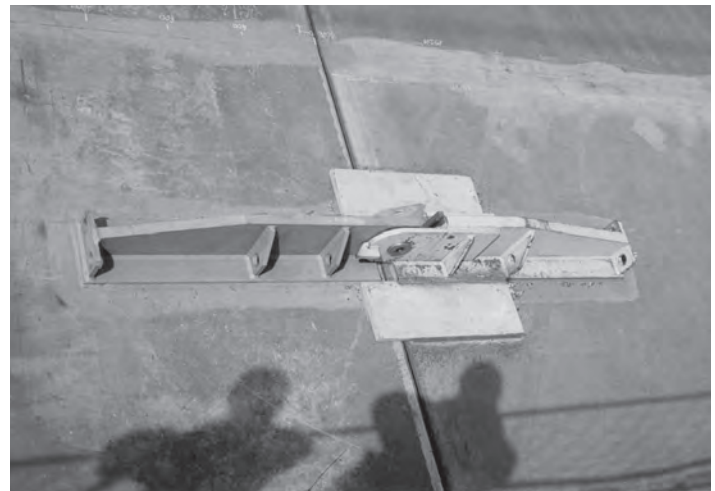


Figure 15.39 Temporary deck connection

of the deck with open bottom joints and this persists until a relatively large proportion of the span has been erected, delaying the start of bolting or welding of the joints, unless special measures are taken, such as the temporary adjustment of hanger lengths.

Deck erection equipment

The lifting equipment used for erection is influenced by both the sequence and the type and weight of the deck structure. There are three basic methods.

- Erection using lifting gantries (Figure 15.41) supported by the main cables, which are progressively moved along them as erection proceeds. The lifting equipment can be either self-contained strand jacks or rope tackles, with the operating winches positioned at the tower. Gantry lifting is suitable for the erection of both box

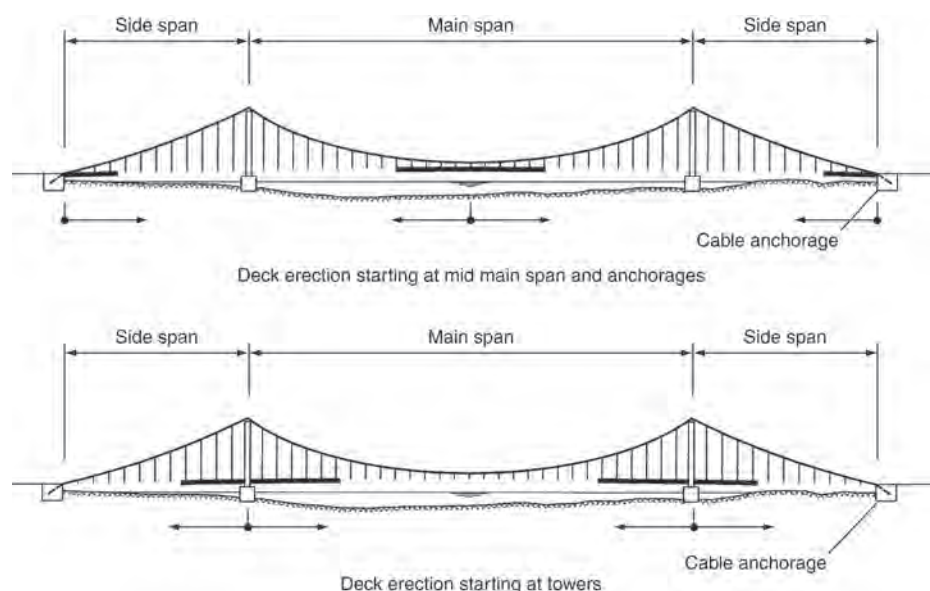


Figure 15.40 Deck erection sequence

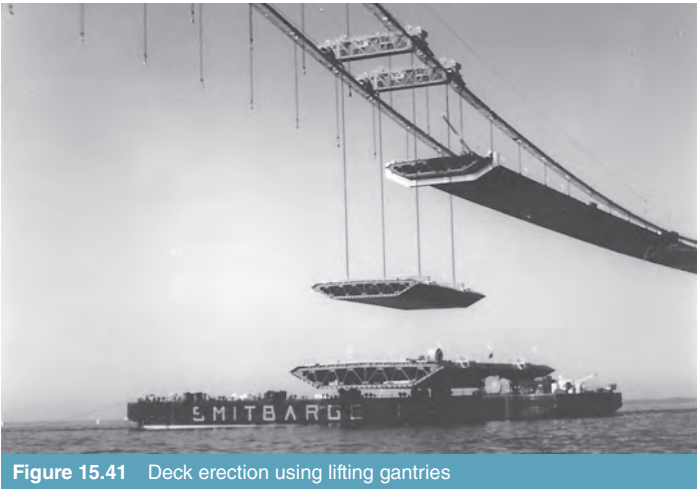


Figure 15.41 Deck erection using lifting gantries

girder deck units and preassembled sections of truss-stiffened decks.

- Lifting using cranes working on the already erected deck. This method is suitable only for an erection sequence starting at the towers and for truss-stiffened decks erected in relatively small sub-assemblies.
- Lifting using floating cranes. This method can be used for either basic erection sequence.

Cable wrapping

The circumferential wrapping of the cable (Figure 15.42) is carried out using a machine consisting of a fixed structure resting on the cable, onto which is mounted a rotating ‘flyer’ assembly with two, three or four reels containing wrapping wire. The rotation of the flyer assembly, synchronised with movement of the machine along the cable axis, simultaneously lays two, three or four turns of wrapping wire at a tension of around 1.5 kN over sufficient paste applied to completely fill the interstices between the wrapping wire and the cable wires. Spring-loaded ‘fingers’ on the flyer press the wires into tight contact with the previously laid turns and assist in controlling movement of the machine along the cable. The best quality ‘push’ wrapping is obtained when operating the machine upslope, working away from the completed wrapping, but a short length of cable adjacent to the upper cable clamp corresponding to the length of the machine cannot be wrapped in this way, and must be wrapped with the machine reversed and working over the completed wrapping.

Alternative bridge configurations

Although most suspension bridges are constructed as three-span structures with earth-anchored cables positioned at the deck edges, many variations from this layout are possible.



Figure 15.42 Wrapping machine

Bridges with more than one main span

The most important of these is the use of more than one main span where the span to be bridged would require an excessively long main-span length. The Taizhou bridge, with two 1080 m long main spans, was the first long-span bridge to be constructed using this arrangement. The structural behaviour of a three-tower bridge differs from that of the conventional layout, as the central tower does not have the longitudinal restraint effect of an adjacent side span, and a conventionally flexible tower design would therefore produce excessive deflections from loading in one main span (Figure 15.43).

However, if this tower were to be made longitudinally stiff by (say) an A-frame configuration

- the out-of-balance cable at the centre tower would be impossible to resist by saddle friction and cable slippage could occur
- the tower and its foundations would have to be designed to resist very large longitudinal moments.



Figure 15.43 Two main span bridge with flexible centre tower

The central tower design must therefore compromise; the stiffness must be large enough to limit deck deflections but not so large that unacceptably large out-of-balance cable forces are generated at the tower saddle. This was achieved for the Taizhou Bridge by the use of a semi-flexible tower in the form of an inverted Y-shape; a similar tower arrangement has been chosen for the proposed Chacao Bridge.

Self-anchored bridges

In self-anchored bridges, rather than the main cable forces being resisted by earth anchorages, they are transferred into and resisted by the deck structure, producing a balanced structure in which only transient traffic and wind loads are transferred to the foundations. Self-anchored bridges suffer from the disadvantage that the deck structure must be designed to carry a large compressive longitudinal load and must also be completely erected and temporarily supported before cable construction can commence, a reverse of the normal sequence. This requirement for temporary deck supports limits the application of these structures to relatively short spans and where a potentially lengthy obstruction of the main span is acceptable. The application of self-anchored structures is therefore limited to situations where a cable-stayed bridge is generally a more suitable solution. The most recently constructed bridge of this type is the new San Francisco–Oakland Bay bridge, which, in addition to being self-anchored, has an unusual arrangement of a single leg tower flanked by a 385 m main span and a 180 m side span. A further unusual feature of this bridge is the use of a twin box girder deck and the inclination of the main cable and hanger planes.

Hybrid part cable-stayed bridges

The support of the deck nearest the towers by a theoretically more efficient cable-stay system, with only the central part of the span supported by the main suspension cables, can produce, relative to a pure suspension cable system

- a saving in suspended cable material quantities
- smaller, more economical, anchorages
- higher, more expensive towers
- an increase in main-span vertical stiffness, making the bridge more suitable for rail traffic
- a potential saving in construction time.

As the stays introduce compressive forces towards the towers, the suspended deck must be continuous with movement joints at the anchorage span ends only and deck erection must necessarily commence at the towers, working outwards. Although the first major suspension bridge (Brooklyn) had a combination of main suspension cables and diagonal stays, few hybrid bridges have subsequently been constructed, the most recent example being the 1408 m main-span Third Bosphorus Bridge, completed in 2016.

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Chapter 16

Movable bridges

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This chapter is an elementary introduction to movable bridge engineering. Movable bridges are classified and various types are described and illustrated with examples built in the UK, Europe and America. Span drive and stabilising machinery are treated and the interdependency between the superstructure, mechanical and hydraulic machinery and electrical controls is emphasised. A movable bridge is a machine and, as such, dynamic effects should be considered. Major design issues, including safety and redundancy, are discussed, as well as design specifications and future trends in the architecture and engineering of movable bridges.

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Introduction

Movable bridges are essential infrastructure in the highway, railway and waterway transportation systems of many countries. A bridge is termed movable if it has one or more spans that can be quickly moved by machinery permanently mounted on the bridge. Movable bridges are built where an acceptable vertical profile for a fixed railway or roadway crossing of a legally navigable waterway is not feasible. An acceptable profile for a fixed bridge provides the required clearance for navigation, has permissible vertical grades for vehicular and railway traffic and minimises the adverse impact on present and future use of land contiguous to the bridge. Aesthetic and environmental considerations may also influence the desirability of a movable span. Because of the higher operating and maintenance costs of movable bridges compared with fixed bridges, and the inconvenience of bridge openings to the travelling public, bridge owners usually prefer fixed to movable bridges for new crossings and replacements. However, for crossings at which few bridge openings per day would be required to permit passage of vessels, overall long-term economics may favour a movable bridge. It all depends on the relative amounts of vehicle and waterway traffic and the required vertical clearance for shipping in the channel.

Sometimes, over-water bridges are built with a span that may be temporarily removed and then replaced on infrequent occasions, usually by floating cranes, so as to increase navigation channel clearance. Such bridges are called removable, as distinct from movable, and are not treated herein.

Worldwide, there are thousands of movable bridges of many types, with spans ranging from a few metres to longer than 300 m. Some are more than a century old, with most in the 50–90 year range. The actual quantity is unknown because few

countries have a national inventory. In the USA, there are about 3000 active movable bridges, of which about 2000 are highway bridges and 1000 support railway tracks. British Waterways have 400 movable bridges under their purview and the canals of Belgium are crossed by 200. Finland has about 40 movable bridges and about 25 operate in Sweden. There are hundreds more in Europe, but data are unavailable.

New movable bridge construction is under way in Australia, Canada, Europe, the UK and the USA, and in developing industrial regions elsewhere. One reason is economic globalisation, which has resulted in a vast increase in ocean transport and concomitant increase in vessel size. The first generation of container ships built after 1960 were less than 200 m long and 30 m wide, with a capacity of 1000 units. The fifth generation, built after 1997, are 360 m long and 43 m wide, with a 7000 unit capacity. The larger traffic volume and increased vessel size have led to the construction of larger movable bridges.

Although movable spans were built for military bridges since ancient times, it was the advent of canal construction in Europe, Great Britain and North America that made more movable bridges necessary. Many fixed masonry arch and timber bridges were built across those waterways, but where it was impractical to build approaches to crossings having acceptable vertical grades, movable bridges were constructed. Because most movable bridges were manually operated, ingenious mechanisms were devised for gaining mechanical advantage (Whitney, 2003). By 1800, the principal types of movable bridges had been developed, although in rudimentary form (Hovey, 1926).

The railway boom of the nineteenth century provided another impetus for building movable bridges and introduced steam power for operating them. These railroad movable bridges, mostly of the swing type, usually had to be replaced

every few decades because of the rapid increases in locomotive and railway carriage weights. In the USA, the trusses of early swing bridges had timber compression members and wrought iron tension members. Timber was replaced by cast iron and wrought iron as the metal industry developed, and by 1865 a railroad swing bridge with a draw 300 ft (91.4 m) long had been erected across the Mississippi River at Clinton, Iowa (Birnstiel, 2008a). In the next section, the major types of movable bridges are briefly described.

Types of movable bridge

General

The motions of all movable bridge spans are a combination of rotation and translation; the differences between types are due to the axes selected for these displacements. In almost all existing movable bridges, the spanning element (girder, truss, etc.) is moved as a rigid body to create a passage for vessels. However, in some movables the shape of the spanning structure is changed (usually by means of fluid power devices) to create the required waterway clearance. In terms of primary displacement and axes of displacement, movable spans are usually categorised as given in Table 16.1.

The displacements mentioned in Table 16.1 are termed primary because there are other, secondary, motions associated with some types of movable bridge that are necessary for securing and releasing the movable span. Most of the movable bridge types listed in the table have subtypes, some of which are described subsequently. The most common categories of movable bridges are shown in Figure 16.1. The kinematics of some types of movable bridge are discussed by Wallner and Pircher (2007).

Figure 16.1(a) depicts a simple trunnion bascule bridge with a counterweight (CWT) fastened to the bascule girders. The trunnions are usually fixed to the bascule leaf and rotate in bearings mounted on the bascule pier. However, bascules

have been built with trunnions that do not rotate, and the leaf rotates about them in bearings that are part of the bascule girder (known as a trunnion girder design). In either case, the leaf rotates about a fixed horizontal axis, similar to a see-saw. A counterweight may fully or partially balance the leaf structure. At some bridges, the counterweight is hung from the girder by tension members (links). There are many subtypes of trunnion bascule, further distinguished by the location of the counterweight (if any), trunnion support or machinery, as will be described later.

Rotation about a horizontal axis that simultaneously translates is the defining feature of the rolling bascule (often called a rolling lift). The most common rolling bascules are those based on a series of patents granted to William and Albert Scherzer of Chicago, Illinois, starting in 1893. Almost all rolling bascules are nominally balanced by counterweights fixed to the bascule girder. The counterweights may be located above or below the deck, or outboard of the moving span. Figure 16.1(b) depicts a single-leaf rolling bascule bridge with the counterweight fastened to the bascule girder below the deck.

Most early twentieth century movable bridges were of the swing type, with rotation about a fixed vertical axis, as shown in Figure 16.1(c). If L_1 equals L_2 , the swing span (draw) is said to be symmetrical (common for most swing spans), otherwise it is known as a bobtail swing span. Some bridges have been constructed with $L_2=0$ at narrow waterways. Swing bridges have subtypes based on the type of rotational bearing at the pivot pier.

Figure 16.1(d) depicts a span that moves vertically along a fixed vertical axis in order to obtain additional vertical clearance for navigation. It is termed a vertical lift span. There are many subtypes, distinguished mainly by the location of the span drive machinery. The modern vertical lift bridge design is credited to JAL Waddell, who designed the South Halsted Street Bridge across the Chicago River, which was completed in 1895 along the lines of his USA patent.

Retractable bridges translate horizontally, usually along a straight path, which may be normal to, or at an angle to, the channel, as shown in Figure 16.2(a). There is a retractile bridge with a curved track in Copenhagen. If the translation is at an angle to the channel (approximately 45°), as in the figure, the movable span does not have to be lifted or depressed in order to clear a path for motion. The Borden Avenue Bridge over Dutch Kills in New York City was built according to this concept and has been in operation for over 100 years.

The horizontal angle between the longitudinal axes of the roadway and the movable span may be zero, usually for pedestrian or other lightweight traffic. However, provision has to be made for traffic to move from the street level to the bridge deck level. An example of an in-line retractile bridge of later vintage is the Pont

Type of displacement	Category
Rotation about a fixed horizontal axis parallel to the channel	Trunnion bascule and cradle- or roller-bearing bascule
Rotation about a horizontal axis parallel to the channel that simultaneously translates normally (at a right angle to it)	Rolling bascule
Rotation about a fixed vertical axis	Swing
Translation along a fixed vertical axis	Vertical lift
Translation along a fixed horizontal axis	Retractable and transporter
Rotation about a longitudinal axis transverse to the channel, which may simultaneously translate normally	Gyratory
Rotation about a number of horizontal axes parallel to the channel	Folding or coiling

Table 16.1 Types of movable bridge

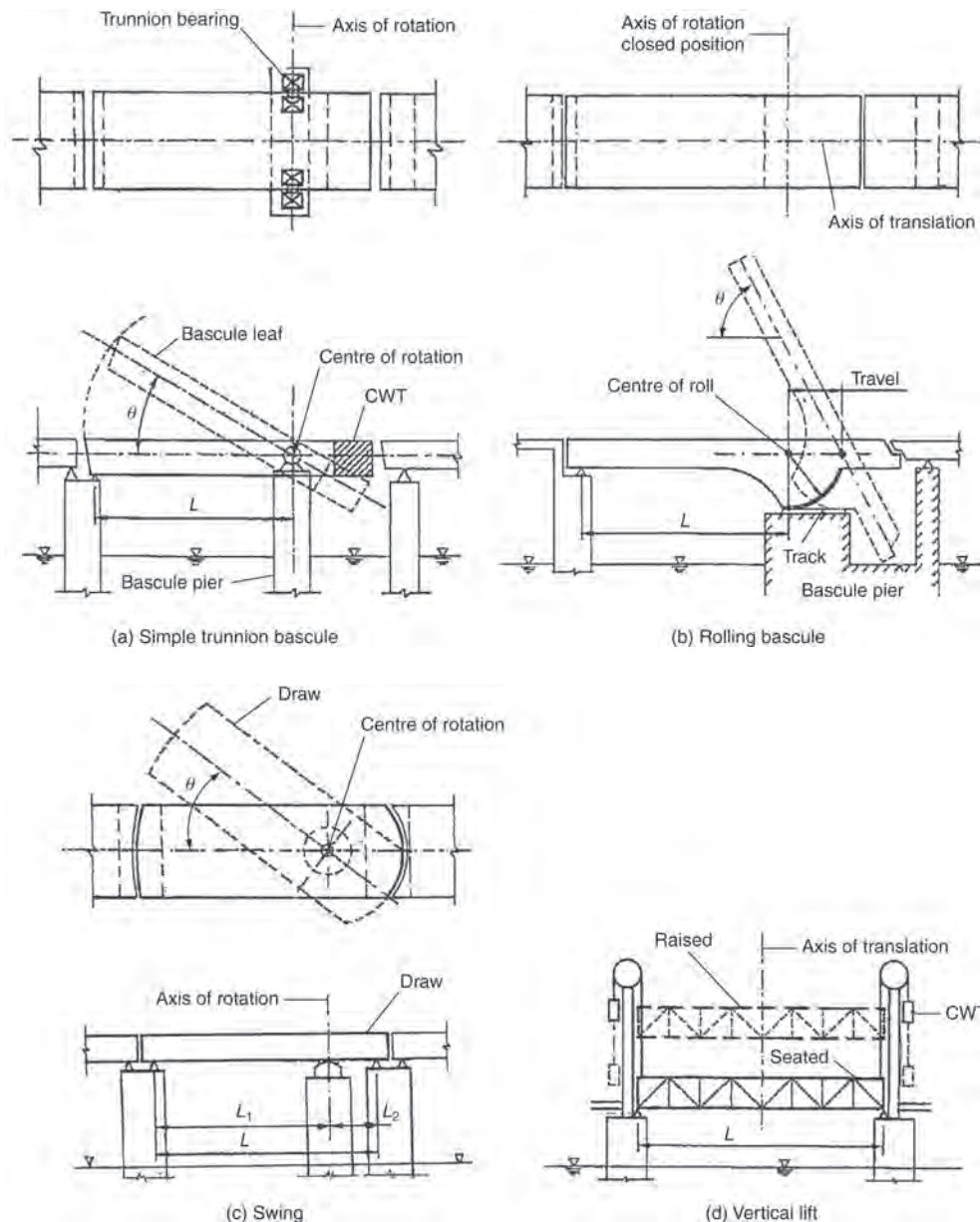


Figure 16.1 Common forms of movable bridge (CWT, counterweight)

des Belges between Trouville and Deauville, France, which has ramps at the closed position.

If the movable span does not roll on rails at ground level, as depicted, but rather rolls on rails mounted on an overhead structure that completely spans the channel, it is called a transporter bridge. The retractile platform is hung from a carriage that runs along the bottom of the overhead spanning structure.

A concept for a gyratory bridge, based on a USA patent granted to E Swensson in 1909, is illustrated in Figure 16.2(b). It is a version of a trunnion bridge, but the span rotates about a longitudinal axis instead of an axis transverse to the channel.

Only three bridges of the gyratory type exist (as of 2019).

Figure 16.2(c) depicts a concept for a folding bridge, a special case that is sometimes termed coiling. Rigid deck panels fold about a number of horizontal axes, only one of which is at a fixed location. The other folding axes translate, of course. Folding bridges have been built, in which all of the transverse folding axes translate during motion. This is an example of a movable bridge in which the spanning structure shape is changed to provide navigational clearance.

The basic primary motions and types of movable bridges have been described. The subtypes and pertinent features will be discussed for each type under separate headings.

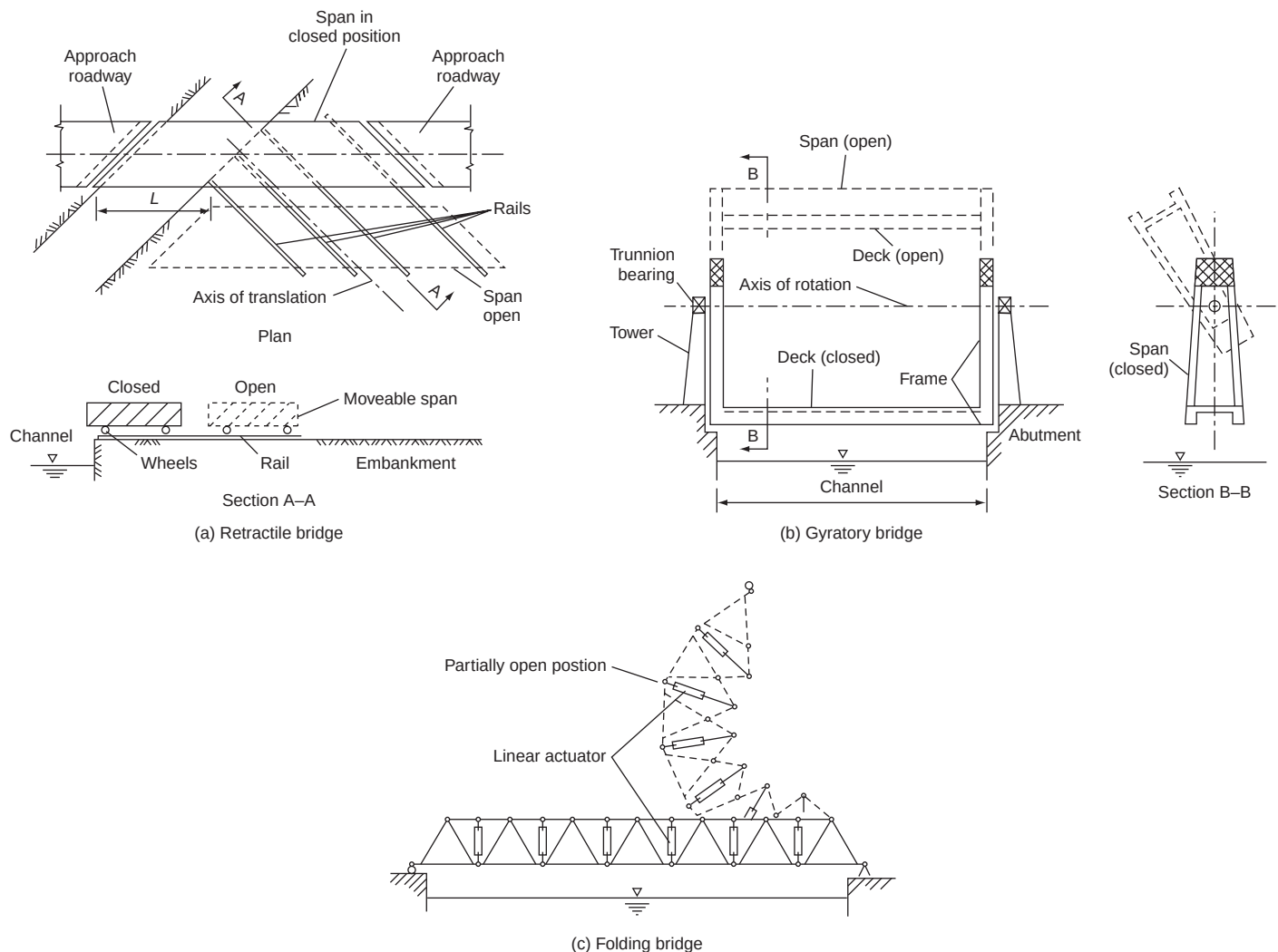


Figure 16.2 Some unusual forms of movable bridge

Bascule bridges

Simple trunnion bascule

Simple trunnion bascules are an old type of bridge and are often selected where unlimited vertical clearance is required for navigation. They have been built in many configurations: deck, pony and through; single leaf and double leaf. Figure 16.1(a) depicts a single-leaf (single-flap) deck bascule. Usually, each leaf is nominally balanced by a counterweight, which is fixed to the bascule girder. Such bascules are called balanced (although they are not completely balanced, for operational reasons). Counterweights are installed to minimise the size of the power transmission system components needed to operate the bridge, and to provide a relative measure of safety in the event of a failure in the span drive system. Of course, the counterweight has a cost and requires space, and simple trunnion bascules have been constructed without, or with partial, counterweights. Imbalanced bascule bridges are receiving increased attention

because of new seismic design criteria. In some areas, bridges are being designed for 1000 year return period events, with horizontal accelerations approaching 0.5g. In such cases, unbalanced leaves have the advantage that their mass may be less than half of a balanced bascule design. Unbalanced trunnion bascules are usually powered by hydraulic cylinder drives.

Medium- and long-span simple trunnion deck bascules require bascule piers with pits if the roadway profile is close to the high-water level of the channel, in order that the counterweights remain dry during leaf rotation. It is desirable for the counterweights to remain dry so as to maintain the balance of the bridge at all angles of opening. When the counterweights are partially submerged, more power is required to operate the span. To avoid the need for large and deep pits, the counterweights may be located remote from the bascule girder, usually overhead.

In the USA, patents for bascules with remote counterweights were granted to Joseph B Strauss about the year 1910, and bridges with his specific counterweight arrangements are often

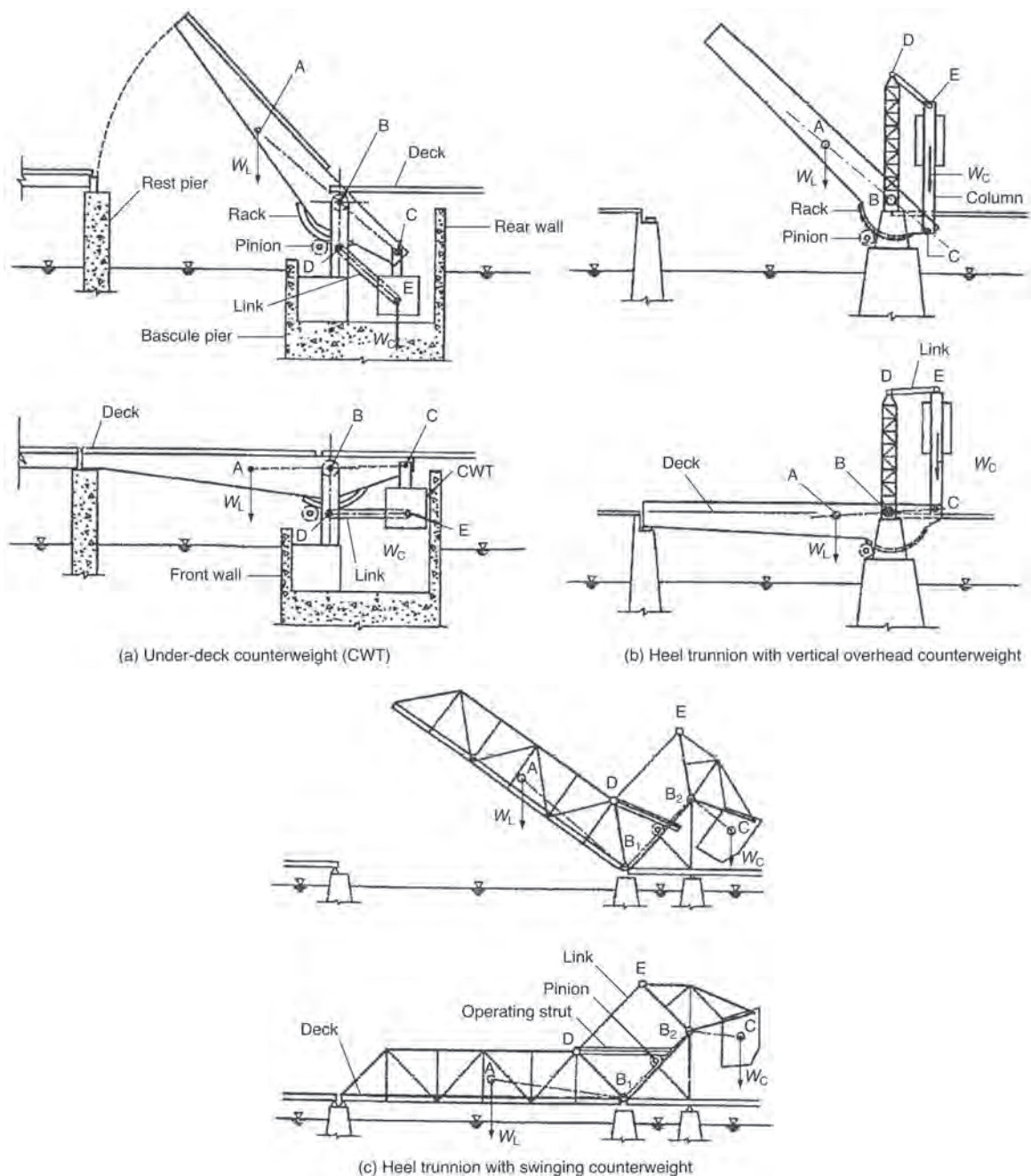


Figure 16.3 Strauss bascule bridges

called Strauss bascules. Three common versions are shown in Figure 16.3; the under-deck counterweight bascule (Figure 16.3(a)), the vertical overhead counterweight bascule (Figure 16.3(b)) and the heel trunnion with a rotating counterweight (Figure 16.3(c)). The distinguishing feature of all Strauss bascules is the parallelogram connecting the counterweight to the movable leaf. Motion of the leaf is by rotation of the leaf about a fixed horizontal axis. In some situations, it is not the depth of the required pit but the front-to-back dimension of the pit that needs to be minimised. In such cases, articulating the counter-

weight by suspending it from the leaf, as in Figure 16.3(a), may be advantageous. In this section, three of the articulated counterweight trunnion bascules promoted and engineered by Strauss will be described.

Strauss under-deck counterweight bascule

Figure 16.3(a) depicts a single-leaf Strauss bascule with an under-deck counterweight. The centre of gravity of the leaf is at A and it rotates about the trunnion B. The counterweight hangs

from trunnion C, and the direction of the vertical axis C–E of the counterweight is maintained by the link D–E between the counterweight and the trunnion tower. The geometrical figure B–C–D–E is a parallelogram. The need for this link has often been questioned. One argument for its need is that, at a small angle of opening, the friction in the counterweight trunnion bearings might not permit the hanger to rotate such that the axis C–E remains vertical. As the angle of opening increases, the moment applied to the bearing at C would increase; when it exceeded the bearing friction moment, the counterweight would swing free. This sudden motion could cause a dynamic load on the leaf, which might interfere with control of the moving leaf. Excessive friction in the counterweight trunnion bearings is often due to improper lubrication. The bearing friction induces a bending moment in the counterweight hanger, which produces repetitive bending stresses. Hangers on some bridges have failed in fatigue owing to this bending, resulting in collapse of the leaves.

An important feature of this design is that points A, B and C may be located along a straight line. The counterweight can be sized so that the imbalance value is zero, assuming a frictionless hinge at C. In practice, the hinge has friction and the counterweight is sized so that there is an upward reaction at the toe when the bridge is in the closed, seated position. Positioning the centre of gravity of the counterweight remote from the bascule girder usually reduces the required width and depth of pit from that needed if the concrete counterweight were directly attached to the bascule girder – an economic and architectural advantage.

Strauss vertical overhead counterweight bascule

Instead of hanging the counterweight from the bascule girder below the deck, the counterweight may be positioned overhead, as shown in Figure 16.3(b). This is advantageous at sites where the road profile is located close to the high-water level in the channel and the bascule pier cost must be minimised. Again, the balance principle requires that the line joining the centre of gravity of the leaf at A with the hinge C should intersect the trunnion axis at B. Line C–E will remain parallel to B–D if the line D–E is parallel to B–C. Lines B–D and C–E need not be vertical; they just need to be parallel.

Vertical overhead counterweight type Strauss bascules were built for minor roads in remote areas where appearance was not an important consideration. Operation is usually by rack and pinion, with drive machinery mounted on the pier, or, at sites subject to flooding, at the deck level, bracketed from the fixed structure.

Strauss heel trunnion bascule

A single-leaf Strauss heel trunnion bascule with overhead rotating counterweight frame (rocker frame) is shown in Figure 16.3(c). The geometrical figure B_1 –D–E– B_2 is a parallelogram and the centre of gravity of the counterweight at C is located so that the line B_2 –C is parallel to the line between the centre

of gravity of the leaf A and the heel trunnion B_1 . As a result, the ratio between the leaf dead load moment about B_1 and the counterweight moment about B_2 remains essentially constant during rotation of the leaf.

The leaf rotates about the heel trunnion B_1 in response to a force transmitted to the leaf by the operating strut (a rack is fastened to the strut), which is hinged at the top chord joint D and engages the output pinion of the span drive machinery, which is usually mounted on the fixed counterweight support frame. (Heel trunnion bridges were also built with the machinery mounted on the leaf.) The trunnions at B_1 , B_2 , D and E are heavily loaded during motion. The reaction at the heel trunnion B_1 may reverse direction during motion, depending on the proportions of the structure; this effect should be considered when evaluating the condition of the heel trunnion bearings.

Many bridges of this type were built for railroads in the USA and Europe. Some are huge machines with leaves exceeding 50 m in length. The longest span was 79.3 m, for the St. Charles Air Line Bridge in Chicago, erected in 1919. Because the channel was relocated later, the bridge was moved and, in the process, the span was shortened to 67.1 m.

In 1919, a double-track Strauss heel trunnion bascule with a leaf span of 50 m was built across the Thames River in Connecticut, USA. It was replaced in 2009. The history of continuous operation for 89 years over that important waterway and its replacement by a span drive vertical lift is described by Richter *et al.* (2010).

Bélicor bascule

Figure 16.4(a) depicts a trunnion bascule with a remote counterweight that is not connected to the movable leaf by a parallelogram. To maintain the closing moment about the trunnion approximately equal to the opening moment at all angles of opening, the counterweight rolls down a curved track of specific geometry. It is denoted the *Bélicor bascule* because a diagram of this bridge type appears in de Bélicor's book of 1729, with a mathematical derivation of the track geometry. Bélicor bascules were built in the nineteenth century for railroads crossing the Morris Canal and other narrow waterways in New Jersey. A historic highway Bélicor bascule now in use is shown in Figures 4.30 and 4.31 of Birnstiel *et al.* (2015).

Balance beam bascule

Another type of trunnion bascule with a remote counterweight is the balance beam or *Dutch-style* bascule bridge shown in Figure 16.4(b). This ancient concept is also known as a *lifting bridge*. To maintain the balance of the system at all angles of opening (neglecting friction), the geometric figure A–B–C–D must be a parallelogram. The structure above the bridge deck is subjected only to wind and dead loads; the weights of the balance beam, tower, counterweight and part of the leaf. In the closed position, the deck girder is simply supported at the abutments and supports the live load, as well as part of the self-weight. For stability in the double-leaf configuration,

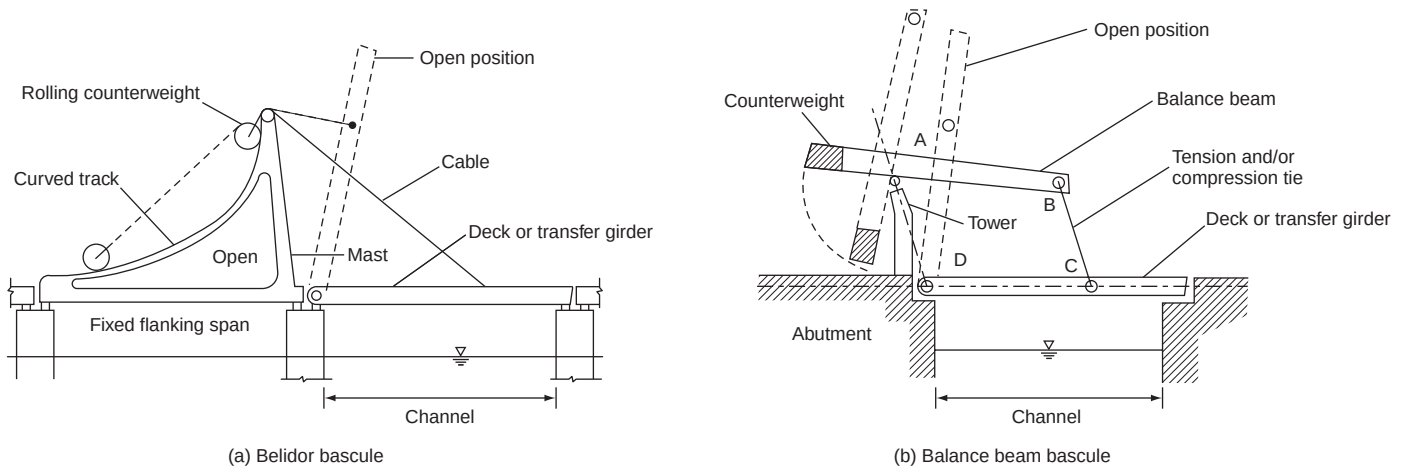


Figure 16.4 Other bascules with remote counterweights

additional supports are required. The classic double-leaf configuration is familiar from the famous paintings by van Gogh of *The Langlois Bridge at Arles, France*. The additional supports for each leaf in that bridge are inclined struts that bear on ledges of the abutment masonry when the leaf is lowered, and slide along vertical guides when it is raised. Another version has the additional support located forward of the heel trunnion. The girder acts as an anchored cantilever to equilibrate live load. Again, the superstructure mainly resists dead load and wind.

Recently, it has been proposed that the additional support

for the leaf be located atop the tower and that the deck girders be made continuous at midspan (for live load) by means of a mechanical joint (Saul and Humpf, 2007). In this scheme, the superstructure resists live load in addition to dead load.

Rolling bascule

In May 1893, William Scherzer applied for a USA patent for a rolling bascule bridge that became known as a rolling lift bridge – a term that is still used. The patent was granted post-

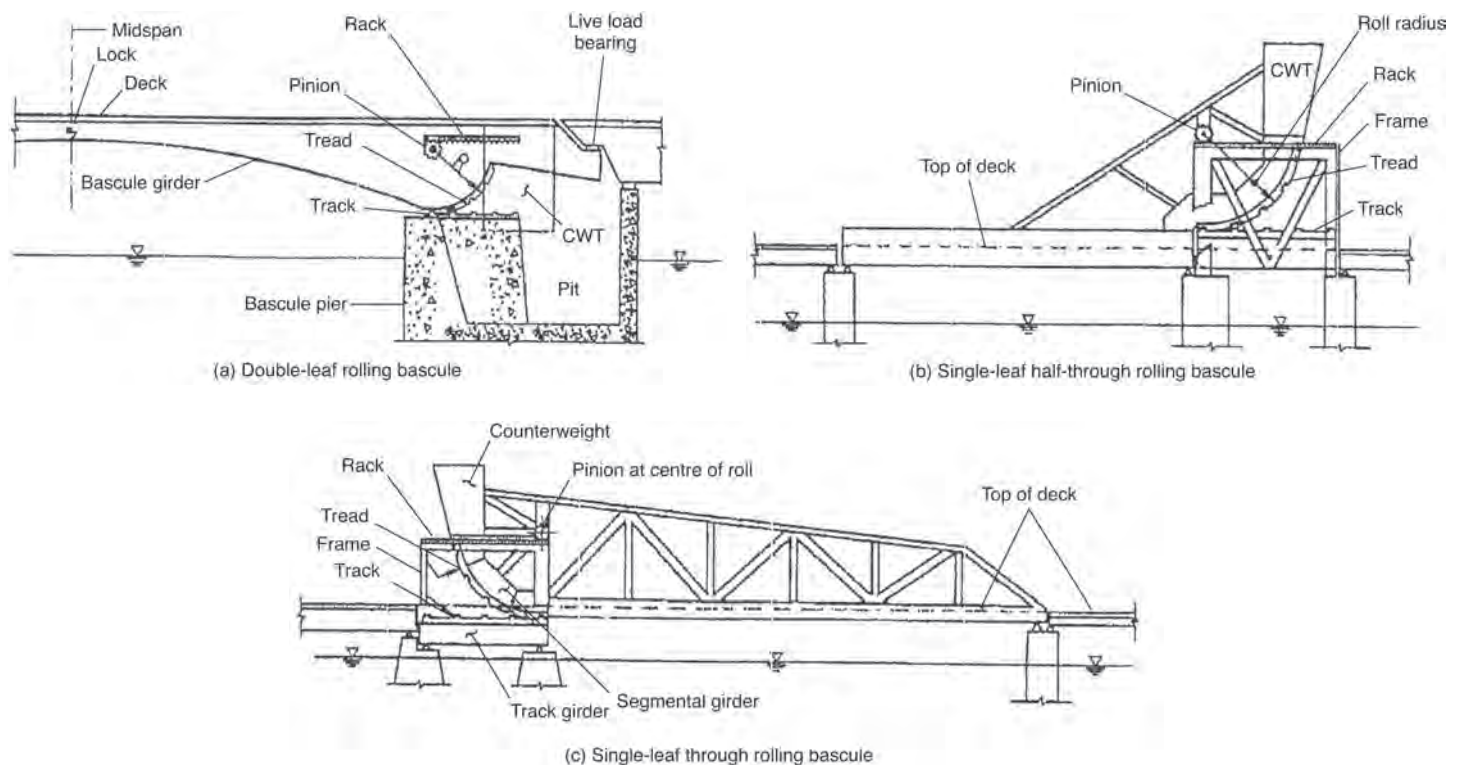


Figure 16.5 Rolling bascule bridges

humously to his brother, Albert, in December 1893. The first two rolling bridges designed by William Scherzer, but not built until after his death, are depicted in Figure 1.15 of Birnstiel *et al.* (2015). The Van Buren Street Bridge was his second design, but the first completed, in 1895. The leaf motion is due to rotation about a transverse horizontal axis, which simultaneously translates in the longitudinal direction. The movement is akin to that of a rocking chair. Three common types of rolling lift bascules are: the deck type, the half-through (pony) plate girder or truss and the through truss. They are illustrated in Figure 16.5.

Rolling bascule bridges are characterised by cylindrically curved parts of the bascule girders (or trusses) at the ends, usually located over the bascule piers, but sometimes on an adjacent fixed span. Because of their large size, these parts of the girders or trusses of the early Scherzer bridges were assembled from cast steel segments and the girders were called *segmental girders*, a term still in use. Each segmental girder may be viewed as a segment of a wheel. However, instead of rotating about an axle, these wheels are rigidly attached to the bascule leaf. As the ‘wheels’ roll rearwards or forwards on the tracks, the bascule leaf rotates open or closed. Slippage between the segmental girder treads and the tracks on which they roll is restricted by lugs or teeth on the treads that mechanically engage sockets in the track, or vice versa.

The rolling lift bridges shown in Figure 16.5 are all operated by pinions located at the centre of roll that engage racks. Rotation of the pinions pulls the leaf forwards or backwards. It is preferable that the rack be located above the pinion instead of below it, as this provides two advantages. The distance between the track and the pitch line of the rack is greater, thus increasing the mechanical advantage for operating the bridge. It also prevents the pinion teeth from bottoming out in the rack if there should be excessive wear or deterioration of the treads and tracks. Some rolling lift bascules were built for railroads with pinions located above the centre of roll. The racks of those bridges are not straight but are S-shaped in the elevation view; an example is the Amtrak bridge over the Hutchinson River in the Bronx, New York City.

Hydraulic cylinders are also used to operate rolling lift bridges. On some bridges they are located outboard of the segmental girder, in a horizontal position, with the piston rod end connected at the centre of roll, but the cylinder need not be horizontal nor connected at the centre of roll.

Roller- or cradle-bearing bascule

In three consecutive years, beginning in 1899, John Philo Cowing received US patents for a ‘bascule lift-bridge’. In the patent text, he called it ‘a revolving bascule bridge’. The type has also been known as a cradle-bearing bascule. His patents show double-leaf bascules, in which the roadway live load is to be supported by three-hinged arch action. The land ends of each leaf are two structural steel box rings, which rotate about a horizontal axis, similar to the axis of a simple trunnion bascule in Figure 16.1(a). One of the patents shows a physical

trunnion at the axis of rotation. This would lead to structural indeterminacy. The rings are in parallel vertical planes, and rest on rollers. The patent of 1899 shows rollers mounted in sleeve bearings in iron castings mounted on the masonry pier. In the 1900 patent, the rollers are separate elements, joined (chained) together by retaining rings. This construction is similar to that of current antifriction roller bearings commonly used in machinery, for example, automobile wheel bearings. The box ring is the upper race for the rollers and the base casting is the lower race. In the 1901 patent, the rollers are shown mounted in the box ring and bear on the base casting. None of these three schemes had provision for equalising the load on the rollers.

The 1899 patent also shows a single-leaf roller-bearing bascule with a girder span for railroad application. The architects WilkinsonEyre adopted this bridge type for two single-leaf bascule crossings of Bellmouth Passage in London (see Figure 16.6) and for the indicative (conceptual) design of the Johnson Street Bridge in Victoria, Canada. A double-leaf cradle-bearing bascule along the lines of a Cowing patent was opened to traffic in Cleveland in 1907 and decommissioned in 1957 (Birnstiel *et al.*, 2015).



Figure 16.6 Bellmouth Passage Road Bridge, London, UK

Piston-stayed bascule

For most movable bridges, the spanning structure is subject to a rigid body displacement in order to create a passage for ships. For example, the swing bridge superstructure is rotated about a vertical axis in order to clear a channel. The leaf of a simple trunnion bridge is rotated about a horizontal axis as a rigid body so as to remove it from obstructing the channel for tall vessels. In both cases, mechanical or hydraulic machinery is used to move the structure. In this process, the structure deforms slightly because it is elastic and the gravitational forces acting on it change during the movement.

The idea of changing the shape of the bridge to clear the channel by changing the length of the structural members using mechanical means goes back to China in about 500AD, where catenary suspension bridge cables were wound on capstans

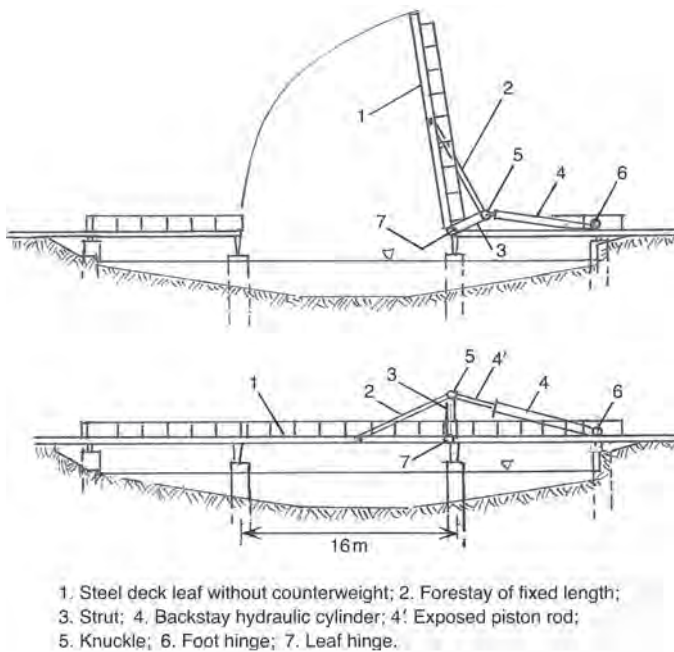


Figure 16.7 Piston-stayed bridge at Tervate, Belgium

at the land anchorages. The bridge deck could be lowered by unwinding the capstans, or, conversely, raised by tightening them (Needham *et al.*, 1971). A similar movable bridge was constructed in Germany in the twentieth century, except that hydraulic cylinders were used at the anchorages instead of capstans. The shape of the structure is changed to make more room for navigation.

Ney and Adriaenssens (2007) replaced two members of a bascule bridge truss with hydraulic cylinders at Tervate, Belgium, and called the unbalanced bascule a piston-stayed bascule. The cylinders serve two functions: they are structural members when the bridge is closed and are part of the machinery to open it. There is metal-to-metal contact between the piston and the cylinder head in the bridge-closed position so that the cylinder and the piston rod form one continuous structural tension member, while the cylinder is not pressurised. Figure 16.7 shows the bridge in the open and closed positions.

Chicago-type bascule

Completion of the Illinois and Michigan Canal in 1848 created a navigable waterway across the continental divide, from the St. Lawrence River through the Great Lakes, the Chicago River, the Illinois River and the Mississippi River to New Orleans. Shipping through the Chicago River and

the development of the railroads after 1850 created an economic empire centred at Chicago, particularly at the Chicago River main channel and its North and South Branches. This necessitated many bridges, which, because of the topography, had to be movable. Early bridges were mostly swing bridges with centre piers in the narrow waterways. Increasing vessel sizes made navigation more difficult and resulted in outages owing to ship collisions with draws. In 1893, a vessel collided with the Van Buren Street Bridge (a swing bridge). The City of Chicago decided to replace it with a Scherzer rolling bascule. William Scherzer designed it and the bridge was opened to traffic in 1895 (Birnstiel *et al.*, 2015). The City was not happy with the bridge because modifications had to be made early on, the rolling action caused foundation movements and there was the matter of royalty payments. John Ernst Ericson was the Chicago City Engineer at the time and he had the department study three forms of movable bridge: the folding type (Weed Street), Waddell's vertical lift (South Halsted Street) and Tower Bridge (London). This study is discussed by McBriarty (2013) and Phillips (2008). The study conclusion was that the best movable bridge type for future movable bridges to be designed and built by the City of Chicago was the fixed trunnion bascule, such as Tower Bridge, but with the trusses inverted so as to obtain the maximum vertical clearance between high water and the underside of the closed bascule leaves. The early architectural studies for Tower Bridge are described in Smith (2019). (Tower Bridge trusses are shown in Figure 1.12 of Birnstiel *et al.* (2015)). At the time of the study, the enormous Sanitary and Ship Canal project, an extension of the South Branch, was underway. The Drainage Commission also needed movable bridges and it selected the Scherzer rolling bascule type, even-

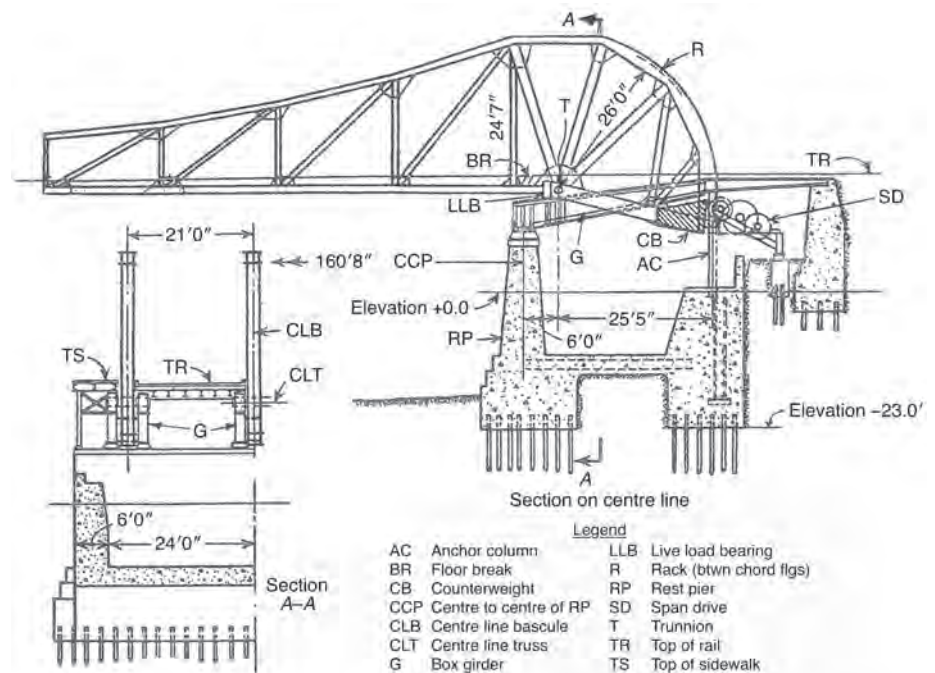


Figure 16.8 Cortland Street Bridge, Chicago, IL, USA

tually contracting for 17 bridges. The City, however, began designing its own trunnion bascules. There were a series of designs, now called 'generations'. The first bridge opened to traffic, although the third designed, was that at West Cortland Street (then called West Clybourn Place). It was opened to traffic in 1902. Because as-built drawings of the bridge were unavailable and the pits could not be inspected, the following description of the bridge is based on Becker (1944) and Holth (2019). The bridge is no longer operational.

Cortland Street Bridge, over the north branch of the Chicago River, was one of the first-generation City designs and was termed a 'fixed-centre, double-leaf counterbalanced bascule bridge'; in current terminology, a trunnion bascule bridge. As shown in Figure 16.8, each leaf comprised three trusses, spaced 21 ft (6.4 m) between centres. The spaces between trusses provided for a trolley track and one generous traffic lane for horse-drawn wagons or, later, motor vehicles. The trusses varied in depth from half-through at the middle of the bridge to through at the trunnions. Full depth at the trunnions enabled rigid horizontal X-bracing at the truss top chords and stiff portal (lateral) X-bracing at the trunnion panel points.

The floor breaks were located on the river side of the trunnions so that live load always tended to close the bridge. The tail ends of the trusses rotated downward into slots in the roadway on the shoreward side of the floor breaks. The trunnion bearings were mounted on box girders spanning the tail pit in the longitudinal direction, as shown in Figure 16.8. There are live load supports on the river side of the trunnion bearings (a matter of controversy at the time of construction). Truss bumpers at the rear end of the trusses were intended to stop overtravel as the bridge was closed and transfer the uplift force resulting from live load to the anchor columns.

The trusses were rotated by electrically driven machinery located under the approach roadway; pinions engaged curved racks, which were mounted between the top chord flanges through a maximum opening angle of $76^{\circ}58'$. The teeth of the cast steel rack had openings between them so as to minimise clogging of the rack by dirt or ice. The teeth of the operating pinion dislodged debris accumulations.

As shown in the figure, the bridge was counterweighted such that the centres of gravity of the movable leaves were slightly forward (to the water side) of the trunnions. For Cortland Street, the counterweight comprised cast iron blocks bolted to the trusses within their lateral envelopes. Because drawings for Cortland Street were unavailable to the author, Figure 16.8 is a composite developed from data for the West Division Street of the North Branch

crossing (the fourth 'Chicago' design) and photographs in Holth (2019). Tail pit construction did not follow the illustration on page 308 of Hill (1904).

The first road-railway bridge built of aluminium alloy, located in Sunderland, UK, was of the Chicago type, except that it did not have the overhead truss bracing. The Chicago engineering study led to the adoption of many trunnion bascule bridges but with different spanning structures.

Swing bridges

The movable span of a swing bridge, also termed the draw, rotates about a vertical axis, which is often called the pivot axis. If the pivot axis is at midlength of the draw, the draw is said to have equal length arms or be symmetrical. Sometimes, the arms are not of equal length, and the draw is termed unequal-armed or bobtailed. The dead load (self-weight) of a

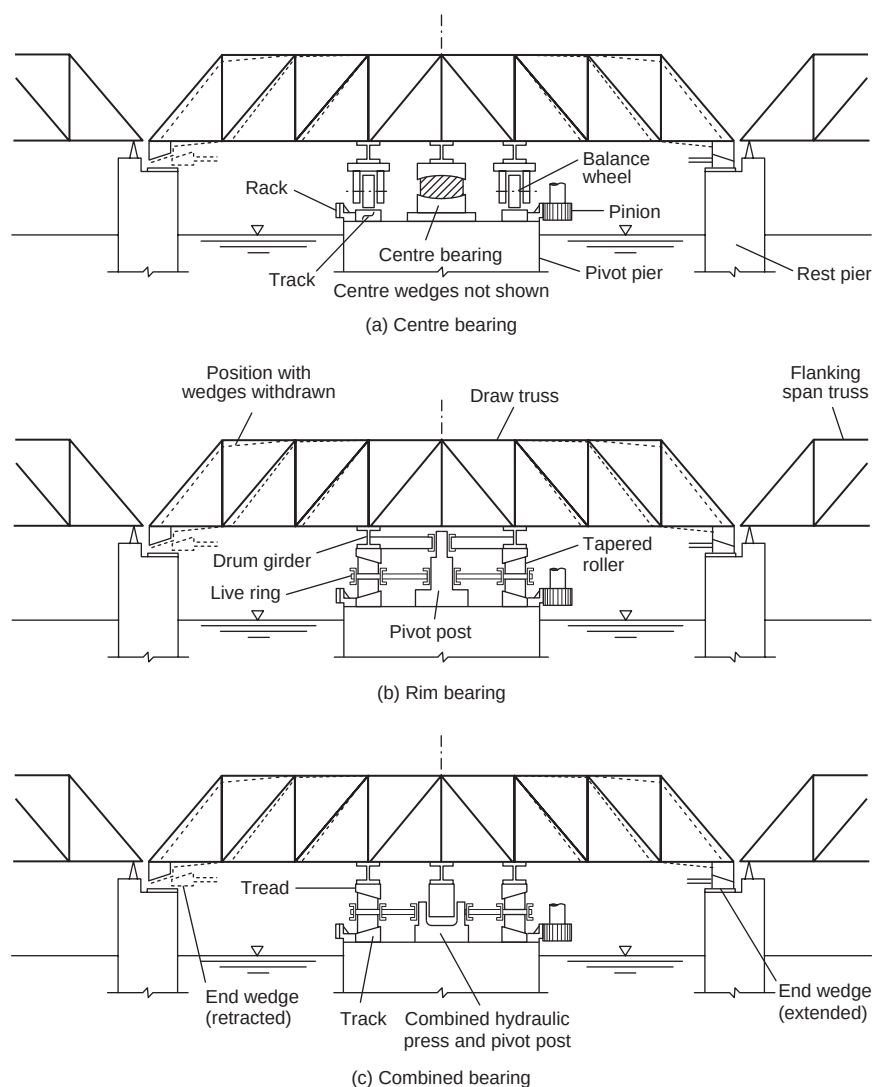


Figure 16.9 Types of swing bridge

swing span is usually balanced about the pivot. Hence, bobtailed spans require counterweights at the ends of the shorter arms for balance. However, short-span swing bridges were built with only one arm and no counterweight, but special pivots were required for stability. See Figure 4.26 in Birnstiel *et al.* (2015). Those pivots have to resist bending about any horizontal axis.

Figure 16.9 shows an equal-arm swing bridge with various pivots. The clear width of the navigation channel attainable is only about 80% of the length of an arm, or about 40% of the length of the draw, because of the space occupied by the pivot pier, the rest piers and the fenders. To obtain a wider channel, two swing bridges may be built in tandem. This was done at Yorktown, Virginia, where a tandem swing bridge, the George P. Coleman Memorial Bridge, was opened to traffic across the York River (Virginia, USA) in 1952. Each centre-bearing draw was 500 ft (153 m) long, giving a 450 ft (137 m) wide clear channel (Quade, 1954). The bridge had a two-lane roadway, which became inadequate with the increased traffic 40 years later, and it was replaced by a wider bridge of similar design built on the original piers (Green, 1996). A double-draw swing bridge with a distance between pivot axes of 167.5 m (550 ft) was built across the Suez Canal at El Ferdan, Egypt, in the early 1960s (Sedlacek, 1965). In 1967, it was destroyed by Egypt during the Six-Day War. The current tandem bobtailed swing bridge located there was completed in 2001. It is larger than its predecessors, to accommodate a widened canal. It is 340 m (1115 ft) between pivot axes (Adrian *et al.*, 2000; Tomlinson *et al.*, 2000).

Centre-bearing swing

Swing bridges are also categorised according to the manner in which the draw is supported at the pivot pier when the draw is in the open position. If all the dead load is supported by a pivot bearing at the axis of rotation, it is termed centre bearing. The draw weight is usually balanced on this pivot bearing, which may be mechanical or hydraulic. To prevent the open draw from tipping excessively under unbalanced loads, such as misplaced live loads or wind, balance wheels are provided that roll on a large-diameter circular track concentric with the pivot bearing. When the draw is balanced, these wheels normally clear the track by about 5 mm (0.2 in). The design intent is that the centre bearing support all the dead load when the draw is open. Figure 16.9(a) is a diagram of a centre-bearing swing bridge. Pivot bearings of plain mechanical hardened steel on bronze are the norm. However, spherical antifriction rolling element thrust bearings have been installed. The new Third Avenue Bridge across the Harlem River in New York City is a centre-bearing swing bridge with a spherical, antifriction roller thrust bearing supporting a draw weighing 5800 kips (2630 t). Spherical bearings with fibre-reinforced polymer (FRP) contact surfaces have also been installed instead of hardened steel-bronze or antifriction rolling element bearings or hydraulic bearings.

The live load on centre-bearing swing bridges is primarily supported by centre and end lift devices (usually wedges or

eccentric rollers), which are actuated when the draw is returned to the closed position. They support the free ends of the trusses and also provide a firm intermediate live load reaction for the trusses at the pivot pier.

Rotation of the draw is by means of mechanical or hydraulic machinery, or a combination thereof, except for small bridges, which may be hand-powered. When the mechanical span drive is mounted on the draw, one or more pinions, mounted on downward extending pinion shafts, engage a circular rack mounted on the pivot pier and rotate the draw. For bridges utilising hydraulic slewing cylinders for rotation, no rack is required for swinging.

Some bobtailed centre-bearing swing spans are supported on a bronze spherical bearing (called a throne) and are tilted about a transverse horizontal axis on opening and then backwards after the span has been returned to the closed position. They are called tilting swing bridges and may be found along narrow waterways in Europe. If the draw is counterweight-heavy, jacks at the tail end lift the tail and thereby force the toe downward onto the forward rest pier to close the bridge. A description of the reconstruction of a counterweight-heavy tilting bobtail swing bridge with excellent graphics of the new machinery is given by Perrier (1994). A few bobtailed swing bridges are span-heavy and, at closing, the toe is lifted by jacks, and bearing blocks are inserted to support the toe in a raised position.

Another type of centre-bearing swing bridge is the pivot post swing. In the closed position, all vertical loads are reacted by a circular structural (usually elastomeric) service bearing of large diameter (Figure 16.10). To swing the draw, a hydraulic cylinder (or a group of cylinders) lifts the draw and it is rotated by the slewing cylinders. When the draw reaches the desired open position, the hydraulic pressure is released and the span drops onto the service bearing. To close the draw, the

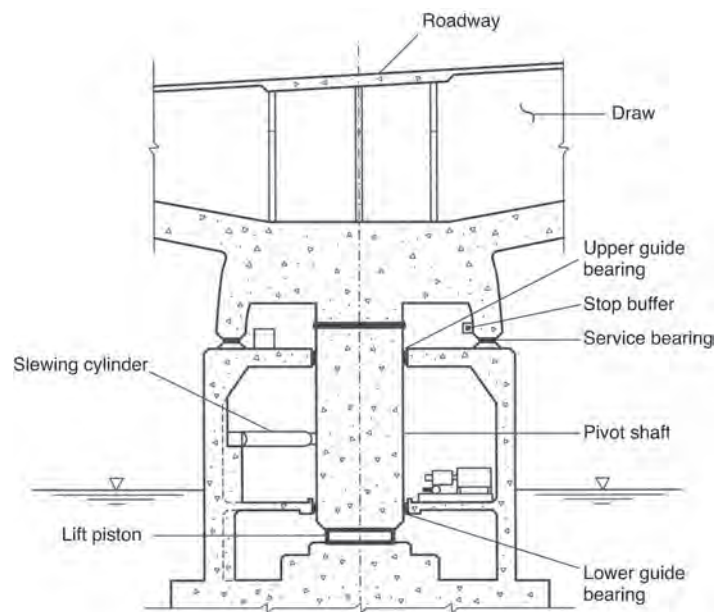


Figure 16.10 Pivot shaft swing

procedure is repeated, but rotation is in the opposite direction. The original lift–turn cylinder (or lift piston) for the Harbor Island Swing Bridge in Seattle, Washington, USA, is described in Hamilton and Lund (1985).

Rim bearing swing

Swing bridges in which all the dead load is supported by conical rollers when the draw is in the open position are termed rim bearing. As shown in Figure 16.9(b), the rollers run on a circular track whose diameter is about the same as the spacing of the outer swing span trusses or girders. When the bridge is closed, the rim bearing supports both dead and live loads. Rim bearings are used for wide, heavily loaded, swing bridges, such as those over the Harlem River in New York City (Birnstiel *et al.*, 2015, Figure 2.24), or long spans, such as at El Ferdan (Tomlinson *et al.*, 2000). Special load-equalising framing is usually provided to transfer the loads from the bridge trusses (which may number two to four) to the circular drum girder so that it is nearly uniformly loaded along its length (at least for dead load). Various schemes for transferring the superstructure loads to the drum girder are described in Hovey (1926, vol. 1). The load is transferred through the drum girder to a conical tread plate supported by conical rollers. The conical rollers bear on a circular conical track plate bolted to a circular stool, which is bolted and grouted to the pivot pier. Rotation of the draw may be by the same means as for the centre-bearing bridge. Alternatively, the mechanical span drive may be mounted on the pivot pier, in which case the pinion shafts engage a rack mounted on the periphery of the drum girder (Birnstiel *et al.*, 2015, Figure 4.13).

Combined bearing swing

Combined centre and rim bearing swing bridges are those equipped with both a centre bearing and a rim bearing (Figure 16.9(c)). Usually, the centre bearing is mechanical, either a plain or antifriction rolling element, but hydraulic bearings have been utilised. In this type of bridge, the dead load is shared by the centre and rim bearings, with the rim bearing usually supporting most of the dead load. The distribution of live load between the centre and rim bearing is a function of the transverse rigidity of the load distribution framing. Uncertainty about load distribution is considered a disadvantage by some designers. Jacking has been used to control the initial distribution of dead load between centre and rim bearings.

The centre bearing may also be a hydraulic press, as shown in Figure 16.9(d). The cylinder is pressurised during operation by water or oil so as to reduce the amount of dead load that the tapered rollers have to bear. The pivot press of the Barton Swing Aqueduct is shown in Birnstiel (2000a, Figures 12 and 13). Minimising the load on the rollers minimises the energy required to rotate the draw. Some swing bridges have been built that have an external pontoon that serves the same function as the hydraulic press.

Slewing bearing swing

Slewing bearing swings are rim bearing but without the central pivot, as shown in Figure 16.9(e). Rolling element thrust bearings of large diameter, developed for military applications, are used instead of conical rollers. Some designers use the slewing bearings to support both live and dead loads, as they are used in construction machinery applications. Other designers use them only to support the draw while it is rotating. The latter design concept requires means for transferring the weight of the draw to a separate support system. This system has been denoted the lower-turn slewing bearing. Operations of this type will be described later, in connection with the Selby Swing Bridge.

Pontoon-supported swing

Swing bridges have been built that are wholly or partly supported by pontoons, usually across protected waterways with slow currents. Those that were built with one end of the draw supported by a fixed pier are usually on waterways with small fluctuations of water level. Examples of pontoon-supported swing bridges were the Galata Bridge at Istanbul, Turkey, built in 1912 and replaced in 1990 by a bascule bridge, and a railroad bridge across the Suez Canal at Kantara, Egypt (Hawranek, 1936). The Galata Bridge was hinged at one end to a moored pontoon and originally swung by propellers driven by electric motors located in the pontoon at the free end of the draw. The Suez Canal bridge was supported by a fixed pier at the hinged end and was also swung by a propeller drive. Many of the shorter draws are swung open by pulling the pontoon at the free end using a chain drive. A large swing bridge with one end supported by a pontoon was built at Osaka, Japan (Watanabe, 2003).

Vertical lift bridges

The movable span of a vertical lift bridge is raised in order to provide additional clearance for the passage of vessels. It is usually balanced (nominally) by counterweights connected to wire ropes that pass over sheaves at the rest piers of the lift span. The sheaves are normally situated atop towers but they may be located inside the towers or piers. The purpose of the counterweights is to minimise the energy required to move the span. This type of bridge is known as a balanced vertical lift. Vertical lift bridges are categorised by the arrangement of the drive machinery. Simplified diagrams of some types of balanced vertical lift bridges – the span drive, the tower drive, the connected tower drive and the pit drive (or table lift) – are shown in Figure 16.11. For large lift bridges, the weight of the counterweight ropes is so great that they themselves are counterweighted by an auxiliary counterweight system.

Since Waddell's design and patent of the first practical vertical lift bridge, many such bridges have been erected. Owing to Waddell's patents and the relationship between his

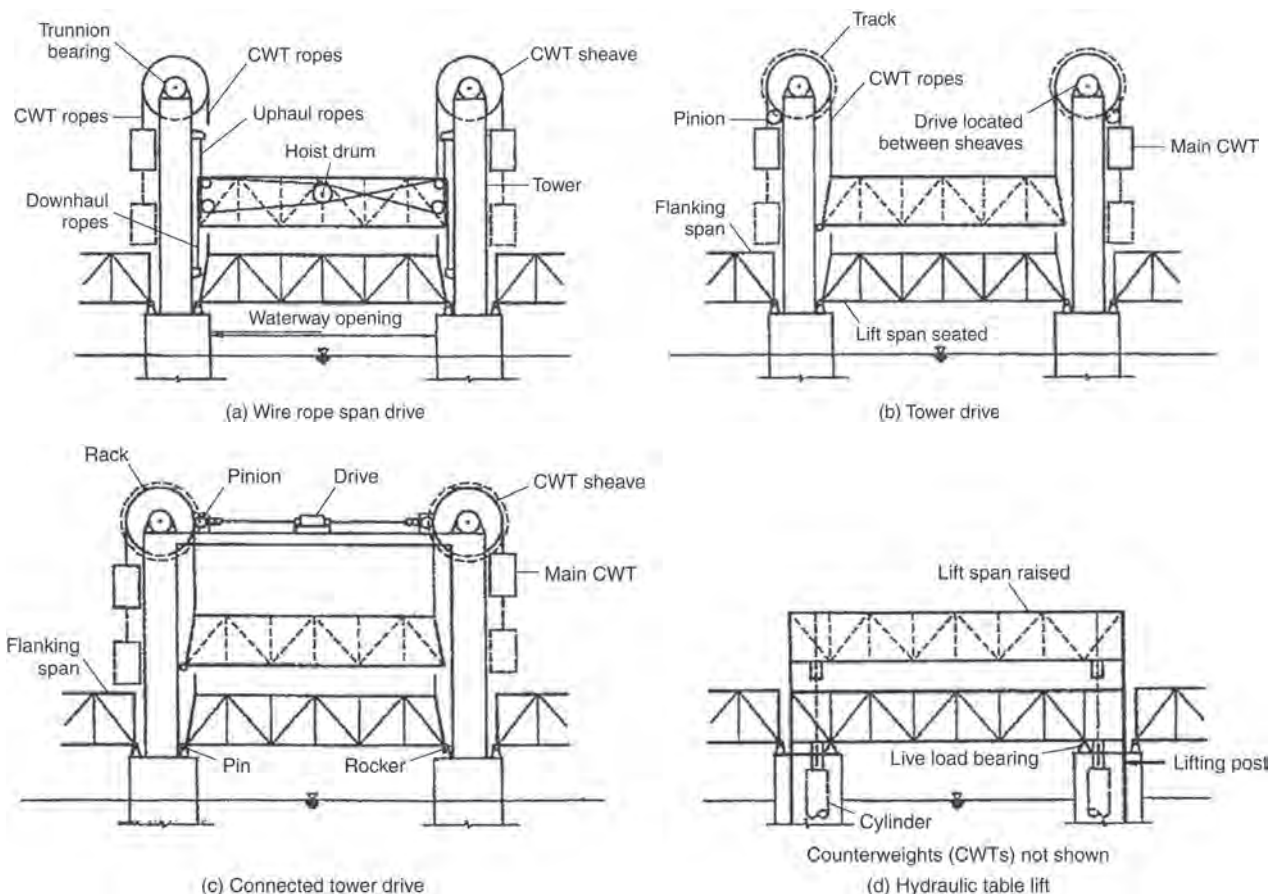


Figure 16.11 Vertical lift bridges

and successor firms, and firms that were founded by John Lyle Harrington, a former partner of Waddell, most large vertical lift bridges built in the USA prior to 1950 were designed by three consulting engineering firms (Nyman, 2002).

Span drive vertical lift

Figure 16.11(a) depicts a span drive vertical lift powered by wire rope hoist machinery located on the span, usually at midspan above the roadway or railroad tracks. The wire rope system hauls the lift span up or down. Because the four rope drums (one for each corner of the lift spans) are usually geared to a common drive shaft, the span cannot skew appreciably longitudinally or transversely, unless there is a problem with a haul rope. This type of span drive vertical lift bridge was perfected by the Waddell firm, and many were built by the Pennsylvania Railroad Company.

Wire rope span drive vertical lift bridges have been built with the primary drive at midspan transmitting power through long line shafts to the secondary speed reductions and hoist drums located at the ends of the span. An example of such a bridge is the double-track Burlington Northern Santa Fe Railroad Bridge at Portland, Oregon, USA. The lift span has a length of 516 ft (157 m) and the normal lift is 146.25 ft (45 m), giving a

vertical clearance of 200.7 ft (61 m) above low water (Birnstiel and Tang, 1998).

Span drive vertical lifts that are raised by a rack-and-pinion drive at the towers have been built for shorter spans as, for example, those over the Illinois River, USA.

Tower drive vertical lift

In the tower drive vertical lift bridge depicted in Figure 16.11(b), there is span drive machinery in each tower to rotate the counterweight sheaves. The forces necessary to raise the span are transmitted to the counterweight ropes by friction. The action is similar to that of a traction drive passenger elevator in a building. Both ends of the lift span should raise and lower at the same rate so that the lift span remains horizontal and does not wedge itself between the towers during motion. There are various electrical or electronic means of controlling the drives in the two towers, so that skew is kept within permissible limits. It should be noted that the force necessary to raise the lift span at each end may differ, owing to unequal machinery friction, and so on. Because of variation in the extension of counterweight ropes and their slippage on the sheaves, mechanical means are often provided to facilitate length adjustments. The world's longest vertical lift span, 558 ft (170 m), is a tower drive vertical

lift that supports a single-track railroad across Arthur Kill between Staten Island, New York and Elizabeth, New Jersey (Heddefine and Kuesel, 1959). The longest-span tower drive vertical lift highway bridge is the Marine Parkway–Gil Hodges Memorial Bridge over Rockaway Inlet, in the Borough of Queens, New York City, with a lift span 540 ft (165 m) long, erected in 1937.

Tower base drive vertical lift

Figure 16.12(b) shows the general arrangement of the machinery of tower base drive vertical lift bridges. The operating winch drums are located within the base of the tower. As shown in the figure, the counterweight ropes are attached to the lift span and pass over the counterweight sheaves to the counterweights. By fastening the haul ropes to the counterweights, slippage of the counterweight ropes in the counterweight

sheave grooves does not tilt the lift span. Figure 16.13 depicts a base drive vertical lift bridge.

Connected tower drive vertical lift

Figure 16.11(c) shows a connected tower drive vertical lift bridge. The lift span is balanced by counterweights. However, because this type of lift bridge is most suitable for short spans of low to moderate lift, the counterweight ropes are usually not balanced by auxiliary counterweight systems. The span drive machinery is mounted on the structure connecting the towers. Primary machinery is located in a machine room at midspan, from which line shafts extend to secondary right-angle speed reducers at the towers, midway between the counterweight sheaves. From the secondary reducers, shafts extend to pinions that engage the circular racks fastened to the counterweight rope sheaves. The force necessary to move or hold the lift span

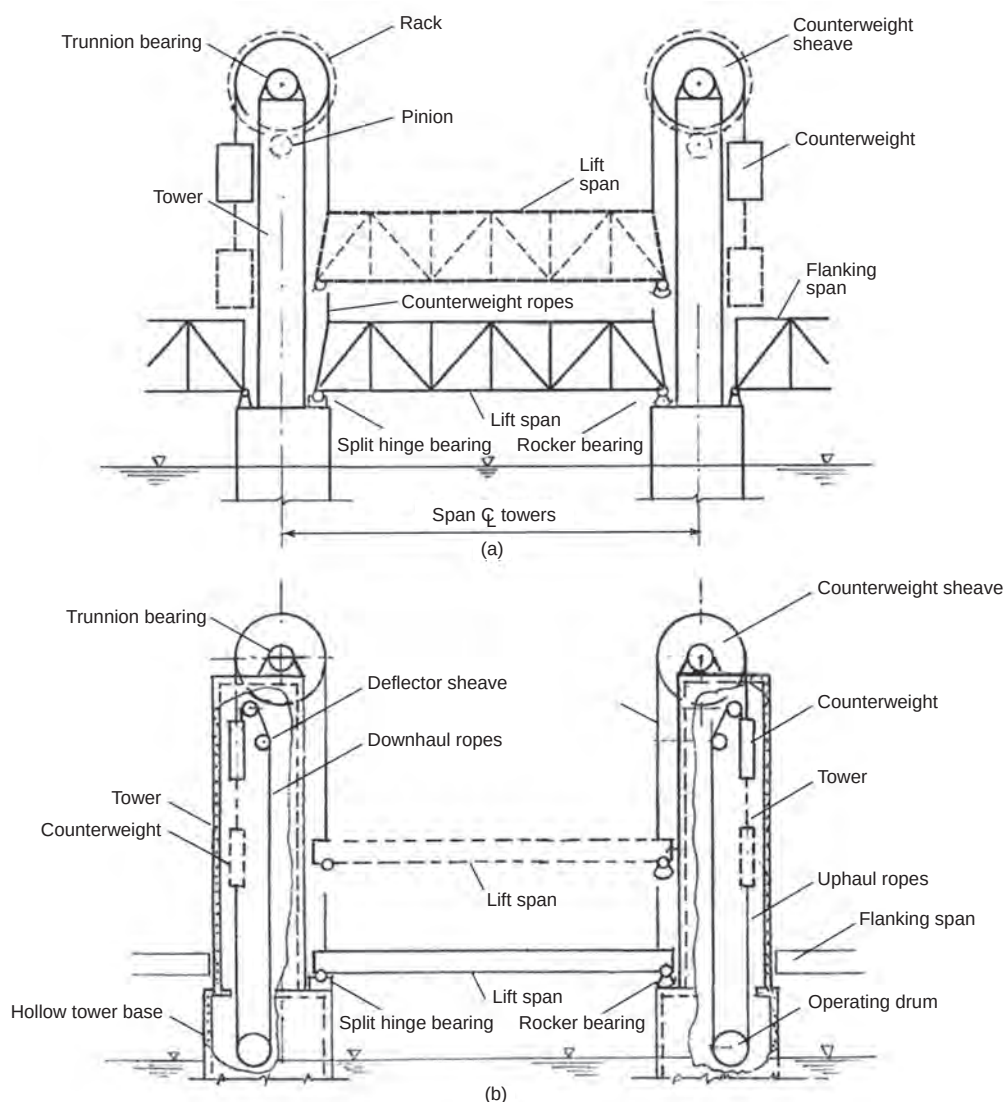


Figure 16.12 Tower drive and tower base drive vertical lift bridges



Figure 16.13 Centenary Bridge, Manchester, UK (photograph courtesy of C Birnstiel)

is transmitted between the sheaves and the counterweight ropes by friction. To compensate for variable elongation of the hoist ropes, adjustable couplings are incorporated in the line shafts.

Pit drive vertical lift

Pit drive vertical lift spans or table lift spans, as they are sometimes called, are raised and lowered by machinery located within pits that are situated inside the rest piers. They may be symmetrical table lifts, having span drive machinery in each pier, or asymmetrical, with span drive machinery on one end of the span only. The pits usually extend downward below the high-water level in the channel. The design objective is, of course, to avoid the need for towers above the roadway. This is accomplished by fastening downward extending legs (lifting posts) to the ends of the lift span, as shown in Figure 16.11(d). The lifting posts guide the movable span during motion and resist horizontal forces applied to the span when it is not seated on its structural bearings. One or two lifting posts may be provided at each end of the span, depending on the deck width. If counterweights are used, the counterweight ropes (not shown in the figure) are fastened to the lower ends of these lifting posts and looped over sheaves located above and then connected to the counterweight. If counterweights are not used, the lifting system usually includes hydraulic cylinders. This type of vertical lift bridge is suitable for low lift heights.

Retractable

Retractable bridges translate along a fixed horizontal path. The shorter spans are supported by wheeled bogies that roll on tracks in order to clear the waterway. Few rolling retractable bridges were constructed in the USA. Most rigid units that are mounted on wheeled trucks roll on crane or railroad rails oriented at 45° to the navigation channel. An example is the Borden Avenue Bridge across Dutch Kills in the Borough of Queens, New York City. Examples of in-line retractable bridges, in which the longitudinal axis of the bridge coincides within the axis of the tracks are the Pont des Belges mentioned earlier and the aluminium pedestrian bridge alongside the Hörn Fold- ing Bridge in Kiel, Germany.

Because about one-half of the movable span cantilevers beyond the tracks when the bridge is rolled off the rest pier supports, retractile bridges that roll on land are not suitable for spanning wide channels. To overcome the limitation of cantilevered rigid units on wheeled trucks, floating spans have been utilised that retract under the flanking spans (which may be fixed or floating). They are pontoons, or are supported by pontoons, and are usually moved by wire rope systems. The longest retractile span is the Ford Island Bridge at Pearl Harbor, Hawaii. The draw is a single reinforced concrete pontoon 930 ft (283 m) long that provides a 575 ft (175 m) clear waterway opening for navigation when withdrawn under the fixed trestle approach (Abrahams, 1996).

Pontoon retractile bridges are constructed when the depth to adequate foundation material is excessive or a very wide waterway channel is necessary. Such is the case for the Evergreen Point Bridge over Lake Washington at Seattle, Washington. Storms have created waves that have damaged the retractile span mechanism at least three times since it was constructed, around 1960 (Daniels, 1999).

Transporter

Transporter bridges comprise an overhead fixed span on which a track is mounted for a carriage that can be moved longitudinally. The track may be self-propelled or drawn back-and-forth by a continuous cable drive. A gondola is suspended from the movable carriage by means of cables or latticed structural steel hangers. The gondola transports pedestrians and vehicles across the channel. The bridge type was said to have been invented by the French engineer and contractor Ferdinand Arnodin late in the nineteenth century. There is controversy on this point – but there is no controversy on his involvement as contractor or consultant on about 15 transporter bridges built in France, England, Spain, South America and the USA, one of which had a span in excess of 1000 ft (305 m). An Arnodin transporter bridge operates at Newport, south Wales, UK (Birnstiel *et al.*, 2015).

Another operating transporter bridge is that across the North Sea–Baltic Canal at Rendsburg, Germany. The traveller runs underneath a high-level double-track railroad bridge that was opened to traffic in 1913. The continuous trusses of the bridge have a main span of 140 m with a vertical clearance of 42 m. As is evident from Figure 16.14, the cabin is suspended from inclined wire ropes fastened to the traveller. The horizontal distance between landings is 120 m, which can be traversed in 90 s. Recently, the gondola was damaged by a collision with a vessel and will be replaced. Operation of the transporter is to resume in 2021.

Gyratory

An unusual movable bridge concept of current interest is the so-called gyratory bridge. The operating principle is that a



Figure 16.14 Transporter bridge at Rendsburg, Germany (photograph courtesy of J Baetke)

movable span rotates about an axis crossing the waterway. In the description of the US patent granted to E Swensson in 1909, the axis is elevated above the closed movable span; see Figure 16.2(b). At both shores, there is a tower with a trunnion at an elevation of one-half the lift height. The axes of the trunnions are collinear. At each end, a portal frame (a vertical crank arm) extends downward to the movable span, which may be a box truss. To open the bridge for marine traffic, the crank arms are rotated 180° about the trunnion axis. The movable span, which is below the trunnions when the bridge is closed, is then directly overhead when the bridge is open. The waterway clearance is approximately twice the height of the trunnions above the roadway. Counterweights are provided at the free ends of the arms so that the centre of gravity of the moving mass can be made to lie on, or close to, the axis about which it rotates. Waddell (1916) referred to the gyratory lift bridge as ‘another freak structure – impractical, uneconomic, but exceedingly ingenious’.

The author is aware of only three existing movable bridges that may be classified as gyratory: (a) the Gateshead Millennium Bridge across the River Tyne near Newcastle; (b) the Sint-Annabrug over the Dender at Aalst, Belgium, both of which rotate about longitudinal axes transverse to the channel; and (c) the Scheepsdalebrug in Brugge, Belgium, which rotates about a longitudinal axis transverse to the channel that simultaneously translates parallel to the channel (it retreats from the channel, as does the axis of a Scherzer bascule, on opening and advances to the channel on closing).

The Scheepsdalebrug may, alternatively, be classified as a ‘Scherzer-like’ rolling bascule. It is oriented at 90° in plan to the normal situation. It could be considered a two-girder rolling bascule with a track on each side of the channel, instead of two tracks normal to the channel. The disadvantage of the Scheepsdalebrug orientation compared with normal rolling bridges is that waterway clearance is limited when the bridge is in the open position.

Folding

The concept of a bridge deck that folds about one or more transverse horizontal hinges in order to clear a waterway is

not new. These early folding bridges were designed using lightweight members arranged in a balancing fashion so as to minimise the force necessary to operate them. Figure 16.2(c) shows a folding pedestrian bridge in which the top chord shortens to coil the bridge. The change in length can be accomplished using screw or hydraulic actuators. At Paddington Basin, in London, the hydraulic cylinders are vertical when the bridge is closed. Pressurisation of the cylinders changes the shape of the geometrical figures so that the bridge folds into a coil. It is shown in the closed position in Figure 16.15. A larger folding pedestrian bridge, in Kiel Harbour, will be described in the section on significant movable bridges.



Figure 16.15 Folding pedestrian bridge at Paddington Basin, London, UK (photograph courtesy of C Birnstiel)

Summary – types of movable bridge

Configurations of the major types of movable bridges and some subtypes have been illustrated. They are denoted as: trunnion bascule, rolling bascule, swing, vertical lift, retractile, transporter, gyratory and folding. More subtypes exist but they are not shown because of space limitations. The percentages of the movable bridge types vary from country to country. On British Waterways, the proportion of swings is 75%, of bascules 20% and of vertical lifts 5%. All types are represented on Belgian waterways, but the percentages of various types are not documented. In Finland, 60% are bascules, 35% are swings and 5% are vertical lifts. An analysis of data for USA movables listed in an appendix to Koglin (2003) indicates that the distribution of the major types in operation is approximately: swings 45%, bascules 40%, vertical lifts 10%, retractile and other 5%. The percentage of movable bridges that are supported by pontoons is 2% (Birnstiel, 2007). In general, the percentage of vertical lifts is greater in countries requiring wider navigation channels.

The basic movable bridge forms originated four centuries ago when canals became important features of European countries. Although some of these types were made obsolete by the requirements of modern transportation systems, they now appeal to architects seeking more dynamic bridge forms.

Structural forms and mechanical–structural interaction

The design of a movable bridge superstructure generally follows the concepts and procedures described for fixed bridges in previous chapters of this book. However, some factors relating to the interaction of the structure with the stabilising machinery need to be considered. They will be discussed subsequently for the major types of bridges under separate headings.

Movable bridge superstructures built during the past century, and before, have been described from the engineering perspective (Hawranek, 1936; Hool and Kinne, 1943; Hovey, 1926; Koglin, 2003; Kunz, 1915; Waddell, 1916). Hool and Kinne (1943) give an excellent treatment of the common simple trunnion bascule in straddle bearings with complete computations, including those for span balancing. Hovey (1926) describes swing bridges thoroughly, especially centre-bearing swings, and explains the precautions necessary for the loading of rim bearings. The vertical lift bridge is discussed in detail by Waddell (1916). These authors mostly discuss American bridges. Hawranek (1936) shows an international array of movable bridges and is strong on articulated bascules. From today's perspective, most of the examples in these books could be classed as practical, straightforward, industrial architecture.

More modern forms, in the sense that they are considered dynamic and 'playful', have been built or proposed and are shown in Bennett (1997), Troyano (2003) and Wilkinson and Eyre (2001). That there is more emphasis on architecture in contemporary movable bridge design is obvious. In the nineteenth and twentieth centuries, the design of movable bridges was dominated by engineers whose objectives were to build sound structures (machines) that were economical with respect to first and operating costs. Aesthetics was a secondary consideration. These priorities led to movable bridge designs that were straightforward industrial architecture, no longer appreciated by a segment of the contemporary public. As bridge construction involved more public funding than in the past, the public agitated for 'better-looking' bridges.

In the 1950s, the architectural philosophy of Ludwig Mies van der Rohe (1886–1969) that 'less is more' became influential. Svensson's (2000) guidelines for bridge aesthetics are generally consistent with that approach. His important factors are 'clear structural statements, good proportions, order, compatibility with the surroundings including colouring and above all, simplicity.'

But society's concept of 'better-looking' changes with time, and now the dictum of Robert Venturi (1925–2018) that 'less is a bore' is accepted by many. Hence, architects are being included on movable bridge design teams – to propose something different and dynamic to the public. As part of this process, movable bridge concepts that engineers considered obsolete for practical reasons are being reinvented by architects.

Bascule bridges

Simple trunnion bascule

Simple trunnion bascules rotating about straddle bearings are essentially parallel cantilever girders supporting a floor deck system. More girders were used per leaf in the early bridges than is the usual practice now. The Scott Street Bridge in Hull has eight girders per leaf connected to a continuous trunnion over seven bearings. Each leaf of Tower Bridge has four bascule girders (Birnstiel, 2000b). Currently, most simple trunnion bascules have two. For extremely wide roadways, such as at the Woodrow Wilson Bridge, the bridge is divided into a number of units, each having two girders.

With regard to structural action, the girder of a single-leaf bascule is a cantilever for dead load and a simple span for live load. For a double-leaf bascule, the girder is a cantilever for dead load. Opinions on the design criterion for behaviour under live load differ among engineers. Some rely on a functioning midspan shear lock, in which case the girder is an elastically propped cantilever (Hool and Kinne, 1943). A midspan lock designed to transfer live load bending moment, as well as shear, was installed on the double-leaf bascule bridge over the Eider River at Friedrichstadt, Germany, in about 1919 (Hawranek, 1936). Theoretically, such locks make the bascule girders continuous over four supports to resist live load. However, deformations in the lock bars and mountings reduce their effectiveness. Moment locks were installed on large double-leaf Strauss trunnion bascules designed for railway traffic, for example, the Sault Ste. Marie International Railroad Bridge, Michigan, a Strauss heel trunnion bascule bridge of 336 ft (102.41 m) span, built in 1914, bridges at Galata (Saul and Zellner, 1991) and Valencia (Ibarguen *et al.*, 2002) and the Woodrow Wilson Memorial Bridge (Arzoumanidis and Bluni, 2007; Foerster, 2006).

An important consideration in the layout of the leaf framing members is the joint between the movable leaf and the fixed structure, especially the clearances for all positions of the leaf, and a means of deck drainage to avoid rain dousing the trunnion bearings and other machinery.

Rolling bascule

Design considerations are similar to those for simple trunnion bascules, except that early double-leaf rolling bascules (Scherzer bascules) were constructed so as to act as three-hinged arches in resisting live load located forward of the centre of roll. This action is illustrated in Figure 16.16. The concentrated live load W is equilibrated by the pressure lines passing through the three hinges: a midspan hinge and the two hinges formed at the bascule piers by the front teeth of the track and the corresponding sockets in the treads.

Double-leaf Scherzer bascules that acted as cantilevers were also equipped with supplementary live load reactions, either

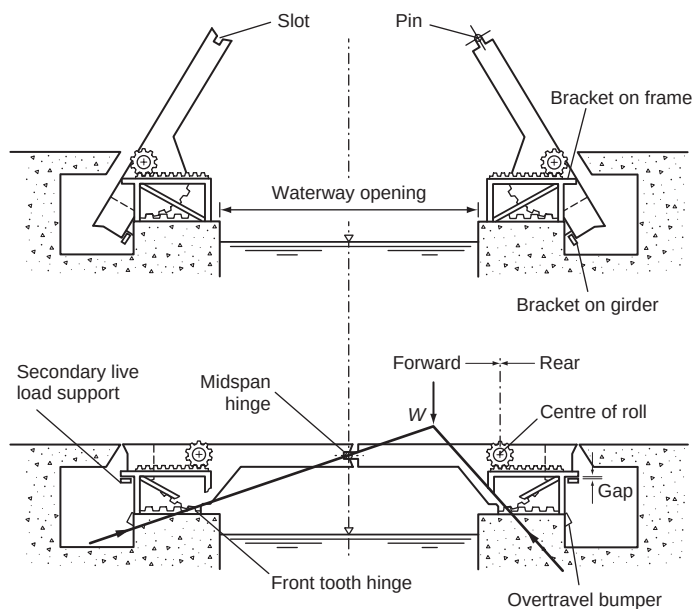


Figure 16.16 Three-hinged arch action of some rolling bascules

at the rear of the leaf or at the front wall. They are not normally active; they only became active when a leaf is lowered too far, as can happen when only one leaf is being closed while the other leaf is open. Live load rearward of the centre of roll produces moment, tending to open the bridge. This action is resisted by the machinery, which is 'wound up' when the midspan hinge is seated on closing, resulting in fluctuating stresses in the span drive machinery as live load passes over the closed bridge.

Because the stability of each leaf when subjected to live load depends on the machinery wind-up, it is important that, for safety, span locks be installed, which will limit leaf movement in the event of brake failure. However, the span lock clearances should be such that the three-hinged arch can deform, owing to temperature change and live load elastic deformations, without restraint. Some vertical movement of the rear of the leaf on loading by heavy wheels at the floor break must be accepted.

In Europe, a few double-leaf rolling bascule bridges were constructed with three-hinged arch action for live load and for a portion of the dead load. In these bridges, part of the counterweight is lifted by machinery at the rear wall of the bascule pit at bridge closing, thereby shifting the centre of gravity of the leaf forward towards the channel. This compresses the arch and seats the midspan hinges prior to the addition of live load. The result is a very rigid bridge in the closed position. Because the leaves are very span-heavy after the centre of gravity is shifted, live loads on the rear of the leaf do not lift the midspan hinge. Examples are the Langebro and Knippelsbro in Copenhagen, Denmark (Birnstiel *et al.*, 2015).

During rehabilitation design projects, some engineers have altered double-leaf Scherzer bridges originally designed for three-hinged arch action from that behaviour to double cantilever action to simplify electrical control design. However,

such an alteration is likely to reduce the rigidity of the closed bridge, which may worsen vibration from vehicular traffic.

Swing bridges

The main structural members of double-arm swing bridges are usually trusses or girders, sometimes cable-stayed. In the bridge-open position they are double-armed cantilevers balanced on the pivot pier, and in the closed position they are supported on the pivot and the rest piers. The centre-bearing swing truss is supported at three points and the rim bearing swing trusses usually at four points. The elevations of the rest pier supports are adjustable so that the reaction at the end of the draw is always upward for any condition of live load and temperature. Maintaining upward reactions at the rest piers is especially important for railroad bridges in order to avoid slapping of the draw because of the heavy moving live load.

During the era of railroad expansion, 1890 to 1910, at least 400 swing bridges were constructed in the USA. Some had draws more than 500 ft (150 m) long and many of these bridges are still operational. Their structural forms varied, mainly in order to obtain the required stiffness and strength, and reduce the effort required to make numerical structural analyses of externally and internally indeterminate bridges. For rim bearing and combined bearing swing bridges, there was the additional consideration that live load should not cause uplift at the girder-truss reactions on the rim bearing drum girder. For example, consider the rim bearing swing span trusses of Figure 16.17. According to conventional beam theory (bending strains only), there would be uplift at support B owing to a live load placed on span C–D. Because span C–D is so much longer than B–C, the uplift at B could, theoretically, be quite large. Of course, the uplift at B would be counteracted by the downward dead load. However, for open-deck railroad bridges, the ratio of live to dead load is often large and, according to conventional beam theory, the net reaction could result in uplift. Uplift at B would be impractical to accommodate in the rim bearing. Fortunately, trusses were usually used as spanning members, instead of solid web girders, and their shear deformations are more significant than shear deformations in solid web girders, with the result that the uplift reaction at B for a truss would be much smaller than for a solid web beam. Nevertheless, designers wanted to minimise the possibility of uplift at B, so the truss diagonals in panel B–C were omitted, or made so flexible that they were ineffective in transmitting vertical shear across panel B–C. Configuring the truss so that panel B–C could not transmit much shear also had the advantage that it reduced the degree of structural indeterminacy by one. This was important at the end of the nineteenth century because it significantly reduced the complexity of structural analysis. This topic is discussed in Birnstiel (2008a), Johnson *et al.* (1909) and Parcel (1936).

Some popular forms of swing span truss used for rim bearing swing bridges in the bridge-closed position are shown in

Figure 16.17. In each diagram, A and D are the ends of trusses at which lifting machinery is located. B and C are supports for the trusses at the rim bearing. For each truss, the features of equations of condition and the structural analysis for the case of gravity loading will be described. Because it is stipulated that there are no horizontal loads for these examples, the static requirement that the sum of the horizontal forces equals zero is, by definition, satisfied.

- **Pratt truss** with weak diagonals BF and CE. With BF and CE inactive (or virtually so), no vertical shear can be transmitted across the centre panel. This is an equation of condition. The structure is externally indeterminate to first degree.
- **Warren truss with verticals**. Members EF and FG are eyebar tension members. When ends A and D are lifted, these bars buckle and are effectively removed from the system. There is no (little) live load shear transfer across panel BC because of the buckled bars. The structure is externally determinate.
- **Lattice truss**. There are no equations of condition. The structure is externally indeterminate to second degree. This structure is used for many UK swing spans. Uplift of truss at B and C is possible, depending on the live to dead load ratio and the type of connection.

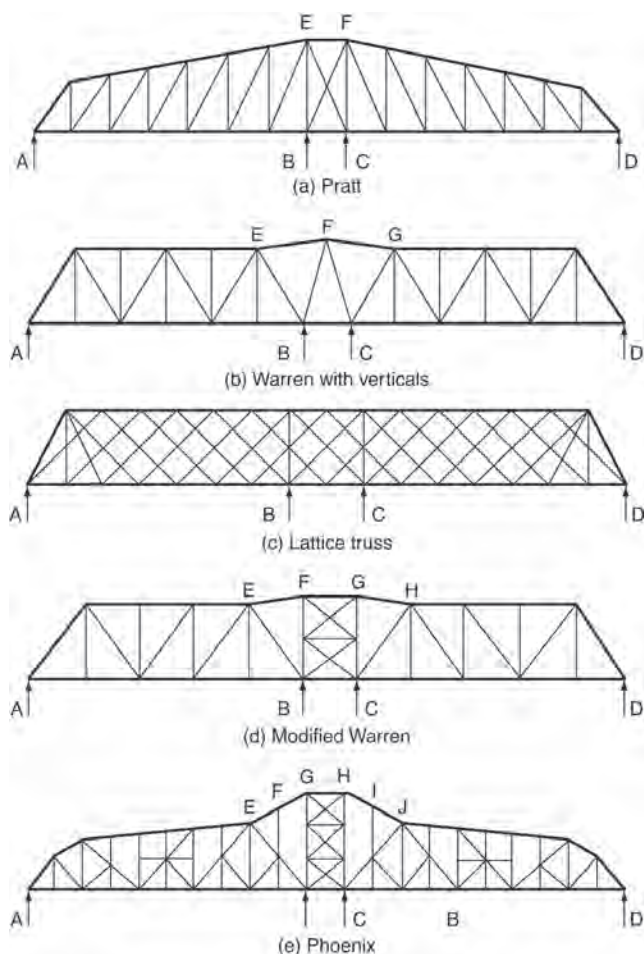


Figure 16.17 Some rim bearing swing bridge superstructure forms

- **Modified Warren truss**. Bars EF and GH are tension-only eyebars, giving two equations of condition. There is no live load vertical shear transfer across panel B–C because of the buckled top chord. The structure is externally determinate.
- **Phoenix draw truss**. Members EF, FG, HI and IJ are eyebars. As ends A and D are lifted, these may buckle. Hence, EG and HJ are equations of condition. There is no live load shear transfer across panel B–C because of buckled bars in the top chord. The structure is externally determinate. A combined bearing swing span 520 ft long was built over the Willamette River in 1907. It was operated until 1989 under very heavy rail and marine traffic.

These examples of engineering approaches for simplifying swing bridge truss models by considering them partially continuous, or simply supported, in the closed position are not necessary today. The computer now makes possible the analysis of almost any elastic structure using the displacement method in matrix form.

Various styles of swing span trusses built in the late nineteenth and early twentieth centuries are described in Dietz (1897), Hawranek (1936), Hovey (1926), Hool and Kinne (1943) and Kunz (1915). The structural analysis of some styles of swing span trusses and girders is discussed in Johnson *et al.* (1909) and Parcel (1936).

A form of movable bridge of current interest is the cable-stayed swing, seemingly introduced by Calatrava in a competition for a bridge across the River Garonne at Bordeaux, France, in 1991. However, the cable-stayed draw is not a new concept. Many cable-stayed swing bridges were constructed in the nineteenth century, especially in North America, from New York Harbor to the Lake District on the north-central Iowa Plains and southward from Ontario to the bayous of Louisiana. Originally built of timber with wrought iron rod stays, most were replaced by wrought iron cable-stayed swings in response to the demand for wider navigation channels. A 300 ft (92 m) long wrought iron draw over the Mississippi River with wrought iron chain stays was placed into railroad service in 1865. It was not replaced until 1887, despite the great increase in locomotive weights during the intervening years (Birnstiel, 2008a).

Secondary motions of swing bridges

The structural behaviour of a draw differs in the closed and open positions. In the closed position, the stress due to dead load in the bottom chords (or flange, for girders) is tension near the middle of the arm (excepting some cable-stayed designs). In the open position, the stress is compression. Hence the deflected shapes due to self-weight are different and it is necessary to account for this situation in designing the stabilising machinery as well as the superstructure.

For symmetrical centre-bearing and all rim and combined bearing swing bridges, the ends of the draws are lifted after the draw is rotated to the closed position. The distance that the ends are raised is chosen such that the ends will not rise under any combination of asymmetrical live load and tempera-

ture change. The intent is that the end reactions will always be directed upward, to avoid ‘slapping’ on the end bearing under moving live load.

Tilting swing bridges

Some bobtailed centre-bearing swing spans (mostly in the UK and on continental Europe) are supported on a spherical pivot bearing (called a throne) and are tilted after the span is returned to the closed position. If the draw is counterweight-heavy, jacks at the tail end lift the tail, thereby forcing the toe downward onto the forward rest pier. A few bobtailed swing bridges are span-heavy; at closing, the toe is lifted by jacks and bearing blocks are inserted to support the toe in a raised position.

Raise-and-turn swing bridges

The secondary motions for the ‘lift or lower then turn’ swing spans is illustrated in a description of the secondary motions for the Selby Swing Bridge, included in the section on significant movable bridges.

Vertical lift bridges

Descriptions of vertical lift bridges built during the twentieth century may be found in Hawranek (1936), Hool and Kinne (1943), Koglin (2003) and Waddell (1916). Since Waddell’s design and patent of the first practical vertical lift bridge, many such bridges have been built.

In Europe, a larger portion of vertical lift bridges are constructed with reinforced concrete towers. Independent towers along the sides of the roadway are popular, especially if the ratio of roadway width to lift span length is large. Independent towers usually require independent main counterweights. Some interesting bridges of this type have been built in Belgium and the UK.

Span drive machinery

Movable bridge machinery comprises span drive machinery and stabilising machinery. Span drive machinery moves (drives) the movable span. It also serves to stabilise the span when the span is not in the fully closed position. Stabilising machinery supports the span when it is at rest and restrains it when in motion, but it does not accelerate the span. Sometimes span drive machinery is denoted *active* and stabilising machinery is called *passive*.

Power for operating early movable bridges was mostly human, and small bascule and swing bridges are still operated by hand. After steam engines became common in the early 1800s, they were used as the power source for bridges well into the twentieth century. In some respects, steam engines were ideal for operating swing bridges; the power (steam) was usually generated directly on the span, the engine was relatively

simple and quiet, and the output rotation was low-speed, high-torque and hence suitable for the application. Marine engines with two cylinders arranged in V-formation were preferred. The direction of rotation was reversible. The disadvantage was the need to maintain a head of steam during the navigation season.

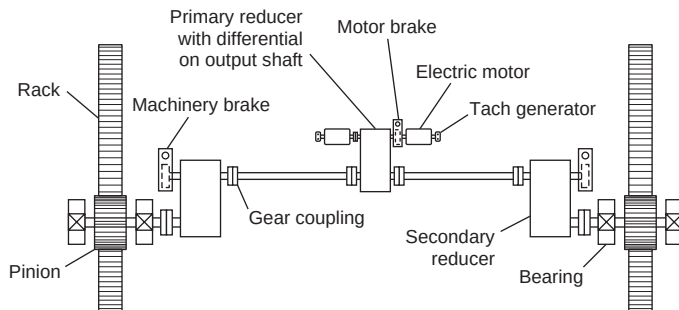
In Great Britain, hydraulic utilities (hydraulic power companies) were established in the 1800s, which transmitted high-pressure water in mains under the streets for industrial power. Liverpool, Kingston upon Hull and London, for example, had such utilities. As the cranes and other dockland machinery were powered by the hydraulic system it was a natural extension to utilise this source for powering movable bridges. An example is the Scott Street Bridge across the River Hull at Kingston upon Hull in East Yorkshire. This double-leaf trunnion bascule was operated by water hydraulic cylinders. Water from the hydraulic utility was increased in pressure by a multiplier and stored at the higher pressure in an accumulator until required to move the leaves. When the hydraulic utility suspended service, electrically driven pumps were installed to supply the accumulator. At other bridges, such as Tower Bridge, the hydraulic pressure was developed on-site by reciprocating steam pumps and stored in gravity accumulators (Birnstiel, 2000b). The local hydraulic utility served as backup. At least one bascule bridge in Belgium is operated by a shifting liquid counterweight. Fluid is pumped from tanks on one side of the axis of rotation (trunnion axis) to tanks on the other side.

In the USA, it was not uncommon to power bridge machinery directly by gasoline or diesel engines, either as the primary or emergency movers. However, as electrical power became widely available, it was adopted for operating movable bridges, and very sophisticated electrical and electronic power and control systems are now routinely installed. The electric motor may drive gearing directly (electromechanical system) or drive a pump, with the power being distributed hydraulically (electrohydraulic-mechanical system). Span drives may be assembled from many combinations of equipment. However, the objective is the same; to convert the low-torque, high-speed, rotational output of the prime mover to the high-torque low-speed rotation needed to move the span. In modern hydraulic systems (fluid power systems), the motor drives a pump, which pressurises a suitable fluid. The fluid is conducted to hydraulic cylinders, which move the span directly, or to hydraulic piston motors, which move the span by rotating a rack pinion, usually through intermediate gearing. In what follows, three span drive arrangements are described at an elementary level: mechanical transmission, hydraulic radial piston motor and hydraulic cylinder. Each is suitable for powering trunnion bascule, rolling bascule, swing and vertical lift bridges.

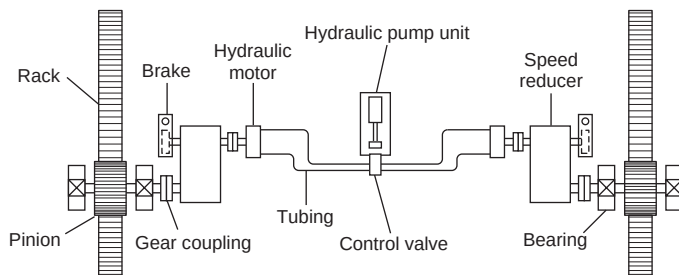
Mechanical transmission

Figure 16.18(a) illustrates a basic mechanical drive that is used to move swing spans, bascules and lift spans. The electric motor or other prime mover is connected to the input shaft of a geared

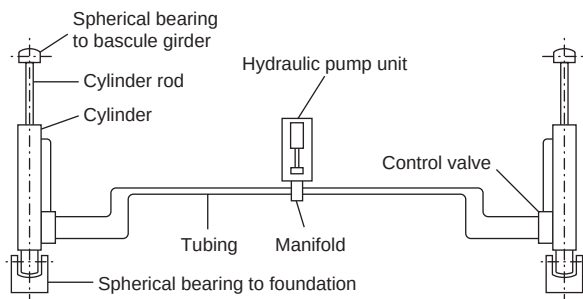
primary speed reducer. A differential, or torque equaliser, is usually incorporated within the speed reducer to equalise the torque in each of the two output shafts, even when their speeds differ (the function of this differential here is essentially the same as that in the drive train of an automobile). A brake, called the motor brake, is coupled between the motor and the speed reducer. This brake is the primary mechanical brake.



(a) Mechanical transmission span drive



(b) Hydraulic motor span drive



(c) Hydraulic cylinder span drive

Figure 16.18 Types of span drive

Not all mechanical transmission drives are equipped with differentials. Some reasons for not using differentials are discussed in WISDOT (2002). Equalisation of braking torque is more important than equalising driving torque, especially for swing bridges. The consequences of unequal braking of different independent drives on a swing bridge are related in Birnstiel (1990a).

Referring again to Figure 16.18(a), the output torques from the primary reducer are transmitted by floating shafts to the secondary speed reducer. For bascule and lift bridges, these

are normally parallel shaft reducers. For a swing bridge, the secondary reducers would normally be of the bevel type so that the output shafts are oriented at 90° to the input shaft. The outputs of the secondary reducers are coupled to pinion shafts. The torque in these shafts is transmitted to pinions that engage a rack. For swing spans, tower drive vertical lifts and simple trunnion bascules, the rack would be circular. For rolling (Scherzer) and heel trunnion (Strauss) bascules, the racks would usually be straight. Another brake, called a machinery brake or shaft brake, is shown on the input shaft to the secondary reducer. Machinery brakes should be installed in the gear train as close to the rack pinion as possible. Of course, the closer the brake is to the rack the greater the required torque rating of the brake. It becomes a matter of compromise between cost and safety. The interior of a speed reducer with a differential is shown in Figure 16.19. A typical spring-applied, thruster-released brake is shown in Figure 16.20.

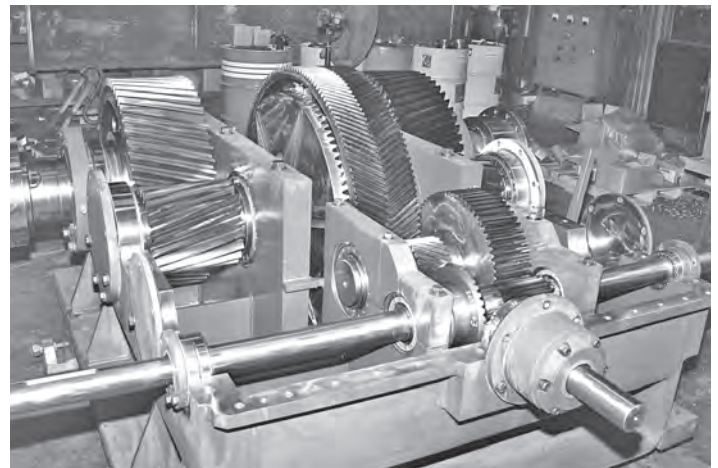


Figure 16.19 Geared speed reducer with differential (photograph courtesy of Steward Machine Co)

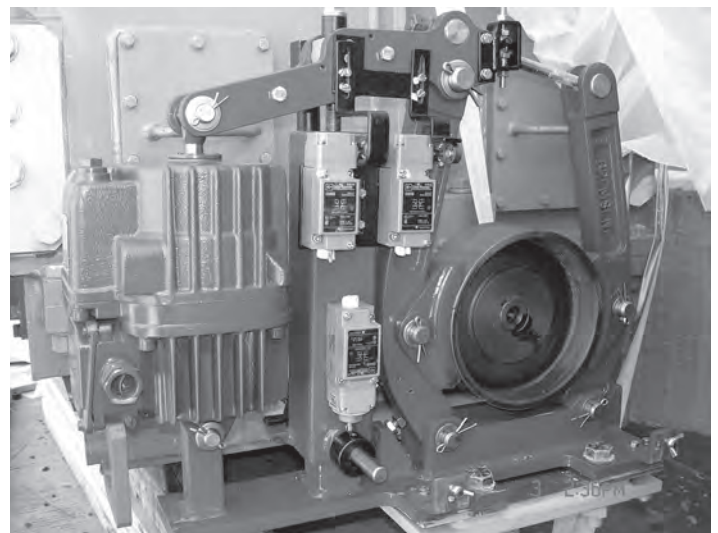


Figure 16.20 Spring-set thruster-released drum brake (photograph courtesy of Link Control Systems)

In the USA, mechanical transmission drives are typically mounted on the fixed structures of simple and heel trunnion bascules and of connected tower and tower drive vertical lift bridges. They are usually on the moving structures of swing bridges, span drive vertical lifts and rolling bascules. Outside the USA, span drives for swing spans and rolling bascules are often mounted on fixed structures.

Hydraulic radial piston motor drive

A span drive diagram in which the electric motor and the primary reducer are replaced by radial piston hydraulic motors is shown in Figure 16.18(b). A radial piston hydraulic motor is depicted in Figure 16.21. The torque output of the hydraulic motors will be matched, to the degree that they are powered from the same pressure source and that the friction losses in the piping from the pressure source to the motors are nearly equal. The remainder of the drive is the same as in Figure 16.18(a). This type of drive can be mounted in the same locations as the mechanical transmission drive or closer to the pinion, with only one gearset between motor and pinion.

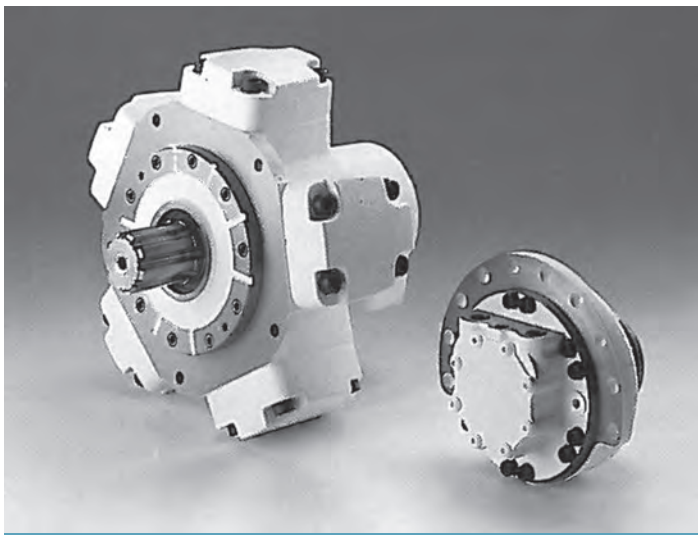


Figure 16.21 Hydraulic radial piston motor (photograph courtesy of Bosch Rexroth)

Hydraulic cylinder drive

Span drives whose output is transmitted to the moving span by linear action, as shown in Figure 16.18(c), have been installed on simple trunnion bascules, rolling bascules, swing bridges and vertical lift bridges. The cylinder mountings shown are for a simple trunnion bascule or pit drive vertical lift bridges. A hydraulic cylinder is shown in Figure 16.22.

Various cylinder mounting arrangements for bascule, swing and vertical lift bridges are shown in Schmitt (1984) and Vanderzanden and Rahe (2016). A common cylinder mounting for simple trunnion bascules is shown in Figure 16.23. An

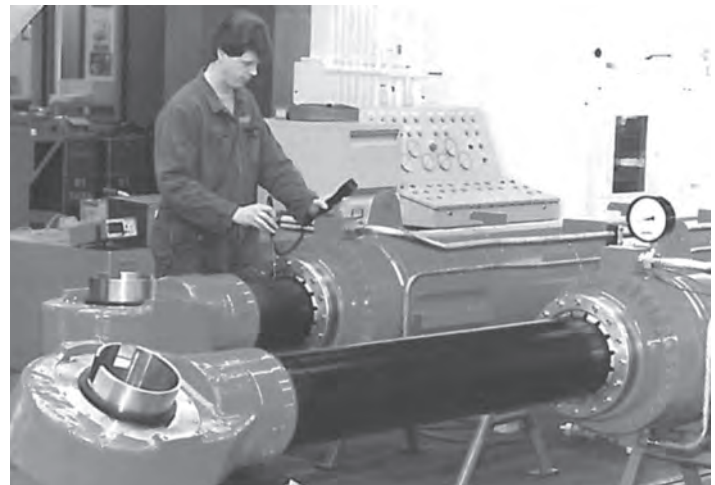


Figure 16.22 Hydraulic cylinder (photograph courtesy of Bosch Rexroth)

interesting bascule bridge with cylinders located above the roadway was built in Belgium (Ney and Adriaenssens, 2007). It is termed a piston-stay bascule because the cylinders are part of the backstays. Thus, the backstays serve both as structure and as part of the operating mechanism. See the section on bascule bridges.

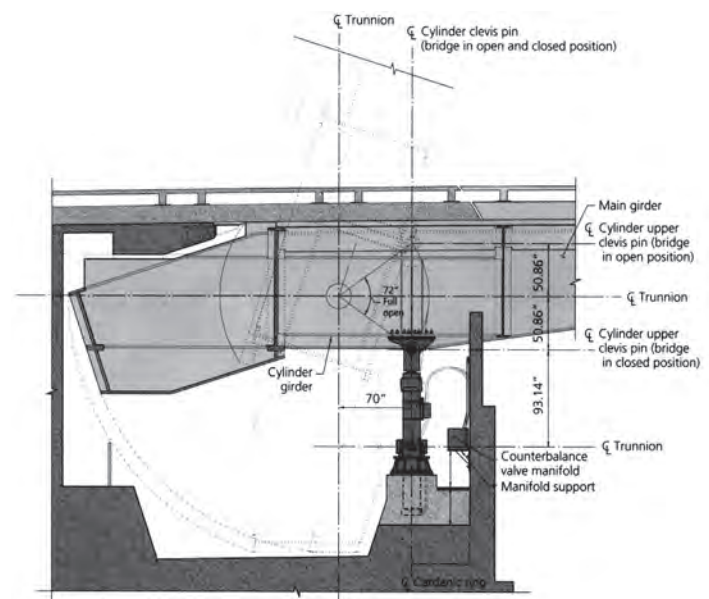


Figure 16.23 Trunnion bascule with hydraulic cylinder drive

Stabilising machinery

Besides the machinery that moves the movable span, machinery is required to stabilise it when at rest or moving. Major items of stabilising machinery (or passive machinery) will be noted for bascule, swing and vertical lift bridges. Detailed descriptions of stabilising machinery are beyond the scope of this chapter.

Trunnion bascule

Stabilising machinery for a single-leaf trunnion bascule bridge is shown in Figure 16.24. It comprises the trunnions, live load reactions, centring device and toe locks. The trunnion bearings are normally bronze plain linings. However, self-aligning rolling element bearings and plain spherical bearings with polyurethane linings are finding increasing favour. There are many variations of toe locks: screw actuated, crank-type and hydraulic.

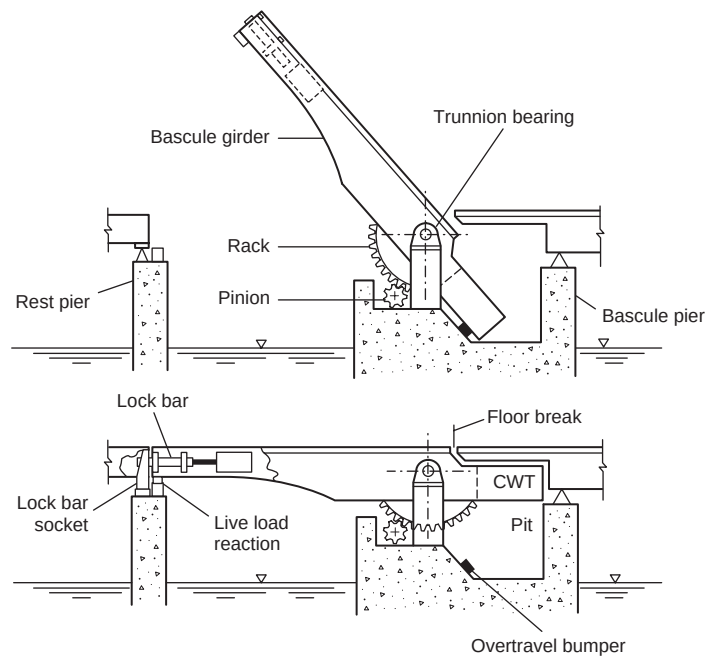


Figure 16.24 Stabilising machinery for single-leaf simple trunnion bascule bridge

Common arrangements of simple trunnion bascule bridge bearings are shown in Figure 16.25. The straddle bearing arrangement (Figure 16.25(a)), in which the trunnions are securely fixed in the web of the bascule girders, is an old arrangement that is still used for heavily loaded bascule girders. Straddle bearings were also used for bascule leaves with more than one girder, as for Tower Bridge, where the trunnion shafts are continuous over eight roller-bearing pillow blocks to support the four bascule girders of each leaf (Birnstiel, 2000b).

Because the work of aligning straddle bearings is often considered to be too difficult nowadays, short trunnions are sometimes cantilevered to bearings outboard of the exterior bascule girders, as shown in Figure 16.25(b). The advantage is that two bearings fewer are required, but the trunnions still have to be aligned; this is usually accomplished using an adjustable feature at the interior end of the trunnion, where it is fixed to the interior deck framing. This bearing arrangement is often denoted the Hopkins type, although the basic design was used long before the first Hopkins bridge was designed. However, because of the flexibility of the interior framing, wobble

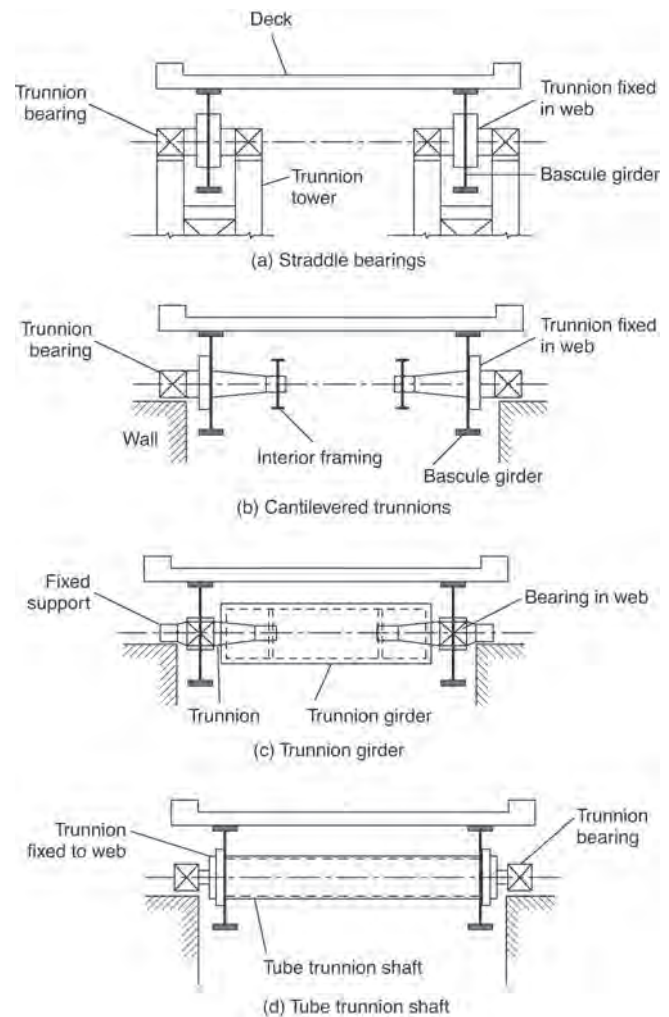


Figure 16.25 Trunnion bearing arrangements for simple trunnion bascule bridges

sometimes occurs, resulting in wear of the sleeve bearings. To overcome this problem, spherical roller bearings or spherical plain bearings are now routinely specified, even for very large bascules. Another advantage of cantilevered trunnions is that the trunnion column may be located outboard of the leaf, thus permitting a larger leaf opening angle for the same span.

After the Second World War, simple bascules of the trunnion girder type (Figure 16.25(c)) were built in the USA. The feature of this design is that the trunnions do not rotate; the bascule girders rotate about the trunnions. The plain bearings mounted within the girder webs are usually inaccessible. Inadequate maintenance has led to excessive friction in the trunnion bearings on some bridges, which is difficult to correct.

The tube trunnion shaft design (Figure 16.25(d)) was developed to overcome difficulties with the other types. The tube is mounted to the bascule girders and rotates with them in self-aligning trunnion bearings. In essence, the trunnion shaft is full length, as at Tower Bridge, but supported only by two outboard bearings.

Double-leaf bascules have, of course, no central rest piers. The live load supports are at the front wall or at the rear of the counterweight. A double-leaf bascule highway bridge with rear live load reactions is described elsewhere (Birnstiel, 1996). The advantage of live load reactions at the front wall is that the cantilevered length of the bascule girder is minimised. However, live overload can cause uplift on the trunnion bearing caps if the distance between the live load supports and the trunnion bearings is short compared with the half-span. To overcome this, some bascule bridges are equipped with primary live load reactions at the front wall and secondary live load reactions at the end of the counterweight. There is a small gap at the secondary reactions at erection so that they will only act when the front of the leaf is heavily loaded. There is also a gap under the trunnion bearing cap so that the trunnion may rise without damaging the cap.

Double-leaf bascules in the USA normally have shear locks at midspan. The shear locks are usually custom-designed for each bridge. However, pre-engineered lock bar assemblies are now popular, especially with disc spring cushions in the lock bar receiver sockets (Figure 16.26).

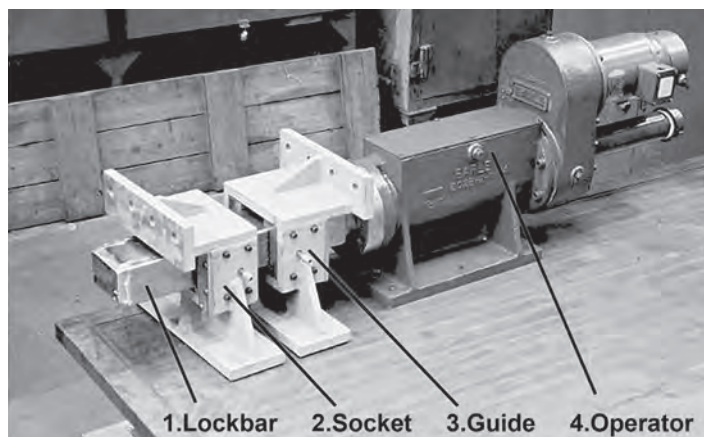


Figure 16.26 Lock bar assembly (photograph courtesy of Steward Machine Co)

Engineers sometimes detail the midspan joints so that they resist live load bending moment as well as shear, as for example at the new Galata Bridge (Saul and Zellner, 1991). An older, smaller, double-leaf trunnion bascule with midspan locks arranged to develop live load bending moment resistance is the Kronprins Frederiks Bro across Roksilde Fjord, Denmark. Another, completed in 1919 and rehabilitated in 1957, is across the Eider River near Friedrichstadt, Schleswig-Holstein, Germany. Midspan locks were provided at the Sault Ste. Marie International Bridge, Michigan, to make the double-leaf bascule behave as a simply supported beam for live load. The Woodrow Wilson Bridge, recently completed at Washington, DC, appears to be the first modern double-leaf girder bascule equipped with moment-resisting midspan locks in the USA (Foerster, 2006).

A detailed study of support and stabilisation systems of double-leaf bascule bridges was presented by Phillips (2016).

Rolling bascule

Stabilising machinery for rolling bascules may include tread and track, live load supports, tail locks and midspan shear locks, as shown in Figure 16.27. There are many varieties of these devices and only a few will be described.

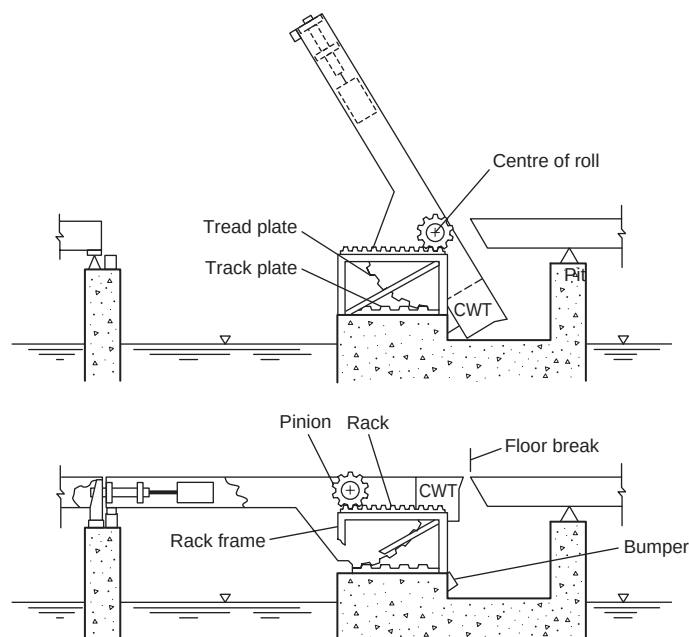


Figure 16.27 Mechanical components of single-leaf rolling bascule bridge

Treads and tracks may be castings, forgings or plates (the bearing surfaces should always be machined) that transfer the weight of the rolling mass plus live load to the bascule pier foundation. All the load is transmitted by line contact between the tread and track. The early rolling lift bridges designed by the Scherzer Company had comparatively thin treads and tracks, for initial economy. The result was that the line loading caused plastic deformation of the metal each time the leaf was rolled. In effect, the steel was being cold-rolled during bridge operation, which lengthened and spread the tread and track, but not by the same amount. The repeated plastic straining caused accumulated damage, which limited the life of the tread and track.

Because friction between the tread and the track may be insufficient to transmit the horizontal force needed to hold the span in the open position against wind and to maintain the alignment of the tread and track during operation, the track usually has upward-projecting lugs, which engage corresponding socket holes in the tread. In effect, the tread is a segment of a large gear and the track is a rack. For leaves supported by two bascule girders, both girders have tracks with lugs. For leaves with three or more bascule girders, it is usual, in the USA, for the tracks of all girders to have lugs. In Europe, usually only the exterior girders have lugs. For photographs of the tread and track of an early Scherzer bascule bridge, see Figure 2.17 in Birnstiel *et al.* (2015).

Double-leaf rolling bascules may have midspan joints similar to trunnion bascules if they are designed as double cantilevers for live load. However, it is more common that the midspan joints are Scherzer shear locks, which have no moving parts. One leaf has what are commonly called jaw details at the tips of the bascule girders and the other leaf has diaphragm details at the bascule girder tips. During closing of the span, when the leaves rotate downward and translate towards each other, the jaws and the diaphragms engage each other and provide vertical and lateral shear restraint between leaves. See Figure 3.16 in Birnstiel *et al.* (2015).

It should be noted that many Scherzer rolling bascules (especially deck bridges) were designed to act as three-hinged arches to resist live load on the forward part of the leaf. The longest of these is the Market Street Bridge, now named the John Ross Bridge, in Chattanooga, Tennessee, USA, with a 310 ft (94.5 m) span, built in 1917, and still in service (Birnstiel *et al.*, 2015).

Swing bridge

Centre bearing

Most mechanical pivot bearings are two-part bearings, bronze on steel, with a spherical interface. Some older bridges have three-part bearings, comprising a steel concave disc resting on a convex bronze lens, which, in turn, rests on a concave steel disc (Hovey, 1926). Commercial antifriction spherical rolling element thrust bearings are now also utilised as pivot bearings (Bluni and Connolly, 2004).

Centre-bearing swing bridges are also built with hydraulic bearings. Essentially, they are hydraulic jacks, which are used to raise the draw off its central support just prior to, and during, the rotation of the draw. An air-over-oil intensifier or accumulator was introduced for a swing bridge in Hamburg in about 1905. Modern hydraulic pivot bearings rely on higher-volume pumps, making accumulators unnecessary. An example of a centre-bearing swing bridge with a hydraulic pivot bearing is the Harbor Island Swing Bridge across the Duwamish River in Seattle, Washington, USA. It is a tandem bobtailed swing bridge of prestressed concrete with pivots spaced 146 m (480 ft) apart (Johnson, 2004). The pivot bearing of the bobtailed cable-stayed Samuel Beckett Bridge in Dublin, Ireland, has a fibre-reinforced (FRP) fabric layer, in order to minimise rotational friction (Cutter *et al.*, 2011).

Combined centre and rim bearing bridges may have a central pivot with a hydraulic press to reduce the load on the rim bearing during rotation. The fluid used in the early English hydraulic centre bearings was water, stored under pressure in gravity-loaded accumulators. An example of such a construction is the Barton Swing Aqueduct near Manchester, England. See Figure 12.13 in Birnstiel, 2000b).

The pivot bearings of tilting swing spans are usually mechanical steel-on-bronze sliding bearings.

Rim bearing

The superstructure of a rim bearing swing bridge usually rests on a circular girder called a drum girder. A conical plate (the tread plate) is fastened to the underside of the drum girder. It bears on a set of conical rollers, whose axes are oriented radially, intersecting with the pivot axis. The rollers, in turn, roll on a conical plate that is fastened to a chair casting. The nest of conical rollers is held concentric with the pivot axis by the live ring and horizontal radial members connected to a bearing that rotates about the pivot post. Other radial struts also connect the drum girder to another bearing at the pivot post, in order to force the draw to rotate concentrically with the rim bearing. The pivot post does not support vertical loads from the superstructure. The design intent is that all the self-weight of the superstructure be supported by the rim bearing when the bridge is in the open position. When closed, the rim bearing supports live and dead loads. Photographs of the rim bearing of the Madison Avenue Bridge in New York City are shown in Figure 2.24 of Birnstiel *et al.* (2015).

Antifriction slewing bearings developed for the military and adopted by the mobile construction crane industry are also used for rim bearings. Slewing bearings 5 m (16 ft) in diameter with three rows of rollers were used for the rim bearings of the Næstved tandem swing bridge in Denmark (Thomsen and Pedersen, 1998). The cable-stayed swing span supporting the new A63 Selby bypass over the River Ouse in England also rotates on a slewing bearing. The design concepts for the bearings of these two bridges were different. The bearings at Næstved support dead load plus live load and impact; at Selby, the slewing bearing is only subjected to dead load.

End lifts and centre lifts

The ends of swing bridges are lifted (see prior discussion on secondary movements) to accommodate the self-weight deflection of the cantilevered arms when the draw is opened. For centre and rim bearing swing spans that have structurally continuous (or semi-continuous) girders or trusses between the

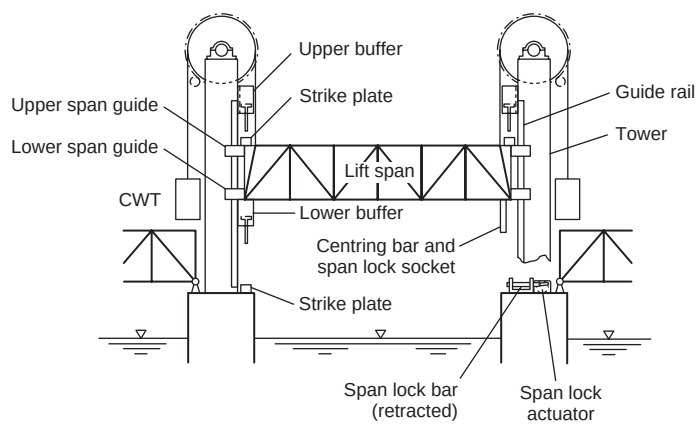


Figure 16.28 Tower drive vertical lift bridge stabilising machinery

rest piers (continuous beams over three or four supports), the draw ends are lifted so that there will be an upward reaction at the truss ends for all combinations of live load and temperature gradient. This should be so, although the truss reaction detail at the rest pier can provide only an upward reaction. Many devices have been developed to serve as end lifts, including wedges, toggle lifts, eccentric wheels and hydraulic jacks. Space is not available for further discussion of this topic.

The ends of tandem swing bridges are not lifted at closing because to do so would only tend to tilt each draw. The trusses must be designed as cantilevers for the live load. However, shear locks are provided to maintain matching elevations at the floor breaks when the bridge is open to roadway traffic (Quade, 1954).

Vertical lift bridges

Stabilising machinery for vertical lift bridges is shown in Figure 16.28. The major stabilising mechanical equipment are the wire rope, counterweight sheaves and trunnions, buffers and strike plates, guide rail and guides (sliding or rolling), live load reactions, centring devices and span locks. The buffers for vertical lift bridges are usually air-operated so as to minimise maintenance. The centring device is desirable, to align the lift span laterally during seating; it is an important and necessary feature of railroad bridges.

Prime mover and controls

Bridge control systems have evolved over the past 80 years so that today's systems bear little resemblance to those designed years ago. New methods of speed control for the span drive motors and solid state technology for the control system are now available. They are adaptations of advances made in other industrial fields, such as crane control and process technology. However, before describing these advances, bridge control design during the past 80 years will be reviewed.

Background for electrical system

As explained previously, the three major types of movable bridge are the bascule, swing and vertical lift bridges. The choice of bridge type was made early in the design phase and was influenced by many factors, such as use, required clearances and cost. For each type of bridge, the choice of electrical equipment was fairly limited. The prime mover was either a direct current (DC) motor or an alternating current (AC) wound rotor motor because the torque of these motors could be controlled. The control system was primitive, consisting of relays and drum switches. While the choices were limited, simplicity of design was advantageous, and many of these bridges are still operating. Basic electrical design was left to the direction of the individual designer, and there were no national codes

until the American Railway Engineering Association (AREA) and the American Association of State Highway Officials (AASHTO) published the first set of recommended practice and specifications for movable bridge design in the early part of the twentieth century. These codes have evolved over the years so that they are a standard reference for most bridge projects in the USA and other countries. The codes recommend minimum design requirements, with the designer given latitude to introduce additional features.

When considering the electrical part of movable bridge design, there are four topics of importance: the prime mover, interlocking, safety and auxiliary devices. The most important is the prime mover. For many years, DC motors were the motors of choice. Their speed could be regulated; they had the ability to resist an overhauling torque and could be built for use in harsh environments. Prior to the advent of solid state devices, such as thyristors and silicon controlled rectifiers (SCRs), power conversion for the standard three-phase AC line found throughout the USA was through a motor generator set that converted the incoming AC voltage to DC by means of a generator. The disadvantage of using a motor generator set was its size, required maintenance and normal wear on the equipment. However, for many years this was an accepted cost, in order to obtain good speed control of the DC motor.

With the advent of solid state devices, such as the thyristor and SCR, speed control could be achieved by converting the three-phase supply directly to DC without the use of a motor generator set. A three-phase power rectifier was used, and control of the output was through a control gate that prevented reverse current, and limited forward current, until the gate was turned on. This control gate regulated the voltage available and thus the speed of the motor.

Wound rotor motor

The motor prevalent in bridge designs throughout the second half of the twentieth century was the AC wound rotor type. It was favoured by designers because its speed–torque curves can be shaped using fixed resistors to meet the torque requirements of the application. The combination of lower equipment cost and simplicity of electrical control design made the wound rotor motor the motor of choice over the DC motor.

Figure 16.29 is a composite of various speed–torque curves available to the designer for selecting resistors to match the required torque. As can be seen, the value of the resistor (in ohms) influences the amount of torque that the wound rotor motor can produce. These secondary resistors, when properly placed in the rotor circuit, can produce a reasonably shaped speed–torque curve.

One of the most common methods of controlling the resistance of the secondary windings of the wound rotor motor was through the use of a drum controller or master switch. The master switch would step or short out successive stages of resistance, allowing more motor torque as the resistance was removed from the rotor circuit until the desired torque was

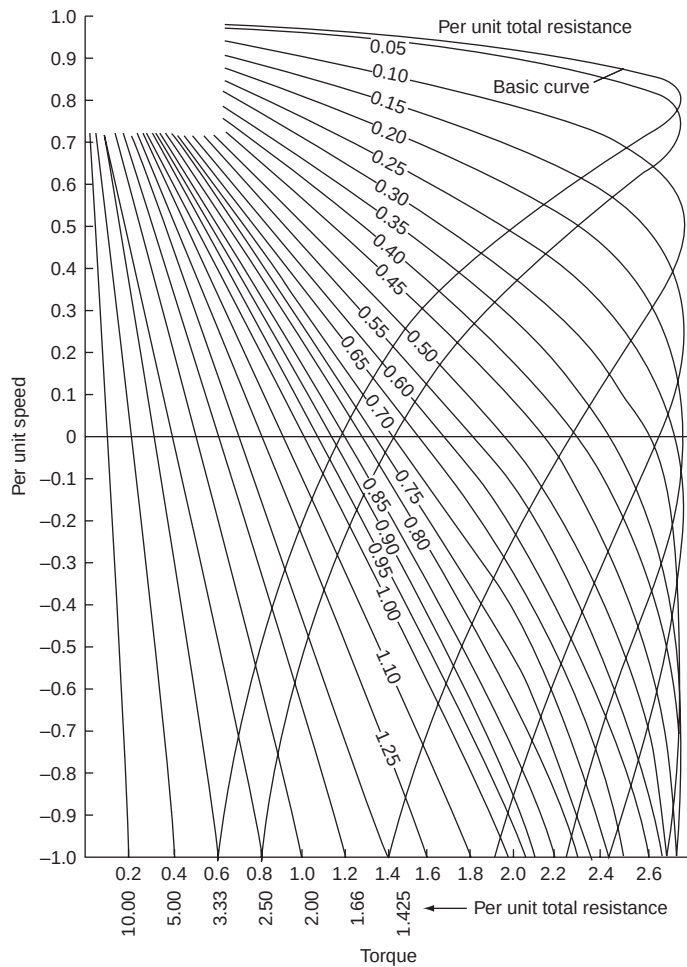


Figure 16.29 Wound rotor motor speed–torque curves (courtesy of Link Control Systems)

achieved. It was not uncommon for a drum switch to have five or six power points. The first power point limited the starting torque of the motor to about 50% of the rated motor torque. The successive points would short out the next stage of resistance and increase the available torque from the motor. The next stage would increase torque even further until the last stage was reached.

During normal operation, the torque developed by the motor was below normal maximum and the speed was above the speed that would be expected at the breakdown torque (Heumann, 1961). By moving the handle on the drum switch to higher controller points, the operator can switch the value of the total resistance, while keeping the value of available torque below the optimum level (Borden, 1996).

Figure 16.30 depicts the end result of the designer's quest. Notice how the torque applied is in a fairly constant range, in relation to speed. As Borden (1996) states, machinery designers should note that there is no explicit way to limit torque other than by using instantaneous overload relays in the power supply circuit. A further problem is that motor control is under the direct control of the bridge operator.

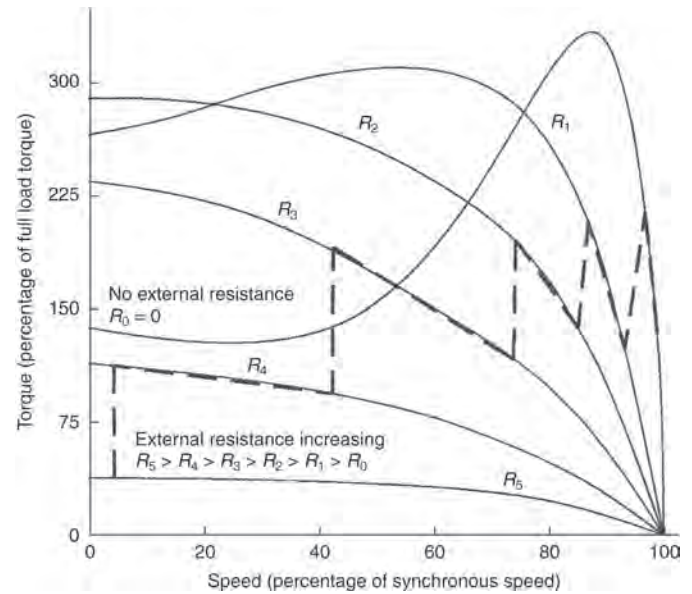


Figure 16.30 Effect of stepping secondary resistance of a wound rotor motor (courtesy of Link Control Systems)

Why the concern with torque? The machinery has a finite torque requirement, as well as a maximum allowable torque rating. By the standards of the National Electrical Manufacturers Association (NEMA), a wound rotor motor is capable of delivering a minimum of 275% of rated torque. The connecting shafts, reducers, trunnions and other mechanical items may not have a rating that will accommodate this high torque, which some regard as excessive for bridge machinery. Trade associations, such as the American Gear Manufacturers Association (AGMA), do not require manufacturers to use a maximum torque input greater than 200% in stress calculations. It appears that the bridge code writing bodies do not recognise that importance of the difference between the electrical and mechanical applications.

Thyristor drive

Among the solid state designs that appeared in the late 1950s was the thyristor or power SCR. This device was adapted to the crane and bridge industry. Its benefit was that the motor voltage could be varied and, with a fixed secondary resistance in the rotor circuit, could control a wound rotor motor. It limited the maximum torque available at the motor through current limiting.

While not perfect in limiting torque, the design of the thyristor drive was an advancement in the industry. The thyristor drive, when used with a combination of a tachometer generator and secondary resistance in a closed-loop system, could effectively control the dynamics of the span. Various speed points, as well as torque, could be maintained; for example, creep speed at a maximum of 80% torque for seating the span. This application is still in use today.

Flux vector drive

The latest innovation in the bridge industry is the flux vector drive. Through the use of digital electronics, a precise waveform can be generated, which will control a normal squirrel cage motor that has been rated for flux vector duty. The motor used in this application is usually a totally enclosed non-ventilated (TENV) design that has a digital encoder mounted on the shaft. The encoder monitors shaft speed and position and sends a feedback signal to the flux vector drive. On the basis of the feedback signal, the drive regulates the current and phase angle by utilising firmware algorithms designed for the particular span dynamics. The flux vector drive and motor combination is capable of producing rated torque from zero speed to twice base speed.

As for the primary thyristor drive and wound rotor motor, the combination of both is capable of AC line regeneration. This regeneration acts as a magnetic brake when the span is moving faster than the programmed command signal. If the span drive motor is moving faster than the governing signal, an over-speed condition is sensed by the drive regulator and the drive controller fires a reverse signal to slow the motor. With a flux vector drive, the regeneration is through a dynamic braking resistor.

Hydraulic radial piston motor

The use of low-speed high-torque hydraulic motors for bridge drives is not common in the USA, although not unusual elsewhere. Selection of the motor is based on the maximum torque and speed required for the application. The most common type is the radial piston design because it can produce high torque at low speeds. If the motor is operated at a constant pressure, the torque output should be almost constant from full speed to creep (Wendel, 1996). The internal features of a hydraulic radial piston motor are shown in Figure 16.21. See also Birnstiel *et al.* (2015).

Hydraulic valves

Hydraulic control valves are used to modulate the fluid passing through them. Flow valves adjust the volume of flow, directional valves control the path and regulating valves control the pressure. These valves are normally electrically operated and are proportional in that they allow an external adjustment to the valve parameter based on the amount of voltage or current passing through the valve coil. The proportional valve is controlled by a solid state control board that usually consists of an amplifier, ramp function generator and other specialty controls. The ramp function generator varies the rate of change to the valve coil, so that an almost linear rate of change can be achieved. This is useful in controlling the acceleration and deceleration of the span. The amplifier may have preprogrammed set points, allowing for selection of set speed points or pressure commands. See also Birnstiel *et al.* (2015).

Auxiliary devices

Other electrical devices are required for movable span operation. Chief among these are traffic control devices for the protection of vehicles and pedestrians, traffic lights, warning gates and barriers (resistance gates). The design goal is to prevent physical access to the movable part of the bridge. A traffic light is not sufficient for stopping people and vehicles. Warning gates that straddle the roadway usually prevent automobiles from entering the movable span. However, for vertical lift bridges, swing bridges and certain bascule bridges, the addition of barrier gates or bollards that physically prevent an automobile or truck ingress to the danger area is required. The barrier is designed to stop a vehicle travelling at a certain speed specified by the bridge owner.

Another auxiliary device is the span lock. Its purpose is to prevent opening movement of the bascule or vertical lift span in the bridge-closed position. There are two versions, mechanical and structural. The structural type is used on some rolling bascules and consists of a jaw and diaphragm. When the leaves are lowered, the jaw and diaphragm are mated and can transfer shear. See Figure 3.16 in Birnstiel *et al.* (2015). The mechanical span lock consists of a lock bar driven into a mating socket. The driving action can be by means of a hydraulic actuator or an electric motor-driven reducer, as depicted in Figure 16.26.

Additional auxiliary devices are needed for swing bridges, for example, end lifts, centring devices and wedges (see prior sections). End lifts perform a function of supporting the ends of the draw when it is in the closed position. When the draw is to be opened, the end lifts are withdrawn so that the draw ends are unsupported. Centring devices are normally mechanical arms that lock in place when the swing span has reached the fully closed position. Centre wedges are driven to support the live load on centre-bearing swing bridges. Centring devices are also necessary on vertical lift bridges, in order to match roadway joints on road bridges when the lift span seats and to align the ends of the movable and fixed running rails of railroad bridges.

Brakes are used on movable bridges, either to stop the span or to keep it in position. Usually, they are either the spring-loaded drum or disc type, with an electric or electrohydraulic release actuator. The actuator has a built-in adjustable time delay so that it releases almost instantaneously, but when power is removed and the brake starts to set, the shoes or pads require about 5 s to fully engage the drum or disc. The reason for this is that the designer wants to avoid shocks to the machinery at braking. Normally, there is a brake for each motor shaft, as well as machinery brakes on the low-speed, higher torque, shafts. Position limit switches monitor the status of the brake thruster position, as well as a manual hand release.

Most maritime jurisdictions require marine notification of span position. This is accomplished through the use of navigation lights, either at the tip of the movable span or in the centre. A combination of red and green lights signal the position of the span. The red light stays illuminated from fully closed until fully open is reached, at which time the light changes from

red to green. In addition, pier lights are used to illuminate the various fenders and piers that project into the waterway.

Normally, a bridge must be opened when called on to do so by a navigator, even in the event of a utility power outage. Alternative means of accomplishing this are part of the design criteria. In many cases, a redundant source of power, such as an alternative utility feed or a generator is installed. These backup sources are controlled by a transfer switch, which, in most cases, is automatic. A loss of voltage on the normal feed is sensed, and a signal is generated to either start the backup generator and switch to it or immediately switch to the alternate feed. This allows the operator to continue operation of the bridge. The control system should be designed so that when normal power returns, the retransfer is not automatic, to prevent a loss of power during bridge operation.

Interlocking

There are various interlocks throughout the control system to prevent unintended operation or where there is a safety hazard to personnel. The goal is to avoid operator error.

Traffic signal interlocks

Each of the warning gates and barrier gates has limit switch contacts that will turn the traffic signals to red, turn on the gate warning lights and sound the traffic gongs if the gate is not in the full up position. This circuit should be independent of the bridge control power switch. If the traffic signal is red and the traffic signal selector switch is in the green position, a gate has moved from the full up position.

Hand crank interlocks

The warning and barrier gates, the span locks and end lifts or wedges, if used, should have limit switches that will open the control circuit, preventing electrical operation if the hand crank has been activated.

Gate door interlocks

The warning and barrier gates have door limit switches that sense when a gate door is open. This switch opens the starter control circuit to prevent electrical operation.

Barrier gate interlocks

The barrier gates cannot be operated electrically if the traffic light is not red, the warning gates are not down, the span locks are not driven and the span is not seated.

Warning gate interlocks

The warning gates are interlocked with the red traffic signal and the barrier gates in the open position. They are also cross-

interlocked in that the off-going gates cannot be lowered if the oncoming gates are not down. In addition, the oncoming gates cannot be raised if the off-going gates are not in the up position.

Brake interlocks

The interlocks of the thruster brakes in the machinery room are arranged so that the lift span cannot be operated electrically if more than one brake is hand released. Use of the brake bypass switch will allow all brakes to be hand released.

Span operation

The span cannot be operated electrically unless all warning gates are down, barrier gates are closed and the lock bars are withdrawn.

Main drive motors

It is not possible to change the main drive motors during span movement.

Overtravel

If the span moves past the full open position, an overtravel limit switch will trip. This causes an emergency stop. The span can only be closed from this position.

Emergency stop

There are two emergency stop buttons in each tower located on the motor control centre (MCC) and near the machinery. These buttons, when pressed, will stop all operations of the span. A red light in the pushbutton will indicate when the button has been depressed and a message, as well as an indication, will come up on the human-machine interface (HMI) display. The button must be pulled out to reset the system. If in normal mode, the drives may have to be reset.

Method of control

The bridge industry has evolved from using a relay-based control system to using a programmable logic controller (PLC) based system. While there are distinct advantages to the PLC system, the simplicity of a relay system should not be overlooked when establishing the design criteria. The first step is to ask the owner who will be maintaining the completed bridge. Often, although much money is spent on the design and construction of the bridge, little thought is given to the ongoing maintenance of the bridge once it is operational. Are the owner's maintenance people capable of diagnosing and maintaining the sophisticated installation? Have they been properly trained? Do they have the requisite spare parts? Will the parts be available in 10 or 20 years when the now installed parts fail? Answers to these questions help shape the course of design for the electrical engineer.

While much can be said for the current state of electrical design, a new layer of complexity has been introduced to the bridge systems. Control systems are now capable of design elements that were not available to the industry 25 years ago. Programmable logic controllers, solid state drives, and the next generation of positioning devices have led to a level of complexity that was unknown heretofore. The benefits of this technology are more interlocking safety, data logging, less stress on the machinery and more precise positioning of the span. The problems introduced are complexity of design, replacement of functionally obsolete parts and the need for increased maintenance training.

Significant movable bridges

La Porta d'Europa bascule bridge

The Barcelona Bridge is a double-leaf simple trunnion bascule bridge with a span between trunnion bearings of 109 m, giving a clear waterway opening of 100 m between the faces of the bascule piers (Arenas de Pablo, 2000; Salcedo, 2003). As depicted in Figure 16.31, the leaves are stayed to resist gravity and wind action, the stays being rigid box members that can resist compression owing to wind and earthquakes when the leaves are not closed. Each leaf is 18 m wide and 68.5 m long, including the tail, with a counterweight on the underside of the 14 m long portion of the leaf rearward of the trunnion axis.



Figure 16.31 La Porta d'Europa bascule bridge (image courtesy of Arenas Associates)

The trunnions are cantilevered outward from the longitudinal girders (Hopkins type trunnions), with one spherical bearing per trunnion. The span drive is of the linear action type, with two hydraulic cylinders per leaf, although a single cylinder can move the leaf against the maximum wind load at reduced speed. Each cylinder has a capacity of 485 t and a 5.5 m stroke. The concept design was by Professor Juan Jose Arenas, the structural design by L Vinuela and JM Salcedo of FCC Construcción and the machinery design by NOELL.

Halsskov Bridge

Rolling bascules have been popular in Europe for medium spans, and much attention has been paid to the aesthetic aspects. Figure 16.32 depicts the rolling bascule across the inner harbour between Halsskov and Korsør on the western side of Sjælland, Denmark. It is a half-through single-leaf rolling bascule of 30.89 m span that supports a roadway and a single railroad track. The 20.80 m wide deck is of steel orthotropic plate construction and the truss members are enclosed welded boxes. There is no lateral bracing at the top chords and the bridge has a very clean, open appearance.



Figure 16.32 Halsskov Bridge (photograph courtesy of C Birnstiel)

The leaf is operated by hydraulic cylinders, one on each side of the bridge, alongside and below the track. This is an unusual configuration, which was driven by architectural considerations. The pistons act on brackets that project downward alongside the trusses near the front teeth. To open the bridge, the cylinders push forward, opposite to the direction of roll. The radius of roll is 6.40 m. More information about the operation of this bridge may be found in Parke and Hewson (2008).

Ennerdale Link Bridges

The Ennerdale Link Bridges across the River Hull on the edge of Kingston upon Hull, in East Yorkshire, England, were completed in 1997. As is evident from Figure 16.33, they comprise two adjacent units of the balance beam bascule type. Each leaf is 13 m wide by 33 m long and weighs 270 t. An overhead balance frame, with the connecting rods at about the 0.7 point of the leaf span, balances the deck. The machinery and controls are very sophisticated, with many degrees of redundancy. Each bridge unit has four double acting cylinders, located shoreward and below the heel trunnions, which act on brackets extending downward from the leaf. Normally, the bridge utilises all four cylinders. Each cylinder is about 400 mm in diameter, with a stroke of 3.5 m, and can exert a force of 200 t at a maximum operating pressure of 200 bar. The normal leaf opening angle is 60° (which is achieved in 1 min), although it can be operated to a full 89° open position.



Figure 16.33 Ennerdale Link Bridges (photograph courtesy of C Birnstiel)

The control system is a very sophisticated PLC (programmable logic controller) system, with extensive use of sensors and other instrumentation. Data logging is performed for every bridge operation. A computer displays bridge parameters and messages in graphic form for the use of bridge operators, but the bridges are also operable without the graphics. A program in this computer also enables engineering analysis of the hydraulic equipment and displays of the fluid power system state. The use of computers to control and diagnose hydraulically operated movable bridges was brought to a high technical level during the construction of this bridge.

Næstved swing bridge

This tandem bobtail swing bridge was built across the Næstved Canal near Næstved, Denmark. It crosses the channel at an angle of about 80° (Thomsen and Pedersen, 1998). The superstructure comprises a welded steel box with cantilevered floor beams supporting a steel orthotropic deck. Each draw is 49 m long and 14 m wide, shaped as a parallelogram, with the ends parallel to the channel. The distance between pivots is 56.86 m along the longitudinal axis of the bridge. The draws are counterweighted to minimise dead load bending in the top of the pivot piers. Figure 16.34 shows one draw in the open position.

Each draw is supported at the pivot pier by a standard roller slewing bearing of 5.0 m diameter, having three rows of rollers. The bearing supports actions on the draw: self-weight,



Figure 16.34 Næstved Swing Bridge (image courtesy of the Danish Road Directorate)

live load, impact, wind, and so on. The inner periphery of the ring bearing has gear teeth (it is a circular rack), which re engaged by the span drive pinions (which are driven by low-speed, high-torque hydraulic motors through planetary speed reducers). Figure 16.35 shows the internal rack on the upper part of the slewing bearing, a pinion, reduction gearing and a hydraulic motor.



Figure 16.35 Drive motor inside pivot pier of Næstved Swing Bridge (image courtesy of the Danish Road Directorate)

Selby Swing Bridge

The A63 Selby bypass crosses the River Ouse in England on a cable-stayed bobtailed swing bridge having a draw with overall dimensions of 95.2 m by 17.96 m. The main span from the pivot pier to the north landing pier is 55.0 m and the back span is 40.0 m. It comprises two welded girders with an orthotropic deck, assisted by stays. Figure 16.36 is a view from the north bank of the River Ouse.



Figure 16.36 Selby Swing Bridge (photograph courtesy of R Evans)

This bridge is of the lower or lift-then-turn type, with a slewing pivot bearing that is only loaded with the self-weight of the draw during turning. This design feature is in contrast to the idea of the Næstved Swing Bridge, just described. When the

draw is in the fully closed position, ready for highway traffic, the bearing is unloaded. Referring to Figure 16.37, the bridge is supported in the closed position (with roadway traffic) by six pot bearings; M, C and S. To account for dimensional changes, owing to variation in ambient temperature, the bearings M and S are mounted on retractable roller carriages. The C bearings are fixed to the underside of the bridge girder. Briefly, the operation to clear the channel (open the bridge) is as follows.

- Raise draw on hydraulic jacks, J, to unload bearings, M, C and S. Vertical clearance is required so that roller bearings M and S and plates under C can be retracted.
- Lower draw so that it is only supported on bearings, L, which are mounted on the top of the transfer structure.
- The draw weight is now on the slewing bearing. Rotate the transfer structure (with bridge resting thereon) as required.

To close the bridge, the procedure is reversed.

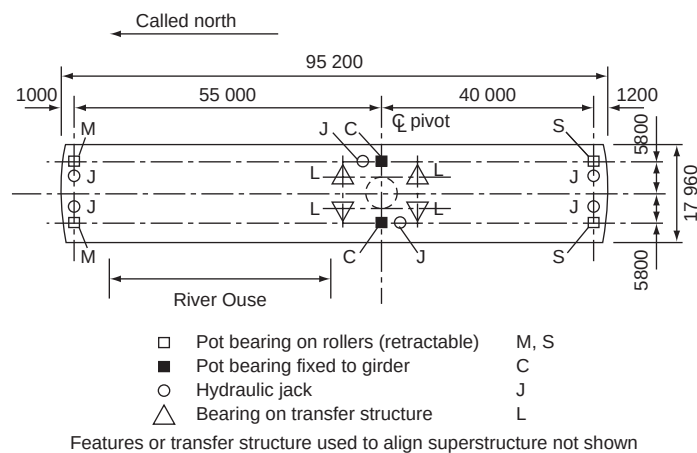


Figure 16.37 Supports for draw of Selby Swing Bridge (dimensions in mm)

The span drive of Selby Bridge comprises six electric motors that rotate pinions engaging a large-diameter ring gear (rack), which is integral with the slewing bearing, through epicyclic gear speed reducers. Normally, the draw is retarded by regenerative braking but a disc brake is provided on each motor for emergency stops and parking at other than the fully closed and fully opened positions. These brakes are spring-applied and electrically released (by electromagnets). The control system ensures that the brakes are electrically released when the motors are energised. A gear-motor brake is shown in Figure 16.38. The foregoing description is based on Kvaerner Markham (2000) and drawings provided by the Highways Agency, Skanska Construction UK and Alan Greaves, bridge manager of the North Yorkshire County Council.

Marine Parkway–Gil Hodges Memorial Bridge

The Marine Parkway–Gil Hodges Memorial Bridge depicted in Figure 16.39 is a tower drive vertical lift bridge with a lift



Figure 16.38 Drive motors at Selby Swing Bridge (photograph courtesy of R Evans)

span 540 ft (165 m) long that can be raised 95 ft (29 m). It was completed in 1937 as the longest-span vertical lift highway bridge. When opened to road traffic, the bridge had two lanes of traffic in each direction and a walkway between the two trusses. Span drive machinery is located atop each tower. The control and switchboard rooms are located in one tower, above the walkway and roadway.



Figure 16.39 Marine Parkway–Gil Hodges Memorial Bridge (photograph courtesy of Hardesty & Hanover, LLP)

The lift span is connected to the main counterweights by 40 main counterweight wire ropes of 2½ in (60 mm) diameter at each tower, ten ropes passing over each of the four counterweight sheaves at that tower (Hardesty *et al.*, 1975). Auxiliary counterweights were provided to offset the shift in balance as the main counterweight ropes move from one side of the counterweight sheaves to the other during operation.

There were two, virtually identical, span drives on this bridge, one atop each tower, mechanically independent of each other. The main motors kept the lift span level longitudinally by means of position resolvers. Electric coupling existed between the two emergency span drives through the use of power synchrotie motors. Each span drive had two main and two emergency drive motors. They were flux vector wound rotor motors operating at 480V, three-phase AC, 60Hz. The main motors are 200hp (149kW) and the emergency motors are 50hp (37kW), both rotate at approximately 850rpm full load speed. This system is being replaced at the time of writing (2019).

Power was transmitted from the main or emergency motors through a primary differential reduction gearset. It contained a clutch, which permitted the differential feature to be locked during normal operation. Each transverse output shaft from the primary reduction gearset led to a secondary differential reduction gearset, which provided equal torques to the two rack pinion shafts that it drove. The pinions engaged circular racks, mounted on the counterweight sheaves, which transmitted the power to the main counterweight ropes via friction. This system is being replaced at the time of writing (2019).

After 60 years of service, a structural inspection disclosed corrosive damage to the bridge decking, the steel framing below it and the lower chords of the trusses. There was also a need for improvements to facilitate traffic movement and safety. Accordingly, Hardesty & Hanover were engaged in 1998 to prepare plans for a structural renovation of the truss spans. The extent of the major structural renovation, which included relocating the walkway outboard of the westerly trusses is thoroughly described in Nyman (2003).

A major electrical and mechanical rehabilitation is still underway at the time of writing (2019); this includes: replacement of all lifting machinery, span guides, PLC control system, main and auxiliary drives and control consoles.

New Jersey Route 13 Bridge

New Jersey Route 13 is supported over the Point Pleasant Canal, a component of the Intracoastal Waterway, by a connected tower drive vertical lift bridge. The span drive machinery is mounted on a platform spanning the towers and drives all four counterweight sheaves simultaneously to raise or lower the lift span. Figure 16.40 is a view of the bridge from the southwest canal bank, with the lift span partially raised. The bridge was built in 1970 to replace a prior swing bridge put into operation in 1924 on opening the canal to navigation. The towers are spaced 92 ft (28 m) apart. They support a roadway with four lanes of traffic. The normal lift is 35 ft (12 m).



Figure 16.40 New Jersey Route 13 Bridge (photograph courtesy of C Birnstiel)

Each corner of the lift span is suspended from six wire ropes of 1.5 in (38 mm) diameter that are looped over cast steel sheaves mounted atop the corresponding columns of the superstructure. The other ends of the ropes are connected to the counterweights (one counterweight for each end of the lift span). Raising and lowering of the span is accomplished by rotating the sheaves using the span drive machinery mounted atop the platform, which spans between the tower legs. All the sheaves are directly connected by gearing without any differential so that, theoretically, the wire rope movements are alike at all sheaves, provided that the ropes do not slip on the sheaves or stretch unequally.

The span drive is powered by two 60hp, 460V AC, wound rotor motors. Only one motor drives at a time; the other motor rotates, de-energised. A spring-set thruster-released brake is mounted at each motor. Both motor shafts are coupled to the primary speed reducer input shaft by brakewheel-type gear couplings. The primary speed reducer has a 2:1 gear ratio and a double extended output shaft. There is no differential in this speed reducer. Each output shaft extension is connected by a longitudinal series of floating shafts to a secondary right-angle reducer with a gear ratio of 7.9:1. The secondary reducers have double extended shafts and the transverse shafts coupled to them transmit power from the secondary reducers to the pinion shafts, which engage the curved racks mounted on the counterweight sheaves.

To permit vertical adjustment of a set of counterweight ropes at one corner of the lift span with respect to the other corners, 'adjustment couplings' were installed on the output shafts of the secondary reducers. These are gear couplings with a special circumferential spacing of flange bolts. To make adjustments, the flange bolts are removed from an adjustable coupling and the hubs rotated with respect to each other by the desired amount. The flange bolts are then reinstalled. The machinery of the Route 13 bridge is typical of connected tower drive vertical lifts of the period.

Johnson Street Bridge

On 31 March 2018, the replacement Johnson Street Bridge was opened to traffic between the core of Victoria, BC, Canada, and the Township of Esquimalt. It is a single-leaf cradle-bearing bascule of 53.4m span from the toe bearing on the rest pier to the centre of rotation of the roller bearing, providing 41.7m of navigational clearance (Griesing and Mileo, 2019). The central deck has three travel lanes and two bicycle lanes, and there are two outboard multiuse decks. Project delivery was by a blend of design-bid-build (DBB) and design-build (DB) contracts. WilkinsonEyre provided the conceptual or indicative design. Hardesty & Hanover engineered the movable part of the bridge and WSP was the prime consultant to the City of Victoria. The contractor was PCL Construction Westwood of Richmond, BC, Canada.

The WilkinsonEyre bridge concept was similar to that of Cowing's 1899 US patent for a cradle bearing, except that it varied in one important way. It provided a reliable means for equalising the loads on the rollers. As shown in Figure 16.41, there are two structural rings, the ends of the two main trusses. Each ring is supported by 12 rollers, mounted in an equalising arrangement. In the figure, the two central rollers are hidden by the hydraulic drive motors. Note that the truss rings do not bear

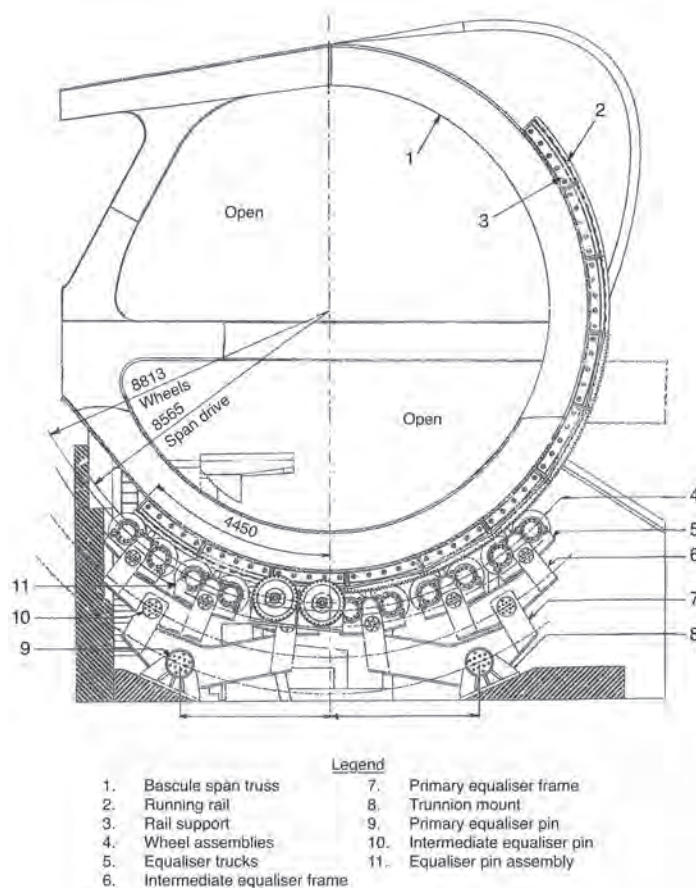


Figure 16.41 Johnson Street Bridge – cradle bearing

directly on the rollers. Instead, there is an assembly of running rail and span support segments, which transfer the loads from the truss ring to the rollers. The roller support system transfers approximately 3000t of span and counterweight to the bascule pier floor.

Gateshead Millennium Bridge

The Gateshead Millennium Bridge across the River Tyne near Newcastle-on-Tyne, UK, was the first gyratory bridge. As is evident from Figures 16.42 and 16.43, it is a pedestrian bridge with a movable deck that is close to the water in the closed position. In plan, the deck is symmetrically curved with a central offset of about one-third of the 105m span. This shallow box deck is supported by cables from a backwardly inclined arch. In the closed position, the deck is inclined; because of its pronounced horizontal curvature, a rotation of approximately 40° about the longitudinal axis is sufficient to create a 25m vertical waterway clearance at midspan. A large trunnion is connected to each springing of the arch. The trunnion axes are collinear. These trunnions are mounted on massive piers, which also



Figure 16.42 Gateshead Millennium Bridge – closed position (image courtesy of Wilkinson Eyre Architects)



Figure 16.43 Gateshead Millennium Bridge – open position (image courtesy of Wilkinson Eyre Architects)

house the operating machinery. Hydraulic cylinders at each trunnion rotate the leaf. Because there is no mechanical machinery connection between the trunnions on opposite sides of the channel, the strokes of the pistons must be closely matched by electrohydraulic controls. If the springing of one arch were to rotate without a corresponding rotation of the other springing, the arch (and deck) would be twisted. WilkinsonEyre were the architects, Gifford and Partners the structural engineers and Bennett Associates engineered the mechanisms.

Hörn footbridge

The concept of a bridge deck that folds about one or more transverse horizontal hinges to clear a waterway is not new. Bridges based on this concept were built for street traffic in Chicago, Illinois, USA, and Milwaukee, Wisconsin, USA more than a century ago (Birnstiel, 2001). A modern application of this concept is the recently erected Hörn Bridge in Kiel, Germany. Figure 16.44 was taken during its operation. The single multi-part leaf spans 25.6 m. In the closed position, the bridge is a single-span cable-stayed bridge with discontinuous girders. At each discontinuity, there is a hinge about which the deck folds. The bridge crosses Kiel Harbour near the railroad station and is heavily crossed during the summer. Witnessing the operation of this bridge does provide pleasure for the tourists whose passage over the bridge is interrupted by its opening. The architects were von Gerkan, Marg and Partners. Schlaich, Bergermann and Partners were responsible for the structural design and the mechanical concept.



Figure 16.44 Hörn Bridge – partially open position (photograph courtesy of J Baetke)

Movable bridge design

Design philosophy

Three notions current in engineering design philosophy for combined structural, mechanical and electrical systems are: safe life, fail-safe and damage-tolerant design.

Safe life design is used for systems that are difficult to repair or whose failure could cause personal injury or costly physical damage. *Fail-safe* design recognises that all physical systems have flaws; hence, redundancies are designed into the system so that the negative consequences of failure are minimised. Fail-safe design is at a level above safe life, usually at increased cost. *Damage-tolerant* design, with its concomitant inspection and maintenance programme, recognises that all physical systems have flaws and that they propagate with time, but that the faults can be managed with the aid of a maintenance programme, to minimise the risk of catastrophic damage. The decision to adopt damage-tolerant design requires simultaneous serious commitment to long-term physical hands-on inspection of the machine or structure, an expensive proposition. See Chapters 4 (on non-destructive evaluation methods) and 16 (on design considerations for aircraft) of Grant (2004); these methods are, by extension, relevant to other structures.

Bridge design standards and recommended practice are based on safe life philosophy. Implicitly, ‘life’ meant a long time, with appropriate maintenance. However, all bridges have flaws, and some flaws are exacerbated during service. In such cases, if it is necessary to maintain traffic on the bridge, or to maintain operation, a damage-tolerant design approach is temporarily required until the damage is repaired. It may be desirable to put limits on the traffic crossing the bridge, to expedite repairs.

Design codes, specifications and recommended practice

Movable bridge design is multidisciplinary, involving the coordinated application of structural, mechanical and electrical engineering. It is an art that utilises solid and fluid engineering mechanics and electrical physics. Like any art, it is based on experience. This experience is recorded in voluntarily developed design specifications and recommended practice in the technical literature. The first comprehensive design specification for movable bridges appears to be that presented by CC Schneider to a meeting of the American Society of Civil Engineers (ASCE) in 1907 (Schneider, 1908). Schneider wrote mostly about swing bridges, the most popular movable bridge at the time. He promoted the centre-bearing type. A specification oriented to bascule and vertical lift bridges was suggested by Leffler (1913). The later recommended practice for movable bridges of the American Railway Engineering Association (AREA) and the specification of the American Association of State Highway and Transportation Officials (AASHTO) were based on the Schneider and Leffler specifications. Current (2019) versions of movable bridge specifications are based on both allowable stress and limit state design (load and resistance factor design (LRFD) in US terminology). The American Railway Engineering and Maintenance-of-Way Association (AREMA, 2019) recommended practice is based on allowable stress. The American Association of State Highway and Transportation Officials (AASHTO) specification in force is

an LRFD version dated 2007 with interim revisions published until 2018. Germany does not have a movable bridge specification, as such, but uses a limit state design specification promulgated for hydraulic structures (DIN, 2005), which is available in English. The Netherlands has issued a semi-probabilistic design specification (NEN, 2017). It is not available in English, except that it also has an English title: *Rules for the design of movable parts of civil structures – Part 1: Movable bridges*. This standard concentrates on technical regulations for the design of the mechanical equipment and functional requirements for the design of the electrical systems. It references NEN 6788 (NEN, 2009) for design aspects of the steel superstructure.

France does not have regulations specific to movable bridges. In practice, the *Rules for the Design of Hoisting Appliances* issued by the Fédération Européenne de la Manutention (FEM), as applicable, are followed. They are published in English, French and German. The current edition is FEM 1.001 (1998).

The design should also address ‘green’ considerations (Snelling, 2009) and climate change (Shields, 2011, 2016).

Peer review of bridge design

The recent conversation about peer reviewing of bridge engineer-of-record (BER) designs was prompted by the collapse of a portion of the Florida International University pedestrian bridge onto US Route 41 on 15 March, 2018. The collapse resulted in six deaths and ten injuries, one a lifelong disability. The part of the bridge that fell was an enormous statically determinate post-tensioned concrete truss. It was a 320 ft (98 m) long I-shaped cross-section with a single Warren-style web. The collapsed truss weight was estimated to be 860t. It was assembled adjacent to the highway and turned and rolled onto the intended permanent piers on 10 March, 2018. The design for the project was said to have been overseen by the university, not by the Florida Department of Transportation (FDOT), an unconventional practice. Because a worker was killed during the accident, the Occupational Safety and Health Administration (OSHA) of the US Department of Labor investigated the failure. The OSHA concluded that the engineer-of-record (EOR) made mistakes in the design calculations and that the peer review (PR) Engineer did not find the mistakes because he did not review the computations of the EOR for the pertinent construction stage.

The National Transportation Safety Board (NTSB) released a report on 12 November, 2019, with conclusions agreeing with those of OSHA (Judy, 2019; Sholli, 2019).

The matter of legally required peer reviews for bridge designs is unclear in many countries. In the UK and Ireland, there are no national regulations regarding peer reviews, but the agencies that build major highway and railroad bridges require them. It is not clear whether they are required by smaller owners or how the peer review is selected and compensated. In the USA, there are no federal requirements for independent PR,

but some states have requirements, as shown for Florida. Some of the major bridge engineering firms will only undertake a design job if the owner engages independent peer review of the bridge design. In Germany, by law, all bridge designs are peer reviewed. A uniform practice for peer review of bridge designs should be established. Ian Firth has lectured on engineering peer reviews of bridge designs (Firth, 2007, 2010).

Movable bridge machinery analysis and design is based on elementary mechanics, except for the modern electrical and electronic controls. Many references are available for design. For information about structural analysis and design, see previous chapters of this book, Ghali and Neville (1977), Michalos and Wilson (1965) and McGuire *et al.* (2000). For mechanical analysis and design see Shigley and Mischke (2001), Spotts (1985) and Spotts *et al.* (2003). Elementary fluid power is covered by Frankenfeld (1984). Fluid power circuit simulation is treated by Akers *et al.* (2006).

However, a successful design requires that the engineer recognises the interrelationship between the machinery and the structure. The structural action of a movable bridge is usually different in the closed and open positions. The mechanical–structural interaction should be considered, especially with regard to displacements and distortions. The machinery needs to be simple and rugged because maintenance of movable bridges is not a priority for some owners. Overpowering or overbraking of bridges should be avoided. Many machinery failures, especially on swing bridges, were due to improperly adjusted brakes having excessive braking torque capacity (Birstiel, 1990a).

The electrical system for a movable bridge should provide for acceleration and deceleration of the span consistent with the mechanical or hydraulic machinery design. But the system involves more than speed control of span drive motors. Power and control are required for auxiliary devices, such as span locks, traffic warning and resistance gates, traffic signals, rail locks and catenary lifts. All these controls must be properly sequenced and interlocked for the safety of land and water traffic.

A movable bridge is a machine. Unless there is design experience for a structure or machine of similar type and scale, a general dynamic analysis of the structural–machinery system for a particular project may be desirable or necessary. Dynamic effects that might be negligible for a particular type and size of bridge may be important for a similar structure of larger overall dimensions or more flexible construction. The events at the London Millennium Footbridge, over the Thames, might be cited as an example.

Some other important design considerations are as follows.

- *Accessibility of machinery.* The machinery should be physically arranged so that every major component is accessible for adjustment, repair or replacement without the need for removal of other major components. Hatches and overhead cranes are often provided to facilitate the removal and installation of equipment. There should be ample space around machinery, to enable workers to use long wrenches and spanners, as well as power drills, and clearance to swing hammers. Machinery distress often occurs many years

after its installation – at temperature extremes and at night, the most unfavourable conditions for diagnosing malfunctions and making repairs.

- **Single-mode failure analysis.** Safety is an important issue in mechanism design, as well as structural design, of a movable bridge. The structural safety issues, such as fracture of critical members and redundancy, are treated elsewhere in this book. Both span drive and stabilising mechanical and hydraulic bridge machinery should be analysed for single point failure (Sööt, 1990). The machinery design should be such that the failure of one component should not cause the bridge to undergo uncontrolled motion, even for a short duration. The current movable bridge specifications are silent on the topic of failure analysis. It is implied that if the design requirements (such as allowable stresses and distortion limits) specified in codes are satisfied, there will not be uncontrolled motion. However, mechanical and electrical components can fail and the system should be analysed for the effects of a component failure. Modes of mechanical failures and their characteristics may be found in Wulpi (2013). A comprehensive review of the influence of material defects on fatigue life is presented in a three-part paper by Zerbst *et al.* (2019a, 2019b, 2019c).
- **Failure mode and effects analysis (FMEA).** This systematic approach to failure analysis was introduced by the military around 1950 and adopted by the aerospace and automotive industries and later by the nuclear power industry. The analysis may be conducted at various levels of sophistication (McDermott *et al.*, 1996). Besides detecting potential instabilities caused by component failures, it is useful in life-cycle cost analysis.
- **Life-cycle cost analysis.** The bridge designer often needs to choose between alternatives – materials, machinery components, systems, and so on. Nowadays, bridges are designed for a useful life of 75 years or more, but some components normally have a shorter life, depending on the number of bridge openings per year and environmental conditions. Examples are wire rope and valves in hydraulic equipment. Consider the case of counterweight ropes of vertical lift bridges. The ropes and the supporting sheaves should be treated as a system when selecting ropes and choosing sheave diameters. One of the factors limiting the useful life of a rope is fatigue of wires, owing to the bending that occurs as the rope passes over the sheave. The larger the ratio of sheave diameter (D) to rope diameter (d), D/d , the longer the rope life. However, the first cost of the sheave increases as $D^{n>1}$. A smaller diameter sheave with a reduced rope fatigue life might be the minimal cost (per annum) combination. Life-cycle cost analysis for bridges is discussed by Hawk (2003). Carnegie Mellon University formed the Green Design Institute, which has operationalised the economic input–output life-cycle assessment (EIO-LCA) method developed by Nobel Prize-winning economist Wassily Leontief. Online tools for applying the EIO-LCA method are available at Carnegie Mellon University (2018).
- **Design quality assurance.** Public safety is an increasingly important consideration for bridge design and construction. Tolerance for design mistakes that result in death or injury is markedly less for the bridge industry now than it was even a few years ago. The spate of bridges built after the Second World War are now over a half-century old and many of these bridges have not been adequately maintained. Failures occurring now, 70 years after the bridges were opened to traffic – during which time they have been modified and are subjected to loading from much heavier vehicles

– are often attributed to design errors because design errors are easiest to find, and the personnel are not around, or in a position, to defend themselves.

- **Designs of the machinery,** as well as the structure should be robust, with redundancy, to reduce the likelihood of sudden collapse or out-of-control movements. The philosophy of Eurocode EN1990 (CEN, 1990) is also valid for movable bridge design.

In some countries, such as the UK and the USA, designs for public works are prepared by governmental agencies or by engineering consulting firms engaged by them, under the design-bid (tender)-build method (DBB method). Drawings and specifications are prepared that serve as the basis for a legal contract between the owner and the contractor who physically executes the work. The effort required to prepare the contract documents for the electrical and mechanical systems can be considerable. Much depends on the type of movable bridge, whether it is a new or a rehabilitation project, and the construction contractor climate in the area at the time of letting. It is not unusual for the owner to be surprised by the high design cost for the electrical and mechanical systems. These costs can be somewhat reduced and absorbed in overall costs by using the design-build project procurement DB method (UK) or project delivery method (USA). The design-build project delivery method (DB), common in Europe, is becoming more popular in the USA. Morledge and Smith (2013) thoroughly treat project procurement in the UK, Europe, the Middle East and China. The project delivery method in the USA is discussed by Drewry (2008), the FHWA (2006) and Pankow (2014).

Construction support

In the traditional public works construction process (DBB), engineering services are required from the designer or other qualified engineer (Council of Engineering Institutions) after the construction contract is let, to attempt to ensure that the contractors comply with the intent of the designer, as expressed in the construction contract documents. These services involve the review of shop drawings and other vendor submittals, inspections, resolution of unforeseen field conditions (especially for rehabilitation projects), assistance with electrical start-up (commissioning) and acceptance testing.

Construction inspection

During the construction of a movable bridge, inspection of the work by the structural, machinery and electrical subcontractors is essential if a properly functioning movable bridge is to be achieved. The bridge as a whole is a machine and the parts have to fit together if it is to operate reliably without the need for excessive maintenance. Inspection of the movable bridge superstructure work is similar to that for fixed bridges, as described elsewhere in this book. Machinery inspection should take place in the shop and should include verification

of material properties, dimensions, finishes and meshing of gear teeth – all of which should conform to the shop drawings that were reviewed under the construction support agreements. The design documents should set out requirements for testing machinery, such as speed reducers and custom-designed actuators. These tests should be carried out and reported, and any necessary corrections made before the equipment leaves the manufacturing facility. Verification of electrical and hydraulic components and circuits should also be done in the shop and the assemblies connected and subjected to simulation tests. The simulation tests should be for all modes of operation and include all auxiliary equipment, such as span locks, traffic warning and resistance gates, traffic signals, navigation lights and control displays. Drive motors and drives should be tested on a dynamometer in the shop to verify the design speed–torque characteristics. The final simulation tests should be conducted in the presence of the designer, prior to transporting the equipment to the construction site.

Field inspection of the machinery installation includes checking the alignment of machinery components, especially at shaft couplings. The misalignment allowance for couplings should not be ‘used’ for installation error; it should be ‘saved’ for structural distortion during span operation and temperature change. Inspection of the electrical installation is important; conduits, terminal boxes, submarine cables, and so on, need to be installed completely, taking into account the marine environment.

Periodic inspection of movable bridge machinery

The collapse of the Silver Bridge across the Ohio River at Point Pleasant, West Virginia, in 1967 brought to public attention the state of public bridge maintenance in the USA. This situation was due, in large part, to the manner in which public works were funded, with federal funds available to assist states and localities with new construction, but comparatively little funding for maintenance or rehabilitation of existing infrastructure. However, the failure of this eyebar suspension bridge resulted in legislation mandating biennial inspection of all bridges supporting public roads, with the federal government bearing a large share of the cost. The original legislation was directed primarily at bridge superstructures but, in practice, inspection of machinery and controls was encouraged at regular intervals.

As no publication addressing the inspection of machinery and controls of movable bridges was available in English at the time, the Federal Highway Administration published a manual for inspectors of bridge machinery (FHWA, 1977). It has recently been revised by the American Association of State Highway and Transportation Officials (AASHTO, 2017). A *Bridge Inspection Handbook* was published by the American Railway Engineering and Maintenance-of-Way Association (AREMA, 2008). Parsons Brinckerhoff published a practical

guide to bridge inspection (Silano and Henderson, 1992), which includes a chapter (Abrahams MJ) on movable bridges. Scopes of work for machinery inspection have been presented (Birnstiel, 1990b).

Future trends

Perceived future trends in movable bridge engineering may be considered under two topics: rehabilitation of existing bridges and construction of new bridges. With respect to rehabilitation, the need for innovative engineering is great. Many movable highway bridges are over 50 years old and in varying degrees of deterioration, owing to environmental conditions, worn machinery and obsolete electrical control systems. Others are geometrically inadequate because of the wider and longer motor vehicles now permitted and structurally inadequate because of the legally heavier trucks, and the even heavier illegal trucks that authorities are effectively powerless to control. American railway bridges are in a similar situation; most long-span movable railroad bridges are over 75 years old, deteriorating and subject to much more fatigue than heretofore because of long unit trains of heavily loaded cars. Regarding motive machinery, there is a trend towards the use of more hydraulic machinery and bascule spans without, or with only partial, counterweights. The use of lighter and more corrosion-resistant materials, such as aluminium, fibre-reinforced plastics and lightweight Portland cement concrete, will be further explored for highway and pedestrian bridges. Thus, there is a great need for rehabilitating or replacing movable bridges, to maintain a viable transportation infrastructure.

New movable bridges are also necessary where underutilised ports are being reconstructed to promote recreational use or where navigation channels are being widened and deepened to accommodate post-Panamax vessels. Decisions as to whether to rehabilitate or replace a bridge, or add a new bridge, are now made as part of a formal infrastructure planning process. In the USA, that process may take decades for a particular project. Because the public is now more involved in evaluating the environmental impact of bridge projects and usually does not appreciate the straightforward industrial architecture of past movable bridges, architects are now playing an increasingly prominent role in bridge design teams – to present something different and dynamic to the public. As part of this process, movable bridge concepts that engineers considered obsolete are being reinvented (Birnstiel, 2004).

In summary, although movable bridges will be replaced by fixed bridges wherever feasible, many movable bridges cannot, practically, be replaced by fixed bridges and need to be brought to current standards. Many need to be rehabilitated because machinery and controls have deteriorated or because they are to be saved as cultural artefacts. New movable bridges will be required to accommodate larger vessels and revitalise ports for recreational use. Many of these will be pedestrian and bicycle bridges, which are candidates for architectural experimentation.

Conclusion

Movable bridges are important components of the transportation infrastructure of many countries. They need to be rehabilitated or replaced in order to compensate for degradation owing to environmental factors and inadequate maintenance, or to meet public demand for modern architectural styles. Common types of movable bridge are trunnion bascule, rolling bascule, swing and vertical lift. To present dynamic 'new forms', architects are resorting to bridge types that were, until recently, considered obsolete. The movable spans may be operated using mechanical or hydraulic (fluid power) machinery. There is a trend towards greater use of hydraulic components. Changes occurring in commercial water transportation, changing attitudes towards personal transportation and increased industrialisation of agrarian countries will require many new movable bridges.

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Chapter 17

Footbridges

Saeed Ziaie WSP

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From the basic to the sublime, from their earliest simplest forms to the most complex structures, footbridges continue to play a significant role in our built environment. Having several distinct features, footbridges are presented as a special case of bridge engineering. In this chapter, a brief introduction to footbridges is presented, which is followed by discussions on brief and general requirements, aesthetics and context. Principles of setting out are described and extended to cover access, alignment and structural proportions. Different types of footbridge are reviewed, with their appropriateness for different circumstances. Detailing and its importance in constructability and maintainability is identified and discussed. A discussion of materials and finishes looks at the traditional as well as modern and more advanced options for both structural and architectural elements, including finishes for decks and walkways. Following construction processes and techniques, ten examples are provided to cover a variety of principles, forms, constraints and situations that may be encountered. It is concluded that footbridge design and construction will continue to inspire engineers and architects to create new forms, employing new design and construction techniques and materials.

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Introduction

From the simple to the sublime, footbridges go back in history and cover some of the earliest and simplest forms of structures, helping people to cross obstacles; in the present day, they can be more complex and sometimes iconic structures, playing an important part in transportation strategies and environmental features.

Footbridges have been common features of our environment for centuries; from their earliest existence, they have facilitated the movement of people. Whether historical remains, architectural marvel, cultural icon or a simple crossing, not only do they connect geographical locations, but they can also serve as social connectors, facilitating general interactions between people. Over busy roads and other obstacles, they provide safe passage for various users, including disabled people and those with pushchairs. Footbridges, also referred to as pedestrian bridges, may be required to provide access for other users, such as cyclists and equestrians. Furthermore, foot and cycle bridges play an important role in transportation strategies and can also be designed as part of public realms or elevated walkways. In some instances, they can also be a means of access between two or more buildings in an urban area or elevated points within mountainous recreational zones (also referred to as sky walks or sky bridges).

For convenience, the term footbridge or bridge shall, generally, be used throughout this chapter to cover

- pedestrian bridges
- pedestrian and cyclist bridges
- equestrian bridges and bridleways
- sky walks or sky bridges
- public realm footbridges and walkways.

From those in isolated communities and developing parts of the world that provide access to essential needs, such as healthcare, education and economic opportunities, to the more complex and iconic structures, footbridges provide a significant opportunity for the designer to enhance the transportation network and people's lives and contribute to sustainable development, achieving a balance between social, economic and environmental aspects.

Footbridges predate any other kind of bridge. The early bridges were very simple structures, built from available natural resources; wooden logs, stone and earth. They were only able to span very short distances and had low structural integrity. It was not until the arrival of the Roman Empire and the introduction of arch structures that bridge building was transformed. The Romans went on to build several hundred bridges during the life of the empire, no doubt contributing to Europe's long history of bridges. To do justice to such rich heritage, we must remember that some of the most iconic examples come from a city that relies on its bridges more than

most. A network of some 400 bridges crosses over the many canals in Venice, providing access for those who live and work or visit this magnificent place. Of those, the Ponte di Rialto, the oldest of the four bridges spanning the Grand Canal, has stood as an engineering and architectural marvel of the Renaissance since its completion in 1591 and is a significant tourist attraction, while remaining functional and vibrant. Since those very early days, mastery of bridge building has continued to advance.

Today, footbridges are generally lighter than other bridges; this allows structures of extreme delicacy and simplicity. Their design can often provide unique opportunities for the engineer and architect to collaborate and explore different forms and materials. A footbridge can often represent a significant communal investment, a symbolic demonstration or a potential site for expression. Lifespans are usually long, leading to timelessness, and footbridges often display their construction date. The act of using a footbridge at walking pace allows a slow and measured interaction with the design of the bridge and its surrounding environment and demands utmost attention to the design of its components. Therefore, its details, tactile or visual, must be carefully considered and resolved. Footbridges are particularly suited to engage sensitively and creatively within their context and environment. Context can be defined in the widest of terms. The parts of any footbridge design include the wider landscape context, approaches and access, materials and their modes of assembly, details, texture, scale, colour, form and lighting.

A variety of special characteristics sets footbridge design apart from other bridges. Some are covered in Box 17.1.

Live load actions can be significant in comparison with dead load actions; dynamic behaviour is usually important.

Owing to their lightweight nature, the effect of collision loads on both the superstructure and substructure must be either eliminated or adequately designed for.

The lightweight nature of the structure, particularly combined with high skews or curved configurations, could result in uplifts, which should be considered in the design of all elements, including bearings.

The low cost of projects relative to other civil engineering works may make expressive or experimental designs economically viable.

Each design is a potential site for exploring the collaboration between architect and engineer. The relative input of the two types of designer varies widely with the complexity and requirements of the project.

The scale of construction suits many types of structural materials, including steel, concrete, masonry, timber, plastics and glass.

Construction technique can have a significant impact on form.

There is a direct and close encounter between a footbridge user and the detailing of the structure.

Box 17.1 Some characteristics of footbridges

While the basic bridge types continue their influence, with an abundance of designs for footbridges, it is unlikely that convergence towards distinct typologies can be achieved. Indeed, as

hybrid forms and ‘pseudo-structures’ of indeterminate action appear more and more often, some structural mechanisms may become dated.

The advent of more cost-effective reliable computer analysis and design software, however, means that statics, dynamics, instabilities, non-linear aspects and construction practicalities can all be readily assessed in the most arbitrary of forms. The resulting framework of practical solutions allows specific opportunities for architectural and structural intentions to be resolved simultaneously.

In a footbridge, technical and aesthetic contributions can be set in balance. The lessons learned from this area of design activity are distinct from more general considerations but can be applied to other complexities. The design of a successful footbridge involves a multitude of design concerns, which, more than any other bridge type, extend the focus of an engineer’s attention outwards and away from the traditional role of designing an economical and appropriate structure. Nevertheless, a good footbridge can result from a straightforward engineering response, presupposing a developed sense of good design.

Traditionally, an aesthetic response in bridge design has been secondary to the development of technical solutions; only in the early part of the twentieth century has it assumed a self-conscious part of the engineer’s remit. Aesthetic consciousness in the design of bridges has grown over the last century, leaving the traditional roles of architect and engineer unbalanced.

The self-consciousness necessary to address the aesthetic concerns of engineers became set in with the Industrial Revolution. First examples were odd mixtures of pre-adaptation and attempted demonstrations of difference and modernity. The Iron Bridge in Coalbrookdale takes timber detailing and casts it in iron. The German-speaking lands of Prussia and Switzerland took a formal approach to the aesthetics of bridge design, culminating, early in the twentieth century, with the work of Robert Maillart (Figure 17.1).



Figure 17.1 The Töss footbridge, Switzerland, by Robert Maillart, 1934

After the Second World War, reconstruction in Germany and France, with scarce resources and a determination to demonstrate technical efficiency as part of the Wirtschaftswunder (economic miracle), promoted aggressive innovation. This trend followed

a more general quest for refinement and truth to structural action, manifest as streamlining in USA and the early phases of the high-tech style in England.

Most recently, many bridge designs, particularly those arising from increased design competitions have been experiments, set pieces and paradigms towards new bridge types. The work of Santiago Calatrava (Figure 17.2) typifies how a strong aesthetic response is now seen as part of the remit.

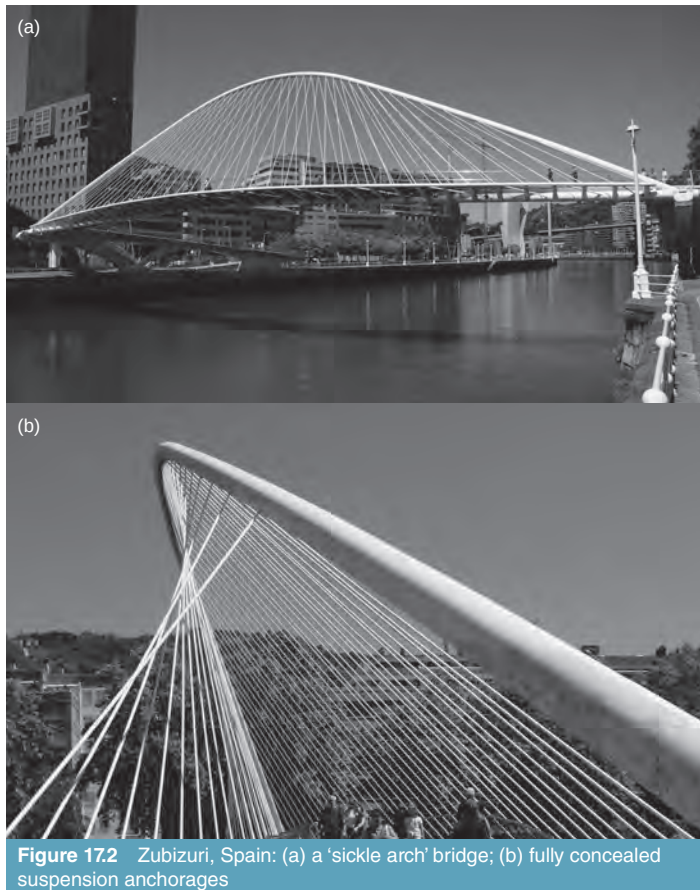


Figure 17.2 Zubizuri, Spain: (a) a 'sickle arch' bridge; (b) fully concealed suspension anchorages

This growing sense of appreciation, in large part, is due to the advances and availability of new design tools. Three-dimensional modelling and visualisation packages have considerably removed limitations and enabled designers to be free to test anything that can be imagined. Analytical systems quickly check overall robustness, stability, long-term creep and thermal effects, as well as construction sequence effects. Patterning and manufacture are automated to make the complex shapes of shipbuilding economic for the bridge builder. Fabrication imperfections still cause challenges, but these should eventually be overcome.

The following sections address technical and aesthetic considerations for footbridge design. Starting with the requirement for a clear briefing document, the process of analysing context is discussed, along with setting-out criteria. Current practice, bridge types, materials and construction techniques are explored and special types, such as movable bridges and

sky bridges, are examined. Sections on aesthetic principles applied to footbridge design and design methods lead to a brief conclusion considering the future development of footbridge design.

Brief and general requirements

This chapter describes technical requirements, as well as other influencing factors in the design and construction of footbridges. A variety of approving authorities and other stakeholders could be involved in the design of footbridges and as the project progresses through the general phases of initiation, planning, execution, control and closure. Depending on the type of client and approving authority and other stakeholders, different project delivery processes might have to be followed. It would, therefore, be prudent for the designer to be familiar with the required process to ensure efficient and correct delivery at appropriate stages throughout the project. Some of the processes that the footbridge designer may have to follow are:

- Highways England's project control framework (PCF)
- Network Rail's governance for railway investment projects (GRIP)
- The plan of works of the Royal Institute of British Architects (RIBA)
- Protocols of local authorities and other overseeing organisations.

In the UK, unless the project is for Network Rail, bridge design generally follows the Design Manual for Roads and Bridges (DMRB). The current DMRB standard CG 300 (formerly BD2), Technical approval for highway structures (Highways England, 2021a), is of specific importance, as it sets out distinguishing requirements for bridge design compared with building structures. This document states the need to produce an approval in principle (AIP), identifying key design parameters, standards, categories of checking and certification and any potential departure from standards that must be agreed with the technical approval authority (TAA).

Footbridge design is particularly sensitive to intended use. Clients and overseeing organisations need to determine user groups and the principal purpose of the bridge before commissioning design work and deciding on the bridge's final location and alignment.

For most bridges, a route sited on a desire line – the shortest route between two points – is the most obvious solution. However, there are many qualifications to this assumption. If a bridge is part of a leisure or landscape route, there may be additional incentives to site the bridge obliquely, avoiding a direct connection in order to take advantage of the views and the topography.

The vertical clearances to a highway beneath the bridge should be in accordance with CD 127 (Highways England, 2021b). For new constructions, this will require a minimum headroom of 5.7 m plus an additional allowance for sag curve compensation. The horizontal clearance from the edge of the carriageway below to the footbridge supports should be

a minimum of 4.5m over the full height of the carriageway clearance envelope. The vertical and horizontal clearances to railways, canals and watercourses should be agreed with the technical approval authority.

If the bridge is to be used by pedestrians, typical widths and parapet heights are indicated. A normal minimum bridge width for pedestrian use only is 2 m.

Where combined use by pedestrians and cyclists is envisaged, an additional width of 2.7 or 3 m will be required, depending on whether the footway and cycle path are segregated by a kerb not less than 50 mm high or by a physical barrier of not less than 900 mm high (see also Table 17.1).

Where combined use by pedestrians and equestrians is required, the minimum width of a footbridge is stated as 3.5 m. However, a larger width may be required where larger numbers of horses are expected to cross the bridge.

The minimum headroom, from the footway surface to the underside of overhead obstacles, such as cable stays, roof enclosures or any other overhead feature, shall be in accordance with Table 17.2.

Minimum heights for footbridge parapets are typically 1150 mm, except over railway property, when they are 1500 mm. This is measured from the highest ground level within 1500 mm of the barrier (which is taken as the datum level). This might have to be increased to 1800 mm in areas prone to vandalism over railways. The designer should take into account all relevant factors, such as high prevailing winds, the height of the structure, and the density and profile of intended use. Reference should be made to mandatory or guidance notes issued by rail, waterway and highway authorities and current relevant local, national and international (where appropriate) codes and standards to confirm dimensions. These include building regulations, British standards and European norms. Lists of relevant British standards, European norms and DMRB standards are included at the end of this chapter.

It should be noted that all information provided here and in the various codes and standards is generally for guidance

and that these are the minimum requirements. It is prudent that the designer gains a clear understanding of the client's requirements and aspirations, which may differ from these minima. In addition to the client requirements, there may also be other constraints and influencing factors, which may affect these requirements. Special concessions may have to be considered where the footbridge is within an existing built-up area or interfacing with existing structures with limited clearances. Any departure from minimum requirements and recommendations of the standards must be agreed with the TAA and measures taken in the design to compensate for any potential deficiency where necessary.

Consideration must be given to the needs of visually and mobility impaired users, and the detail designs developed accordingly (see BS 8300 (BSI, 2001), and other relevant standards). Major components, such as ramps, lifts and staircases, inform the fundamental composition of the bridge and need to be finalised at an early stage.

If cyclists are to share the bridge with pedestrians, different design criteria apply; this has to be agreed from the outset. The bridge deck can be shared and unsegregated, with recommended widths as in Table 17.1 or as agreed with the client and TAA. Parapets need to be raised to a minimum height of 1500 mm above the datum level for foot or cycle bridges over railway property.

Equestrian bridges are another special case; the particular requirements of horses and equestrians need to be taken into account at the outset of the design process and agreed with the client body. For combined foot–cycle–equestrian bridges, the minimum width is of the order of 3.5 m. The typical height of parapets above the datum level is 1800 mm but this does not allow for dismounting on the bridge. If the crossing is not part of a bridleway, horses can be led across. Mounting and dismounting blocks on the approaches to the bridge should be provided.

While there is not a great difference in live load allowances, the requirements of footpaths, cycle tracks and bridleways affect

	Footway: m	Cycle path: m	Total: m
When segregated by a kerb not less than 50 mm high	2.0	2.7	4.7
When segregated by a physical barrier not less than 900 mm high	2.0	3.0	5.0
When segregated by a white line or contrasting surface colours	1.5	2.5	4.0
Unsegregated	–	–	3.5

Table 17.1 Minimum bridge widths for shared use (Table 11.7 of DMRB CD 353 (Highways England, 2020a))

	Headroom clearance: m
For use by pedestrians only	2.3
For use by pedestrians and cyclists	2.4
For use by dismounted equestrians (dismounting provisions in accordance with Section 11 of CD 353)	2.7
For use by mounted equestrians	3.7

Table 17.2 Minimum headroom clearances on footbridges (Table 5.6 of DMRB CD 353 (Highways England, 2020a))

setting out, details and materials – all affecting appearance and the detail design process. Traffic flows on bridges should be analysed and the likelihood of crowd loading determined. Crowd loading consideration is particularly important if the bridge is located in urban areas. As well as defining the user groups and modes of use, footbridges should be designed taking into account relevant factors, such as the design life of the structure, safety, cost, buildability, operation and maintenance. The integration of structure, architecture and landscape design is paramount.

Commissioning bodies and designers should consider the likelihood of vandalism or misuse of a proposed footbridge. Enclosed footbridges are often specified in areas where dropping objects from the bridge or the possibility of suicide might be an issue. Such structures further increase the visual impact of a proposed crossing and should be designed and integrated from the beginning. Safety is paramount but other considerations, such as environmental impact, usability, appearance and economy, need to be considered and balanced together.

Clearances under structures that cross highways, waterways or railway property need to be agreed with the relevant authorities early in the design process and form part of the design brief. Clearances over highways should be in accordance with highways departmental standards. Vertical and horizontal clearances over rivers, canals and waterways need to be agreed with the appropriate authorities; in the UK, they should be agreed with British Waterways and the Environment Agency or the canal authorities. Clearances over rail property should also be agreed with the relevant authorities; in the UK, this is Network Rail. If the proposed footbridge is in a location that affects a site of historical importance, the relevant authorities should be consulted, for example, Historic England.

The client organisation should consider the expected design life of the proposed bridge. In the UK, the service life of a footbridge is usually set at 120 years, effectively as long as possible, although component parts will have a different design life to first maintenance or replacement. Currently, cable systems cannot achieve such longevity and provision for hanger replacement must be made for cable or suspension type bridges.

When considering sustainability, it becomes imperative to produce an efficient structure, using a minimum of material. However the amenity value of a well-maintained structure that is expected to last for over four generations should not be overlooked. When assessing the value of a footbridge, all aspects and stages, including maintenance and decommissioning should be included.

The designer must consider health and safety in relation to buildability, maintainability and decommissioning of the structure and its components. In Great Britain, designers must comply with The Construction (Design and Management) Regulations 2015 and, in fulfilling their duties, must consider all foreseeable hazards associated with construction, maintenance and ultimate dismantling or decommissioning of a structure and eliminate, reduce or control them. Indeed, early consideration

of construction, maintenance and decommissioning may influence the design and construction choice and methods.

Applied live load allowances have a wide range, reaching a maximum for packed and panicking crowds. It is particularly important to consider crowd loading on bridges in urban areas, which could be subject to crowds of demonstrators. A loading of 5.0 kN/m^2 , as specified in load model 4 in BS EN 1991-2: 2003 (BSI, 2003), should be applied in unfavourable parts of the influence surface, longitudinally and transversely. While load reductions may be allowed for long spans, this must be treated with extreme care, particularly in or near towns and cities.

Firework displays over water can also lead to exceptional crowd densities. Any serviceability requirements for fire tenders, maintenance access or occasional vehicle use are unlikely to govern overall but may significantly exaggerate point load conditions and must be considered. Other considerations should include fire tenders, which can be onerous, and maintenance gantries or cherry pickers, which can be designed to spread their own loads.

With bridge designs becoming increasingly lighter, wind effects have become much more significant. Unusual deck, pier and pylon shapes are also more complicated to assess. Typically, buffeting response might produce additional deflections up to 50% of predicted static displacements. Winds come in cycles. A power spectrum of gusts will intersect with the response spectrum of the bridge structure and energy transfers will take place. Inherent damping, in materials and on joints and fittings, may limit the build-up but, unless expensive add-on damping is considered, the structure must be designed to bear the additional movements. Ultimately, strength, deck-level accelerations and fatigue must all be checked.

Aerodynamics interactions can occur in decks, pylons, stays and localised elements. Divergent responses can be extremely dangerous; a small initial displacement, usually from buffeting, drives the structural element from its position and the surrounding air flow is altered, which produces a lift component that is resisted. If this sequence falls into a cycle, an energy build-up can occur. The bridge deck, like a wing, can twist lightly and starts to fly. As the angle of attack gets too steep, the deck stalls and twists back down. The action recurs in the downwards direction and a monotonous twisting action is set up. Galloping occurs when an element is displaced sideways and a lift component appears in the air flow. This is resisted by the stiffness of the structure, which recovers. The element is dragged back through the centre and the effect occurs on the other side. A rhythmic motion can follow.

These effects are dependent on the natural frequencies of the structures. Unfortunately, the vertical and torsional stiffnesses of a bridge deck are usually close, so the two movements can couple. So-called classical flutter is most severe. The deck twists, rises, comes to the horizontal, then twists back down, moving like a fish in one smooth movement; energy build-up is rapid. The best way to avoid the problem is to keep the natural frequencies of vertical and torsional modes a factor of three apart.

Fluctuating live load excitation is also extremely important. Loads are typically delivered through footfall. The cyclical motion of human beings and horses walking is unfortunately close to the frequencies natural to the usual scale of footbridge structures. The celebrated example of the disciplined soldiers failing to break step and thereby driving a suspension bridge to destruction has proved misleading. In 1850, a battalion of French infantry, 478 men, marched out onto the Angers Bridge, a wire suspension structure across the River Maine. The construction was 22 years old and its anchorages were severely corroded. The bridge began to resonate under the tramping boots. As the soldiers reached midspan, the main suspension cables parted and they were pitched into the water, laden with their equipment.

Subsequent observations showed that subtler interactions are at work. A mass of moving people packed closely together tend to fall into step. The response of the underlying structure acts like a metronome, adjusting the pace onto the natural frequency and aggravating the effects. This model of structure and human behaviour correspondence has proved attractive to research engineers and has led to acceptance criteria becoming available for simple and more complicated models.

Such an effect was experienced with the London Millennium Footbridge. This elegant and modern suspension bridge, with a very low sag ratio so as not to impede views of St Paul's Cathedral, has a central span and unequal back spans set out in direct alignment, almost joining the cathedral precinct and Tate Modern art gallery (Figure 17.3(a)). Heavy groups of cables support a lightweight steel girder with extruded aluminium decking. On opening, the structure suffered undue pedestrian excitation and damping was retrofitted (Figure 17.3(b)).



Figure 17.3 London Millennium Footbridge, Foster+Partners/Anthony Caro/Arup, 2000

Mass-tuned dampers prevent the onset of regulated footfalls and viscous dampers limit energy build-up.



Figure 17.3 Continued

Context and setting out

Once the functional requirements of a footbridge are determined in the brief, design progresses to selecting and dimensioning a structural configuration that is economic, efficient in material use and construction resources and achieves a visually balanced composition of elements brought together within the context of the bridge's surroundings.

Context

Consideration of context requires a careful survey and assessment of the controlling factors and is the key to the design strategy that will be adopted. An initial appraisal of the context will assess the relative importance of the bridge, compared with its setting.

The decision regarding placement of a span will include a preliminary estimate of costs of alternatives and an appraisal of the effects of the bridge on its surroundings. The aspirations for the impact of the bridge must be identified. Town planning and landscape intentions must be considered and the relative importance of the bridge to its setting needs to be established. A footbridge can fit the context in an understated way or be dramatic and stand out. The peculiarities of a site may be the primary influences on a design that have to be carefully considered by the designer.

It is difficult and indeed undesirable to establish a set of rules for different types of site. Undistinguished urban places

may benefit from a signature bridge that establishes a sense of place. Alternatively, the addition of one simple element might resolve a visual conflict. Bridges in urban scenes should not be designed in isolation; they should rather have a more architectural quality and balance of shape and form. Given the intended life span of a building, long-term planning objectives should also be accounted for.

A bridge in a dramatic landscape setting might match its grandeur, which may be achieved in a simple and straightforward way. If the setting is pastoral and gentle, the response should have a level of subtlety to match.

Alignment

Site history might dictate the routing of a proposed footbridge where existing routes need to be enhanced or reconnected, for example, over new or widened roads, or across streams or rivers. The alignment of such a bridge should respect its historic routes. Footbridges should work with the contours of approach slopes, masking or understating bridge abutments, ramps or steps by landscaping and adjusting the vertical alignment to improve access. For projects involving new routes, a bridge becomes one element to be manipulated within a landscape or urban master plan.

Footbridges adjacent to or alongside existing bridges should relate to the alignment of pre-existing structures that are to be retained. Running bridges in parallel avoids awkward spaces between the bridges and complements rather than competes. The distance between the new and existing bridges is a matter of judgement and scale. Generally, a minimum of 2 m should be allowed for and the new bridge should be detailed to deter attempts to cross between bridges. Adequate protection measures should also be allowed for where new footbridges are planned close to existing highway bridges.

On new routes, footbridges can be placed to work with the weather and with natural light. Alignment can significantly affect comfort of use and weathering, and prevailing winds may also determine deck conditions. Sun angles dictate the way in which a bridge is perceived as a series of planes; the sun path around a well-designed structure can bring out the mass and form of the primary structure and the inherent transparencies of the secondary elements, animating the visual composition.

Footbridges have the potential to become viewing galleries in themselves by virtue of their slow traverse, and are therefore sometimes oriented and organised to turn a journey into a destination in itself. Views both outwards and ahead can be controlled and manipulated by sinuous alignments and form.

However, other practical considerations regarding alignment may also be relevant. For locations subject to crime or vandalism, it is always better to be able to see routes clearly and completely, without bends to cause anxiety; for equestrian users on shared bridges, it may be better to be able to wait until no bicycles are coming before leading a horse across.

Setting out: structural proportions

While the route is generally determined by anthropomorphic criteria, these requirements must be combined with the structural dimensions determined by analysis. The process of footbridge design involves manipulating the relationship between the overall configuration and form and the individual member size to achieve a balanced composition.

Simple beam elements have an efficient balance of strength and stiffness at span-to-depth ratios of 20–25. For a typical 30 m span footbridge across a road, a 3 m wide deck will span between side beams with an overall depth of 200 mm. This gives a minimal rise to the bridge across the road clearance. If the main bridge beams (girders) are fitted into parapets that are 1.1 m deep, the overall structural depth available is 1.4 m, ideal for the span. The upper compression flanges of the beams need restraint against lateral torsional buckling; this can be provided by introducing a series of U-frames, coupling the two sides of the bridge together into a stable whole. Such a footbridge will be sufficiently light and of a length to be prefabricated and transported by road and lifted into place. Cantilever elements work efficiently at span-to-depth ratios of between 7 and 10. It is sometimes possible to minimise the intrusion of a footbridge on a bankside by arranging side spans to cantilever back from river piers to touch the ground at each end only lightly. This can assist in reducing the depth of the main beam.

Truss frames are most effective at a span-to-depth ratio of about 15. The extra depth over equivalent solid beam elements deals with the larger shear deflections appearing in end panels. For a footbridge of around 30 m span, it is possible to arrange to walk through a box girder comprising trussed sides with upper booms stabilised by cross bracing. For an overall height of 2.5 m, individual elements will have an appropriate scale of about 200 mm diameter. This type of structure can be useful where the required headroom below the structure limits the depth of the structure.

As the rise of an arch increases, so it becomes stiffer. This also influences the susceptibility of the arch to lateral and snap-through buckling. A span-to-rise ratio of about 6 is a good compromise. For a span of 30 m, this will take the crown of a bowstring bridge level with the treetops. A simple arch ring will appear slender, with a diameter of around 400–500 mm. Any asymmetry will necessitate significant thickening of members.

Suspension bridges are efficient because their main structural components, the suspension cables, are self-stabilising. For very long spans, this makes for the best structural form, as the weights of supporting pylons and deck lose their significance. A suspension configuration would be needed to approach the maximum theoretical span of a bridge where material strength is only just capable of holding up the self-weight.

Span-to-sag ratios of about 6 will keep the supporting pylons from becoming too tall. As the profile is reduced, the cables become very thick, as, for example, the cable arrays supporting

the London Millennium Footbridge. In the limit, a stressed-ribbon bridge is reached, where deck and suspension cables are coincident. The requirement for ramps to be of 1:20 slope determines the setting out of such ribbons; current material strengths limit spans to 120–140 m for fully accessible decks.

Cable-stayed footbridges are economic over mid-range spans, 40–80 m, and can be designed in a variety of forms. Closely spaced cables of about 2 m apart at deck level minimise deck thicknesses and produce a diaphanous surface of cables. More widely spaced arrangements produce heavier lines. Cables much larger than 60 mm in diameter will look heavy close up and hangers become soft as their self-weight sag adds to deflections under live loads. Curving deck lines can be efficiently supported if pylons and cable configurations are arranged correctly. Ties joining a curving deck with a straight pylon produce an attractive envelope of lines.

Hangers should be set out at a minimum slope of about 30° to the horizontal to avoid vertical load-carrying capacity being adversely affected by the generation of excessive lateral loads. For a 40 m span bridge, pylons will just be overtopping surrounding trees or three- and four-storey buildings. The vertical pylon will be 600–900 mm in diameter and therefore capable of concealing typical sizes of cable anchorage. In steel construction, profiles intended to catch the light can be reassembled from split standard rolled sections.

Access

Wherever possible, bridges should be designed so that they work with the contours and levels of the site. In terms of economy, every effort should be made to manipulate the alignment to work with levels, mounds, contours and adjacent buildings and structures to minimise the need for steps, ramps, lifts and escalators. This objective usually matches structural and architectural economy and results in a structure best integrated in its surroundings. Access should generally be as short and as direct as possible, avoiding circuitous routes.

Access to footbridges should be by ramps and stairs together if this is practical. If the routing is reasonably direct, stairs can be omitted. Ramps can be designed to enhance the setting of the bridge and should be viewed as a positive design challenge. They should be placed on the principal desire lines of the bridge and designed to integrate with the structural and architectural concept of the main span. The geometry of ramps should be kept simple and legible, whether straight or curved. Landings should be avoided in curving geometry to keep the graceful line flowing vertically and horizontally. Spiral, helical and curved ramps take a considerable amount of space but can be extremely beautiful in the appropriate context and surroundings. As ramps come towards the ground, there may be a case to use a bund to incorporate the lower parts of the ramp and avoid a dirt trap, yet it remains visually more elegant to extend the ramp to the ground with the smallest possible abutment. This would be a sliding connection for floating bridge spans.

Ramps should be designed with due regard for mobility impaired users. Refer to relevant standards, including the

relevant building regulations and BS 8300 (BSI, 2001).

For pedestrian bridges in cities, space is normally very tight and stairs and ramps are difficult to arrange. Lifts may be needed for disabled access if space to provide ramps is not possible. Lifts in rural settings are very difficult to maintain. The incorporation of a flight of steps and a strongly vertical element, such as a lift, at the ends of a bridge crossing is problematic in the sense that the opposing architectural forms are not easily integrated as extensions to the predominant bridge form in the way that ramps can be. Differentiation is normally the best strategy; this can be achieved using simple linear flights or steps that are subtly arranged within landscape forms and terracing.

Steps incorporated in adjacent buildings or structures in an urban situation can result in a clear bridge span without the clutter of secondary structures but may be problematic, owing to concerns about cleanliness or security of the route, which might not be open to view. Modifications can be made to allow bicycles to be wheeled up staircases, in steel channels or grooves. Lifts and escalators for bridge access can similarly be incorporated in adjacent buildings or expressed as separate enclosed elements to the main bridge structure. Bridges should exclude vehicular traffic by means of bollards or barriers, while allowing the passage of pedestrians.

Aesthetics

The preceding sections describe the principal considerations in the design of footbridges, including the development of an aesthetic consciousness for architects and engineers. The key factors that led to a strategy for the design of a footbridge, and the consideration of the details that need to be incorporated by architect and engineer, are necessarily subject to an overall aesthetic control, which should seek to unify the components of the design into the overall concept. The search for a methodology of bridge design using the critical superstructure and one that refers to context for the framing of each design opportunity is a basic requirement for achievement of a successful bridge. The German bridge designer Fritz Leonhardt has written extensively on aesthetic theory with regard to bridges in his book *Brücken: Ästhetik und Gestaltung* (Leonhardt, 1984).

For the architect and engineer collaborating on a footbridge design, even if neither believes in rules that would produce universal success, typical qualities continue to be used in describing buildings and structures and, despite their subjective nature, can be considered as desirable objectives.

The notion of elegance is wide-ranging, and architect and engineer may differ over its meaning. There are connotations of gracefulness, taste and refinement, which can be adopted in iconic structures to demonstrate national and cultural characteristics. For technicians, there is an implication of something done simply and effectively. If a differential equation governing a structural form is elegantly solved with its boundary conditions, is that structure intrinsically elegant? In the physical object itself, there is the overtone of elongation, grace and style.

There is a common assumption that footbridge design is necessarily about elegance because the refinement and attenuation of structural design made possible in this type of bridge reflects truth and economy of means. There may be other routes to other kinds of elegance. In the bridges across the M1, designed by the British engineer Owen Williams, there is no attenuation, yet the bridges are a product of the technology chosen to produce them – from reinforced concrete – and they have a robustness right for their location, and therefore an inherent elegance related to their material and their construction method. Similar examples may be found in timber, stone or steel bridges, each with its own generic form. If well executed, each has an elegance on its own terms, with its form justified by the material.

Systems of *proportion* can be used to order the relative dimensions of the principal elements of footbridges and their relationships to one other. In architecture, there is a long history of the use of proportional systems (Dunlap, 1998). The ancient ratio of the *golden section*, in which a whole is divided so that the two parts relate to one another as the larger does to the whole, has found use among contemporary bridge designers. There is a contemporary variation on the old theory, whereby other number systems are embedded in the proportioning of footbridge structures in an attempt to access an otherwise hidden harmony (Le Corbusier, 2000). This move away from a pragmatic, economic, analytical and structurally rational basis of design is part of a perceived lack, which must be overcome if a unified, pure structure of unarguable rightness in a perceptual sense is to be achieved. Mathematical systems can supplement intuitive methods born of visual design training but are not, however, fail-safe methods enabling engineers to achieve harmonious proportions.

It was once believed that buildings should be anthropomorphically proportioned and that the human body held ratios that formed the natural world and produced stable systems and structures. There is no evidence that ideal structural sizes and visually proportioned systems have any common origin. Our experience of many existing footbridges may set up an anticipation of how a bridge should be shaped but several modern examples go way beyond the expected to create bridges of extreme visual lightness.

The brief for even the simplest footbridge is often of sufficient complexity that the size of elements cannot be determined by structural criteria alone. This introduces an opportunity to tune a composition towards a visually harmonious whole, and a proportional system may have a role to play in its systematisation. The rise relative to span, the breadth and depth of beams, or the height of openings and their length and width all have a relationship, which should be tested in three-dimensional studies to ensure a sense of proportion. In tall cable-stayed structures, the relationship between the parts of the principal masts, above and below cable anchorages, needs careful study and perhaps referral to proportional systems as the size and importance of these elements is so greatly magnified within the overall composition.

Order in bridge design is normally considered desirable.

Order implies an architectural and structural coherence; a minimum of fuss, distraction and disharmony. Order means a well-edited design and a structure that uses a small vocabulary of different components, for example, steel of single types, such as circular hollow sections or parallel flange channels. From this simple language, a composition is built up, which concentrates on key conceptual objectives, such as the clear expression of the span, the way the decks land on the abutments or other concerns that are appropriate to their context. This vocabulary can be applied to different structures perceived as a group, for example, the motorway footbridges providing the signature to a route.

The intention is to reach a design that has an *inevitability* and a composition that would be difficult to alter.

Colour in footbridges should be used selectively and for a reason. Colour in bridges can be used to understate their form against the background – such as duck-egg blue or pale grey against the sky or dark grey against trees. Colour can also be used in this way to allow recession of elements or to bring them forward, such as the understatement of balustrades and parapets and the confident expression of primary structure. Used the right way, colour can enhance and bring out the essence of bridges and their position in the landscape. The use of strong colour that has an artificial provenance or appears foreign to its context plays with sensation and can look well for a short period, after which the effect palls.

The use of *visual corrections* in footbridge design to manipulate perception is, paradoxically, a neglected area of design where cost-inconsequential changes can add significant value. Physical adjustments to architectural elements are needed to prevent an incorrect perception of architectural form, space and line. Under the influence of gravity, one's perception of a long horizontal cornice line adds a sag. The Parthenon has a slight vertical camber to its entablature and cornice line to make it look straight. Bridges normally have a slight arching longitudinally and transversely, even when they are connecting banks at the same level. This may be a structural requirement or for drainage, but its provision will deal with the perceived sag in perfectly horizontal structures.

The eye can perceive horizontal distortions of about 1 in 300; sags appear more prominent than rises. It is usual to precamber a footbridge positive so that it falls towards the level under a live load. Dead load precambers, intended to come out as the bridge is constructed, seldom reach their calculated level, owing to secondary stiffness appearing in elements and on overestimation of material elasticity. Thus, if a footbridge is sized to deflect by 1/500 of its span, and if live and dead load deflections are comparable, it might be appropriate to set a precamber at 1/300 of the live load deflection limit and half as much again to allow for dead load deflection. The bridge should then stay within a visually imperceptible envelope of straightness, whatever actually happens.

Optical illusions or clashes that are evident only in three-dimensional studies may be entirely understated in two-dimensional drawings and need flushing out in the refinement of design drawings. This process in bridge design can apply to

all the components that go to make up the bridge and how it sits within its context, from structural bays down to the size and distribution of columns, the width of handrails and the spacing of balusters.

The issues of light and shadow, and the appearance of the bridge and its details in good weather and bad, with strong shadows or fog, should inform the final assessment of materials and form, detail and assembly, not only from a pragmatic and structural sense but also with reference to the final appearance.

The incorporation of *artwork* and the inclusion of an artist in a team working on bridges is increasingly common practice. This can work positively, with the artist contributing to the overall strategy, with particular elements or areas for elaboration. The important point is for the work to be in the spirit of the bridge design itself.

Ultimately, a balanced approach, integrating the disciplines of structural, architectural, landscape and artistic endeavour, is required to produce a coherent design composition.

Bridge types

Beams and cantilevers

A beam (Figure 17.4), also known as girder, is the simplest structural form for a bridge supported at each end, normally on abutments or piers. Generally a straight crossing, it has to resist gravity loads and lateral loads from wind, sideways forces generated by ordinary walking, impact loads and potential deliberate misuse. Decks are generally more wide than deep. Deck widths of 4–6 m provide plenty of capacity to sustain lateral loads over the spans typically encountered; the main design effort comes in managing structural depth and restraining top flanges prone to lateral buckling. In multispan bridges, bending moments within the continuous members invert themselves over each support so that vertically symmetrical sections with little or no differentiation along their length remain relatively efficient.

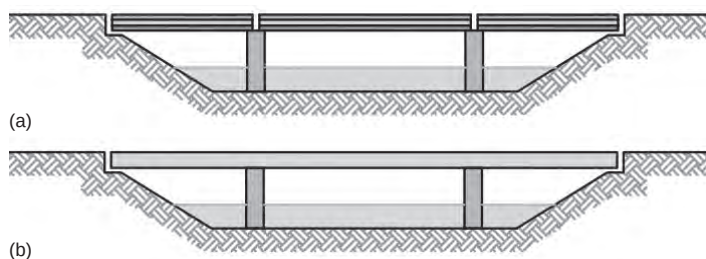


Figure 17.4 (a) Simply supported beams; (b) continuous beam

Similar in structural action to beams, cantilevers take their support from one side only, stretching outwards. They can be combined with simple beams to create a ‘suspended’ central span, at once visually expressive.

If simple bending forms are set up asymmetrically on plan, with either curving or serpentine decks, uneven load patterning

or asymmetrical pier locations, this might result in substantial twists, which will have to be resisted. This is best done using box section elements, cellular tubes with cross-sections of closed sides spread as far out as possible from a centre of rotation. Beams to resist combined bending and torsion forces tend towards forms similar to modern aircraft wings. Such profiles simultaneously minimise material used and maximise aerodynamic stability. The interior spaces of box beams must be carefully considered. At the scale of footbridges, these voids might be inaccessible.

Condensation can cause corrosion. Spaces may be fully sealed and pressure tested with a soap film at the time of manufacture but it is difficult to ensure long-term airtightness. Wax linings in enclosed steel spaces reduce corrosion risk.

The torsions set up by asymmetrical arrangements of supports can be sustained in the deck or the piers or in combination, depending on the visual emphasis to be given to each part. Piers tend to be cheaper and easier to build than deck elements, so substantial abutments and light spans are the norm.

Truss structures

Frames stretched out in space (Figure 17.5) have much better strength and stiffness to weight ratios than simple beams. Triangulated frames, pin-jointed, carry only axial forces in members, so that all parts can be arranged to be working at their limits. Originally developed to overcome the limitations of log sizes in timber buildings, assemblages of struts and ties really took off with the introduction of reliable rolled metal rails. They are easily analysed and the results readily checked. Elements are light to transport and assemble on-site. Construction inaccuracies are readily accommodated. Filigree trusses and geometric patterns capture space and light.

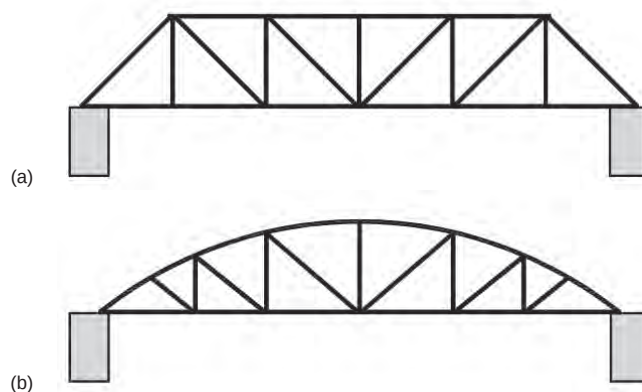


Figure 17.5 (a) Pratt truss; (b) bowstring truss

The mathematician Anthony George Maldon Michell (1870–1956) worked out the optimum forms for trusses in the early 1900s (Michell, 1904) (see Figure 17.6). Space frames in which the members are triangulated in three dimensions, often as assemblies of tetrahedral units, can sustain vertical and lateral loads with very sparse material use. Members interact

to support one another to avoid local buckling and the overall geometry can be adjusted to resist global instability.

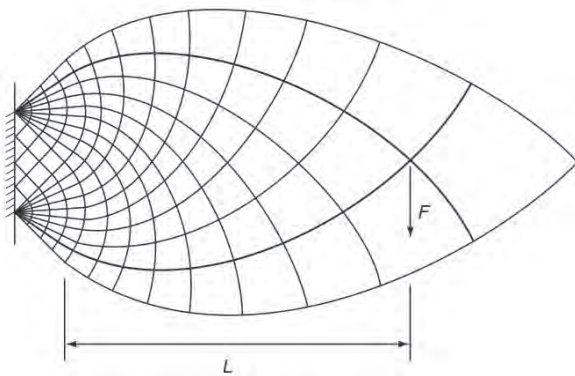


Figure 17.6 An ideal cantilever: Anthony Michell defined criteria for generating minimum weight structures in 1904 (courtesy of Techniker)

Arches

The oldest form of true structures – the arch (Figure 17.7) – acts by directing a line of compression within its cross-section to efficiently utilise its constituent material. This vector reaches its greatest magnitude at the springing points (supports), where the weight of the bridge is directed down and outwards, following the profile of the arch. Flatter structures generate higher spreading forces. For a given span, lateral reactions increase by the square as arch height diminishes. If surrounding conditions allow, these spreading forces can be fed directly into the foundations. Faces of firm rock permit low-profile arches to spring across deep ravines. If the ground is weak, very large thrust blocks, relying on friction not to move, must be provided; otherwise a tension member must connect the bases to

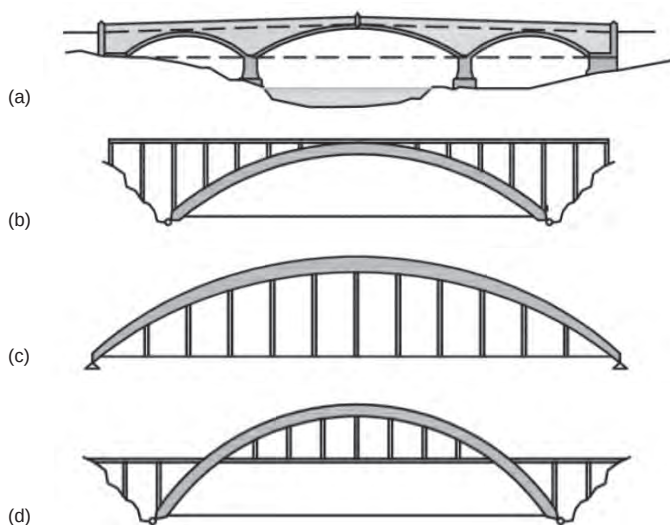


Figure 17.7 (a) Multispan masonry; (b) open spandrel; (c) tied arch; (d) part tied arch

make a self-contained system, making a tied arch. This tying element might be the walkway deck or part of it. Such long members must be sized not to stretch too much or to allow a snap-through instability. They add considerable weight to a design. In early examples of arch bridges, it was common to insert three pins, one at each support and one in the crown. This made analysis easy and neutralised the effects of thermal changes and differential settlements at the supports. The omission of the central pin results in a two-pinned arch, which is more efficient, but more difficult to analyse and has minimal foundations.

Fixed arches without any hinge joints are most efficient if ground conditions are adequate. The tendency for compression structures to shed energy by sudden and gross changes in configuration – buckling – must be dealt with, either by secondary members or sections built up to an adequate level of lateral stiffness.

Setting out an arch on its ideal profile is not always straightforward. The ring itself, supporting only its self-weight, should follow a catenary, the shape of a hanging chain. This is a transcendental curve, described by trigonometric functions. A weightless chain supporting an evenly loaded deck falls into a parabola, an algebraic curve of great elegance.

A real arch must be somewhere between the two; a traditional voussoir arch, depending on how it is constructed, may move between the two as it comes into being. Circular arches, semicircles and arcs, have a visual repose and completeness.

The ancient Romans were well aware that, in an arcade, arches have the advantage of being self-equilibrating. The loss of one bay will not collapse the entire system and requires no visually differentiated end bay.

Arch rings curving on plan generate large sideways forces. These resolve themselves if arches are symmetrically opposed; otherwise, a connection must be made to an additional sideways spanning element, usually the deck. There will be heavy additions of structure to take out these forces and the extra torsions that come with uneven live loadings.

Cable-stayed bridges

Walkways supported directly from ties off pylons are comparatively simple structural systems. The relative stiffness of the deck and supporting array can be manipulated so that a light deck has a more profuse support and a heavier deck has a more dispersed system. Different cable configurations – fan, harp or radial systems – contribute different degrees of repose to the bridge's appearance (see Figure 17.8). Distributing anchorage points carefully can reduce pylon cross-sections. Cable-stayed bridges are practical to construct, either sequentially outwards at large scale or by rigging at typical footbridge size. It is practical to gather a number of stays supporting a walkway curving on plan then adjust the main strut angle to obtain a stable and economic configuration. The dynamism of a slanting tower can contribute to a balanced composition but an overall visual 'thickening' of components is to be guarded against.

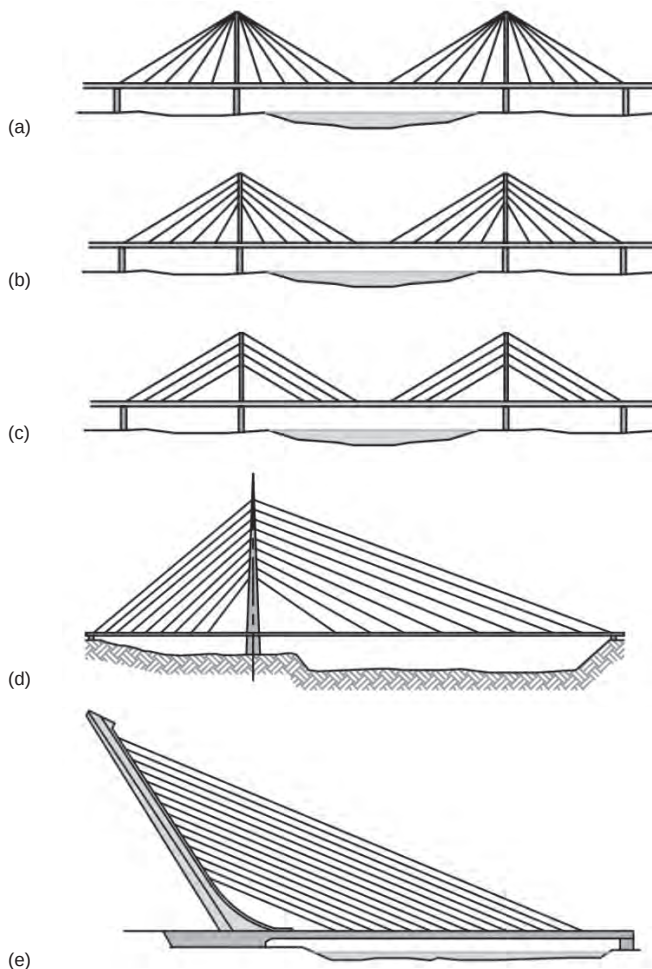


Figure 17.8 (a, b, c) Basic cable-stayed types; (d) asymmetrical cable-stayed; (e) cantilever spar cable-stayed

Suspension bridges

Closely related to arches in structural terms, suspension structures (Figure 17.9) also have a long history of providing practical footbridges, particularly over wider spans. The profligacy of chain and link systems, where a significant proportion of material is employed to form pin plates and pins or bending around link ends, is now all replaced with increasingly strong types of ties, helical cables or parallel strands of hard drawn wire or polyaramid. That the main part of the structure, the suspension cable, is fully and evenly stressed without any possibility of buckling gives the form its pre-eminence over long spans. The weight of the deck, pylons and hangers becomes less significant overall as spans increase. Very large cables can be readily constructed by spinning, laying individual light strands close to one another to make a substantial rope.

Conventional configurations of suspension footbridges owe more to the historical development of cabling and tower construction than to any original development of the form. Overall flexibility needs careful control at such a small scale.

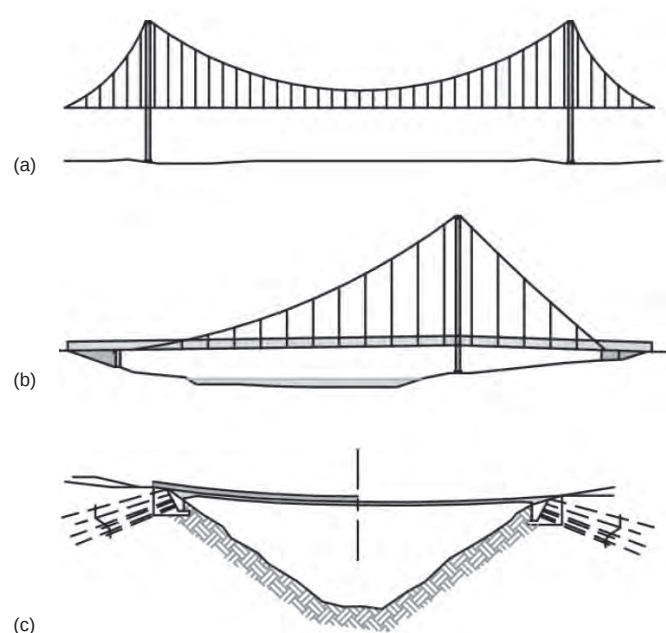


Figure 17.9 (a) Suspension bridge; (b) asymmetric suspension bridge; (c) stressed-ribbon bridge

It is usual to stiffen the deck, often with parapet beams or trusses. An alternative is to stress the deck in a curve opposing the main cable so that a cable truss is formed. For crossings over water, towers are commonly landside, with backstays to tension anchorages beyond. This is an important and expensive area of suspension footbridge design. Mass abutments will be very big not to slide but this may be unavoidable if a suitable substrate cannot be found for tension fixtures, piles or ground anchorages. Catenary side spans are unusual, so channelling the high lateral loads at each end as a compression back through the deck is seldom an option. Serpentine plans can be made to balance with slanted pylons. The deck beams will need additional lateral and torsional stiffening and the abutments are sometimes hard to hide.

The deck weight decreases as the main cable sag deepens. As the profile becomes flatter, the dynamics become less manageable and the deck must be made correspondingly more rigid. In the limit, a stressed ribbon is achieved. The deck becomes a solid band of high-strength prestressed concrete carried on straps. Aerodynamic damping can be readily exploited by adjusting the profile of the bridge but if insufficient stiffness is given to the main girder then excessive movement and pedestrian excitation is inevitable.

Stressed-ribbon bridges

A stressed-ribbon bridge can be considered a type of suspension bridge. As the suspension cable profile is reduced at its limit, a stressed-ribbon bridge is reached, where deck and suspension cables are coincident. Therefore, a stressed-ribbon bridge is a tension structure with low sagging tensioned cables and prestressed concrete deck segments.

Moving footbridges

The additional complexity of moving bridges opens up new avenues for architectural manipulation and sculptural expression of the working parts (Figure 17.10). Opening bridges are usually required in situations where shipping requires large clearances and include swing bridges, trundle bridges, lifting bridges and transporter bridges. The housing and revealing of hydraulic plant, the wonder of the coming together and separation of large pieces of structure, the scope for enhancement of counterweights and the architectural and sculptural potential of the changing perception of the structure and its surroundings, should be controlled not only in the structural sense, but in order to extract the maximum potential from the type in urban, architectural, landscape and sculptural terms. Opening and movable bridges are a special structural case.

The essential strategy is to avoid lifting dead weight. A straight lift on a 60m footbridge might be 15m up on a 100t structure, equivalent to the expenditure of 15MJ of energy (4km h). A bascule would use only 500kJ and a horizontal

swing bridge less than 200kJ, depending on the sophistication of its slewing ring and maintenance.

Generally, the magnitude of the mechanisms and gear and their concealment is significant and requires specialist mechanical and electrical engineers' involvement from the outset.

Enclosed footbridges and sky bridges

Footbridges sometimes need enclosure, for example where bridges connect buildings at high level, in urban situations or where walkways require enclosure for reasons of safety or vandalism, or on sites that are exposed to bad weather. Enclosed bridges vary from those covered to keep snow clear of the crossing, to prevent a build-up of additional dead loading, to those covered for climatic reasons or sealed to maintain an air-conditioned route. Internal clearances vary from 2.3m for pathways to 2.4m for combined pedestrian–cyclist routes and 3.7m for equestrian bridges. Clearance envelopes must be maintained on decks, stairs and ramps. Aerodynamic effects are magnified and will need detailed consideration.

An air-conditioned covered footbridge in Hong Kong and a rainproof ambient covering in the City of London bring bridges into the realm of urban or architectural components and should be considered accordingly. There could be a debate that the expression of the structure as a bridge is so understated in this case that the problem is an architectural one, yet there are examples of enclosed bridges that emphasise the ambiguity between identity as a bridge and definition as a piece of architecture.

Cladding to covered bridges needs support, either from the primary structure, evident in through trusses, or as a separate structure, supported off the deck. In the former, cladding should be developed in sympathy with the structure and coordinated with the principal members, with glazing following the dimensional setting out and rhythms of the primary structure. Maintenance of covered structures should be considered at the outset and might require travelling gantries, which allow access to the entire envelope, if otherwise inaccessible from remote access platforms.

Bridge types – subdivisions

Within the classification of each of the spatial types discussed comes a subdivision into structures made out of the major structural materials: steel, either high-tensile or stainless, reinforced concrete, masonry, timber, aluminium and plastic.

The nuances of form are then determined by the relative strength to elasticity of the materials, their long-term behaviour, robustness and fabrication methods. Steel is a beautiful structural material, in a variety of strengths, one with a clearly defined pattern of elastic behaviour, with a worldwide procurement base and expertise in its use and fabrication. Aluminium can be strong and is highly resistant to corrosion. Despite being relatively lightweight, its softness and high elasticity will always make its structures appear burlier than those of steel.



Figure 17.10 Gateshead Millennium Bridge – tilt bridge, Wilkinson Eyre/Gifford (copyright Graeme Peacock)

Weathering steel is a high-strength alloy developed to patinate with oxide and therefore requires no painting. It may continue to corrode in marine environments and needs detailing to control rust staining in water run-offs.

Stainless steel footbridges are an expensive use of the material, particularly as their fabrication requires a clean bed, free of all iron contamination, to avoid bimetallic corrosion. Duplex steels are not connected to the molybdenum market and so are relatively price stable. If a surface finish is carefully specified, the structure is effectively maintenance-free. Mirror polishing is difficult to keep up, as stainless steel does indeed weather.

Masonry is just again coming into its own, as water-cutting fabrication techniques now permit refined levels of stereotomy and better computer modelling is available to assess unpredictability of behaviour.

Reinforced concrete strengths have increased, so concrete sections can approach much slimmer sizes. Shrinkage cracking, overall shrinkage and long-term creep movements remain critical areas of research in developing concrete use.

Significant advances are being made in timber products. The natural material's characteristics, variable strength and elasticity and propensity for moisture movement and long-term creep under load are being mitigated by better forestry, quality-controlled selection, laminating processes and applications of pressure and humidification to set a timber's condition. Genetically modified timbers for structural use have yet to appear.

Another categorisation of footbridge types would be to divide them according to the way they deal with self-weight support, on the one hand, and localised imposed loading, on the other. This classification accommodates the various hybrids of the simple taxonomy given previously. A major system – beam, cantilever, arch or catenary – is shaped to support the dead load condition and a stiffening element, such as a beam, cable fan or secondary arcade, is superimposed to distribute and subdue point load intrusions into the balanced whole. The two functions may be fused. A bridge arch may be thick enough to bend locally under uneven loadings. Robert Maillart made his arch rings as light as practical, throwing all the stiffening of his bridges into the parapets or deck. The Pont des Arts in Paris, by Louis Alexandre de Cessart, superimposes arches to disperse point loads. Heavy chain suspension bridges have enough apparent stiffness in their superstructure not to need deck stiffening; this is the so-called small deflection principle. Stressed ribbons depend on the stiffest possible deck component for the serviceability of their behaviour under patch loading.

The ideals and applicability of prestressing range across all types of footbridge structures. Forces introduced by internal or external tendons or jacking and wedging across joints are used to redirect actions in imperfect forms. Eugène Freyssinet (1879–1962) first came up with the process in 1911 to win a tender for three arch bridges made of reinforced concrete (Billington, 1976). The purpose of the development was entirely commercial; the material saved outweighed the energy cost of implementation. Prestressing is now sometimes applied

to attenuate member sizes for visual or functional reasons. Additional benefits include an alternative approach to thermal control, constraining expansions within stressed members to avoid movement joints. The longevity of reinforced concrete is improved, as microcracks are forced shut. The phenomenon of apparent stiffness can also be exploited, whereby tensile members can be prestressed to provide a compressive component to the overall system and thereby reduce deflections in certain load cases.

Some footbridges have earned the accolade of being 'art bridges'. Monolithic forms determined by the sculptor's eye are created and analysed using modern computational power. Provided that stability and deflections are all acceptable and the von Mises stresses within the constituent material are not exceeded, the bridge will stand and no structural designation is needed.

Interest in older types of footbridge has not waned completely. Superannuated forms and materials retain their value to designers in two ways. They offer a source of forms, marginalia, on which to draw, distanced from modern convention. The reasons for their superannuation provide insight into considerations to be addressed and otherwise ignored.

Materials and finishes

Materials

The choice of materials to use for the primary structure is obviously related to the structural form and economy of means. Historically, there were few choices beyond masonry or timber, yet now, despite the variety of choice of materials, the old structural forms are still followed. Hybrid forms and hybrid bridge compositions, where different materials are combined, are also possible and can be used to great effect. Beyond the technical, cost and sustainability issues regarding the use and sourcing of materials, there are architectural decisions relating to the nature and expressiveness of the structure, which will indicate a certain choice of material and thereby influence the character and identity of the bridge.

Bridge building materials have varied across time. Originally, timber and organic materials provided the basis of real structures. Usable baulks could reach a scale of 12 m through growth but were often limited to 5 or 6 m by handling requirements. Trusses originated in the strategies needed to combine stick elements. The mental tool of superposition, the building up of larger systems from simpler subassemblies, seems to have governed much of this development. In China, metallurgical innovations to make weapons allowed the forging of chains reliable enough to replace the lianas and hemp ropes of the most ancient suspension bridges. Stone projected power and permanence, and stereotomy, the techniques of setting out cut stone, trammelled the development of the masonry arch. Cramps and superimposed load systems went some way to resolving the weakness of the material in tension. The engineer and mathematician Jacques Heyman has demonstrated how the very low stress levels in masonry mean that the forms employed

are governed by geometry, overall stability and shape, rather than strength.

Materials for footbridges have to be light and strong, with as much stiffness as possible. In addition to longevity, they must be robust and resistant to deterioration. All matter ages, either changing state, oxidising in the weather, or by the direct agency of rot or attrition. Over 120 years, accidents can be expected. The property of ductility, the ability to yield gradually under high local impacts, is very desirable.

Stonework must be set according to its grain and protected from the weather with capping or non-load-bearing copings, which are sacrificial. Some stones do not weather well when set next to one another. It is important to season stone before it is placed. Bricks rely on surface firing and need careful quality control to be built into a bridge. Traditionally, the in-fill above a brick arch is most important and should be either as impermeable as possible or free-draining to limit moisture penetration downwards.

Timber is subject to organic agencies. Good hardwoods are naturally resistant but structural grades of timber are generally the poorest qualities of wood, excepting packing and pulping, so protection is needed. Toxic treatments are increasingly limited by legislation. Heat treating and mechanical surface treatments, compressing the timber fibres to enhance impermeability, are alternative forms of preservation. End grains need protection from the rain and joints should be detailed to run off any penetrations.

Higher-strength steels have reduced ductility. Hanger rods, particularly, must be protected from impact and checked against fracture under extreme cold. Limited areas of material can help keep details compact but welding, testing and defect repair must be carefully planned for. Steel needs barrier protection to resist oxidation or sacrificial coatings, such as galvanising or zinc spraying. A self-healing coating suits the inevitability of tool damage and sling burn during erection.

Of the architectural considerations regarding choice of materials, it is sufficient to mention tactility and reflectance, the ease on the eye and the connotations associated with certain materials. Timber promenades evoke wooden decks on boats and piers. Steel trusses are most common in industrial plant. Brick pavements are reminiscent of Britain's early industrial heritage. Consideration of the context normally leads to a choice of materials, unless the bridge is creating a new context and is the primary visual or architectural force, when new and different materials can be utilised.

Finishes

The way a footbridge shows in daylight depends on finishes. The durability and corrosion resistance of the principal finishes are critical to the long-term appearance and serviceability. The expected design life of the components will have been reviewed within the brief. Secondary elements of the bridge, such as the deck surface, balustrading, handrails and parapet in-fill, all have a requirement for a particular finish, regardless of the actual material chosen. Surface wear, abrasion and impact will

influence the choice of finish to provide toughness, smoothness on handrails and friction on decks.

The flow of water, rain driven by wind, across the structure must be controlled. Features, textures, absorption all play a role. Strakes intended to preclude aerodynamic effect become drip mouldings. The reflectance of soffits over water is important if caustics, light reflections, are to be part of the composition.

Brushed surfaces make planes indefinite in light and soften the overall appearance, while dark forms are recessive and light objects stand out.

Bridge details

Every detail of a footbridge can be experienced at a walking pace; this demands good design and choice of components, and that construction of components is done well. Scale, tactility, colour, reflectance, mass, rhythm and line can all play their part. Using a footbridge, the act of walking on air, between land, under a sky, on a deck, running a hand along a rail, looking out and along and back, is a simple and sensual experience, perhaps a delight. The latent sensuality of the limited number of components that contribute to this, the walking surface, the handrail, the lighting and aspect are all critical. They have to be right for their purpose and their particular context. The positioning of light sources, the reflectance of surfaces and coloration all contribute to the functional use of the bridge and are fundamental to its architectural expression. Transparency and opacity can affect the emotional impact of crossing.

Coincident with architectural treatment, the detailing of a structure has several distinct functions, conjoined or independently conceived. Loads have to be transferred, and this function can be expressed or suppressed. Construction can be demonstrated in a mannered way or made invisible in the finished product.

Decks and walkway surfaces

Bridge decks need to feel and look right for their particular use and location. Bridge decks must be slip resistant, durable, allow drainage and be waterproofed to resist deterioration. The particular requirements for walkways suitable for pedestrian, cycle and equestrian traffic need to be addressed if the bridge has shared users.

The design and specification of a bridge deck walkway surface should seek to provide a surface that avoids slips and falls. A standard slip test, using a device that emulates a human heel losing adhesion, is used to assess prototypes of proposed surfaces and completed installations. Currently, the anti-slip value of traversed decks referred to in CD 353 Design criteria for footbridges (Highways England, 2020a) is specified as equivalent to a mean corrected pendulum test value of 45 units using a standard skid resistance pendulum test in accordance with BS EN 13036-4:2011 (BSI, 2011a). Designers should consider the expected user groups and specify accordingly.

The reflectance of the walking surface is critical to the feel of airiness at deck level. Fugitive colours give way to hard-

wearing greys and neutral tones. All wearing surfaces must have through colours and be readily replaceable.

Timber decking needs to be a group 5 hardwood or harder to survive regular traffic for longer than 5 years, typically English oak, iroko or jarrah. Timber surfaces are subject to wear, especially on stair and step nosing; a timber of grade 5 or above should be used. Planks are often set with open joints, less than 5 mm wide to allow drainage. The absence of tongue and groove joints makes the deck thicker, as each plank must be capable of bearing its point load. Plank surfaces can be machined to profile but still become greasy in the wet. Timber planking is now usually inset with carborundum strips or painted with a grit coating. Travertine and flagstones remain the perfect pavements but add superimposed dead load to lightweight schemes. Steel plates require protection as well as an anti-slip surface and will become extremely hot in sunshine or, at about -5°C , cold enough to stick wet skin. Deck paints, generally epoxy coatings with a filler of grit, are efficient, economic and readily brought back by overcoating. Unevenness caused by weld distortions must be made up using resin fillers, so that surface deviations are generally less than 3 mm in any metre length. It may be preferable to use a proprietary levelling screed, which is a thin film liquid poured into place to set level.

Durbar plate – galvanised steel sheet that has been embossed with a diamond pattern to improve adhesion – is effective and gives walkways an industrial feel. Galvanised open mesh panels are an option, although vertigo and high heels are an issue. Variations on roughness of edge are also possible. Anti-slip values need to be checked.

Concrete surfaces are prone to wear and tend to dust, although silicon sealers can improve the surface robustness. Various treatments to expose an aggregate surface can be used, including brushing shortly after casting, acid etching, blasting and mechanical picking. Precast panels that can be quality controlled in factory conditions can be improved with steam curing and table compaction systems.

Glass, laminated panels and cast-glass units tend to polish rather than scratch. Good slip resistance is obtained by fritting the surface, spraying on glass beading. Blasting back the surface in a pattern is less successful but cheaper, and almost any pattern can be easily stencilled. Only shallow embossing of less than 2 mm is possible on toughened glass; this will wear off over approximately 5 years of steady use. The thicknesses of glass used in decking tends to show green unless iron-free material is used. Cementitious grouts weather better than silicon seals, which can show algae growth. Rigorous risk assessment procedures are required regarding the use of glass in footbridge designs.

For equestrian bridges, there are further considerations to do with the noise generated by hooves on walkways; particular attention should be given to this. Appropriate surfacing or warning signs are advised.

Designers should make their own assessments regarding the suitability of materials in respect of the particular bridge location, use and design life of materials in relation to the

requirements as outlined here and elsewhere in relevant codes and standards.

Generally, any bridge design should specify the design life of deck surfaces, with period to first maintenance or replacement. This should form part of a comprehensive operations and maintenance manual.

Other elements

Parapets

The parapets of a footbridge are key visual elements in its design. As structural elements, they are associated with relatively light loadings. Deciding on the relative importance and visual expression of the parapet in comparison with the structure of the bridge is one of the critical factors in a bridge composition. Footbridge structures that achieve a high degree of slenderness and attenuation need a subtle and well-integrated parapet design, to avoid their structural elegance being overpowered by a bulky and inappropriate balustrade or parapet.

Balustrade posts are the perfect scale for light castings, aluminium or ductile iron. All fixings can be concentrated into the complexity of the element, with simple rails, infills and base plates arranged between them.

All bridge spans, ramps and stairs require parapets. Dimensional criteria should be checked against relevant codes and standards. Handrails should be provided on both sides of the stairs, ramps and decks where the gradient is steeper than 1 in 20. Generally, handrails should be designed in accordance with BS 8300-1 (BSI, 2018). Reference should be made to mandatory or guidance notes issued by rail, waterway and highway authorities and current relevant local, national and international codes and standards to confirm dimensions. These include building regulations, British standards and European norms.

Strainer wires used in balustrades, such as 4 mm lifeline cables, are stressed to stay straight. If they are not to deflect unduly, typically no more than 20 mm under lateral load, they must be preloaded to approximately 5 kN tensile force each between posts typically 1.5 m apart and will then need end posts capable of resisting lateral loads of 50 kN/m.

Panels used in balustrades are usually set between uprights with top rails. These posts might be at 1.5–1.8 m centres so that post and base fixing do not become excessively large. Panels of aluminium will then be 8 mm thick if punched less than 30% and lipped along each edge. Glass will be 12 mm thick, an economic thickness. Steel plate, 4 mm thick, will be practical, and any number of variations on ferrocement and concrete panelling will be available. Profiling panels with corrugations up to 30 or 50 mm deep will generally halve thicknesses. All dimensions must be checked by calculation.

Vandalism is best dealt with by robust, non-absorbent, low-key surfaces. Detailing should be strong and simple. Polished concrete is very resistant to casual damage. Balustrades over motorways or walkways in areas prone to vandalism may need

to be built up higher or fully enclosed and infills brought down to 38 mm gaps or to mesh size, to control throwing. Where bridges are enclosed for anti-vandalism reasons, unclimbable fine stainless steel mesh cladding is normally preferable.

Parapets do not always have to be light and transparent. Solid infills can be successful if they are part of a deliberate overall concept for the shape, form and expression of a bridge.

Solid parapets work with above-deck structures, such as steel beam or truss designs, and can be used to manipulate the bridge crossing experience, giving an increased sense of enclosure and security, as well as controlling views. Solid parapets on slowly curving structures can be dramatic, yet there is the fundamental requirement for security, seeing where you are going and who is around the corner, so care is required when setting out. Solid parapets can be utilised for equestrian bridges at lower levels to shield horses from the distraction of traffic passing below. Open parapets of mesh, which are very transparent on elevation, look solid when viewed obliquely, and, if this is not anticipated, can radically alter the intended look of a bridge and the balance regarding the expression of structure.

Parapet infills often become the area where architects or artists seek to put their stamp on a bridge. Unless it suits the bridge and its overall concept, this is where a collaboration can go wrong. The expression of the parapet comes from its functional requirements but is driven by an architectural or structural concept, which has been developed following the consideration of the context.

Balustrade infills are critical to the main elevations of the bridge and define its appearance. Parapets are often integrated with the structure zone to understate the overall thickness and build-up of a footbridge. Climability of any part of the parapets must be prevented, including diagonal structural members, if employed in the design. The maximum gap between elements should be 100 mm. Climbing of the parapet and in-fill must be discouraged by use of vertical uprights, mesh, rails or strainer wires, which are angled inwards to prevent scaling.

Parapets are the principal elements manipulated by aerodynamicists to stabilise bridge decks. A degree of perforation, around 30%, is often sufficient to provide aerodynamic damping, like a soup strainer waved in the air, to an otherwise unstable cross-section.

Parapet panels are often sized to be replaced by two men to limit the impact of vandalism. A panel of perforated aluminium sufficient to sustain the required lateral forces and 1 m high by 2 m wide, with a weight of less than 50 kg, is practical.

Ramps and inclined bridge decks

Fully accessible ramps should normally be designed to BS 8300-1 (BSI, 2018) and the building regulations. Ramps for pedestrians, cyclists and equestrians are ideally not steeper than 1 in 20. Ramps can be steeper, if justified by reason of economy in setting out, or for sites with limited space. Ramps at 1 in 15 and steepest at 1 in 12 are possible, subject to the provision of landings at intervals. Landings may need to be offset, subject to available space, for added safety. Sloping bridge

decks are also ideally set at a maximum gradient of 1 in 20. Landings on bridge decks are difficult to integrate successfully. If a bridge deck does need to exceed a slope of 1 in 20 or a particular structural type utilises variable gradients, landings may be waived, but only 'in exceptional circumstances and with the agreement of local access and disability groups' (Highways England, CD 353, 2020a). It may be permissible to reduce the access requirements and to omit ramps for rural footbridges that are only reached by rural or substandard pathways and are already inaccessible to mobility impaired users. However, the possibility of approaches being upgraded over the life of the bridge should be considered.

Stairs

Stairs to footbridges should be designed in accordance with current codes, standards and building regulations. Standard BS 5395 (BSI, 1984, 1985, 2010, 2011b) sets out requirements for public stairs. Note the additional requirements to these standards specified by CD 353 (Highways England, 2020a).

Foundations

Proper consideration of soil–structure interaction has the greatest potential for new developments in bridge form. Exploitation of different ground conditions has the most immediate influence on designs intended to reflect their environment. The low arches of the Swiss engineer Christian Menn rely for their economy on precipitous rock faces providing abutments. Footbridges in the alluvial plains must sustain local settlements much more readily.

Bearings

Structural articulations are among the most expressive parts of a bridge. Footbridges do not generally need elaborate bearings. Elastomeric build-ups, sandwiches of steel and rubber that can support substantial vertical loads while sustaining horizontal movement, are usually sufficient. Roller bearings tend to yield and flatten, becoming ineffective. Spherical Teflon-coated bearings are reliable and efficient. Jacking points are essential for bearing replacement. Bearings are generally an expensive component of a footbridge and need careful consideration of whole-life value. Bearings also require ready access for inspection and must be designed with packing plates and room for removal and replacement.

Movement joints

The usual scale of footbridges seems to throw up interesting decision boundaries regarding the control of thermal movement. Temperature swings winter to summer can be up to 65°C (+38°C to –27°C) in Britain and 90°C (+35°C to –55°C) in Scandinavia. Insolation, sunshine falling, particularly on one side of the structure, complicates the issue. Charles Coulomb's (1736–1806) suspension bridge across the Seine in Paris was

not completed because the parallel arrays of chains exhibited too much thermal movement. Surface temperatures can reach as high as 75°C.

The coefficients of expansion of steel and concrete are not far apart. Typically, in a steel or concrete footbridge, joints with a movement envelope of 20mm might be expected at about 30–50m intervals. Timber moves about a third as much but has the added complexities of changing dimension with varying humidity and creeping under long-term load. In the humidity range experienced in the UK (120%–60% relative humidity), a timber truss structure might alter as much as 120%–15% in dimensions.

Lighting

Functional lighting of the deck surface is essential, preferably by way of general urban lighting, low-level spill lighting or lighting associated with handrails or parapets. The use of separate lamp posts is barely acceptable for road bridges and should be avoided on footbridges.

Bridges can be floodlit to emphasise their presence in the landscape. This requires the utmost care to avoid light pollution. The relative balance of the illumination of adjacent buildings and structures in a city is complicated. A bridge might be better underlit in the urban environment.

If lights set outside the structure are used, mountings can be combined with adjacent or high-level structures or the bridge could be lit remotely; however, the overriding consideration is to work with the primary and predominant elements of the design. For example, for a structure in which the cable geometry is understated but emphasis is placed on a beautiful arch, the lighting should work with that balance, picking up the primary form and underlighting the secondary elements. An unselective blast of lighting, illuminating parapets, deck, arch and beam, leads to glare. Floodlighting rarely improves a simple functional scheme that illuminates the walking plane.

Additional care of bridge lighting is needed over river crossings. Reflectivity over water also brings other opportunities and, if done well, can really bring a bridge alive, especially if the waters are slow moving, offering mirror images of the lighting scheme.

Deck lighting can be concealed beneath handrails but the throw will be limited to approximately 2m and will be uneven. A continuous line of light along the bridge emphasises any imperfections in its camber. Point sources concealed in the balustrade uprights are more forgiving. Lighting fixings should be extremely robust and resistant to vandalism. Lighting should generally conform to BS 5489 (BSI, 2020).

Signage

Signage of footbridge approaches may be necessary to indicate destination and the presence of combined or separated footpaths and cycleways. Signage should be incorporated into the design from the outset. Signage on the bridge itself should be avoided and if done as an afterthought can ruin the lines of a

carefully judged structure. Visual clarity of both the design and the route is critical and the distracting clutter of signs on or near a bridge should be avoided.

Illuminated signage or signage connected with the thoroughfare of the bridge crossing, such as road signs or river navigational lights for shipping, needs attention so that its incorporation appears deliberate.

Construction

The way a bridge is constructed can have its own subtlety, recording the preoccupations of designers or contextual issues, such as environmentally sensitive banks along waterways. At the scale of a footbridge, prefabrication is completely practical, its size being suited to medium-sized assembly buildings, redundant airfield hangars or shipyards. Working in factory conditions in familiar surroundings is much safer for operatives, both immediately and over the lifespan of a career in shelter rather than the open. Build quality will be better, and more sophisticated joints, requiring less maintenance, can be made economically. Small bridges can go in one piece. At 30m span, a four-lane highway width, the target weight of a tonne per metre run of length neatly fits in with the capacity of a 30m long 30t weight unescorted lorry.

The parts of a bridge can be manufactured in many different ways. Balustrades might be extruded or built up from flats. Plate girders can suffer from local distortion; cooling welds pull panels out of plane to become more susceptible to buckling failures. The maintenance manual is central to the total life cost of a bridge structure. A good appearance must be maintained. Far too often this is taken to be keeping up the pristine appearance of a newly constructed object. Only occasionally are changing visual aspects and the ageing process considered in designs.

There is a hierarchy of components, descending towards the anthropomorphic scale. A main girder may be sized to match the scale of a mobile crane, whose size is determined by the motorway networks. Deck panels may have joints of ordinary road width. Balustrade uprights can be lifted and fixed by two men. In-fill panels are completed with connections tightened with spanners. Handrails have screws.

Erection procedures have three principal variations. It is increasingly common to use the heavy lifting equipment available to assemble a footbridge structure adjacent to its location and then simply lift it into place. The route or obstacle to be crossed suffers the minimum disruption. Alternatively, construction can be completed adjacent to a final position and the bridge jettied into position. The weight of a footbridge and the glide plates available, with frictions forces as small as 10% of the mass, mean that little shove and braking capacity is needed to move a structure into position. There is a balance to be struck between the efficiency of a finished frame with integral strengthening for the jetting stage and one with a temporary frame added to assist the process and removed later. Finally, there is construction in place. Building on-site makes for dangerous work and increased risk to the construction itself

during assembly, from storm or other inclemency. It may be essential, owing to scale or for economy, if transport is confined or fabrication space difficult to acquire elsewhere.

The final fit-up of large components must be carefully considered. The end pinholes on very large plate girders are sometimes milled on-site, a laborious process, to take account of dimensional inaccuracies resulting from thermal differences between the manufacturing facility and the site. Complex welding fabrications and those precambered by heating can suffer shake out, changes out of tolerance as weld distortions are redistributed during handling and transportation.

The process of jacking a structure, usually in prestressed concrete or composite steel and concrete structures, can be exploited as part of a bridge's expression. The end anchorages of the Swiss engineer Jürg Conzett's Pünt da Suransuns have a clearly mannered set of hasps and wedges, displaying how the bridge has been manipulated into its service condition.

Tool damage, sling burn and a variety of impact injuries are not to be underestimated during construction. Most of the damage an ordinary bridge structure will suffer over its life is inflicted by the contractor during its installation. Most protection systems are best applied under factory conditions; very few are specifically designed for the patch repairs that are almost inevitable. The proper specification of lifting points and physical protection systems, temporary wrappings and edge protectors is desirable but seemingly inordinately expensive in current procurement conditions.

Maintenance manuals and whole-life assessments of footbridges should address the decommissioning procedure. In the dismantling of prestressed structures, this foresight is particularly important. The embodied energy of a reused structure is favourable to the provision of configurations and joints that permit relocation. Keeping material out of the ground, creating a superstructure that touches the earth in the lightest manner possible, must, if recycling is managed properly, result in the best long-term use of resources.

Maintenance issues have been the poor relation in design work, a situation that is no longer sustainable. It may not be necessary to minimise maintenance but it is increasingly important to get the most value from the use of a material, husbanding it to last as long as possible in a condition to be reused wherever possible.

Sustainability

The concept of sustainable development is a response to destructive asocial and environmental effects of the existing approach to economic growth. Sustainable development can also be considered as an approach to development that focuses on integrating economic activity with environmental protection and social concerns.

Construction industry and sustainability are closely linked, with many aspects of sustainability depending directly and significantly on appropriate and timely actions by designers. Engineering design is a very important part of the overall engineering process of analysis, synthesis, evaluation and

execution and its input in sustainability solutions must be provided in partnership with many other interests. This is to

- drive down the adverse environmental and social aspects of engineered products, services and infrastructure
- improve their environmental performance
- improve contributions to a high quality of life.

To ensure a sustainable project, it is prudent to develop a sustainability strategy from the outset and ensure that all stakeholders are supportive of its aims and objectives.

An example of sustainable footbridge building is to be found at The Tide, an elevated riverside park on Greenwich Peninsula, London, where a combination of footbridges and elevated walkways has enhanced the area.

Ten examples

Redhayes Bridge, M5, Exeter, WSP and Design Engine, 2011

Consisting of two inclined steel arches from which a steel deck is suspended on cables (Figure 17.11), Redhayes was the first bridge in the UK to be designed and constructed to the complete suite of Eurocodes.



Figure 17.11 Completed bridge, showing the simple yet elegant structure over the motorway

In conjunction with Design Engine, WSP supported Devon County Council in the design, supervision and construction of the Redhayes pedestrian and cycle bridge, which spans 100m over the M5 motorway near Exeter and reinstates a former public right of way on an existing east–west route that was lost when the M5 was built in the mid 1970s. The alignment of the bridge is designed to make best use of the existing country lane network, connecting separated sections of both the National Cycle Network and the Exeter walking network on opposite sides of the motorway.

The bridge is supported on reinforced concrete abutments consisting of springing blocks (supporting the arches) and bank seat abutments (supporting each end of the decking).

The concept of the bridge falls into two parts, the generation of a steel arch, which creates a strong and dramatic gateway to Exeter over the motorway, and the bridge deck, which is

conceived as a suspended landscape pathway connecting the two sides.

The bridge was designed and fabricated so that it could be lifted into position as one complete structure, minimising installation time and disruption to road users.

TRUMPF Footbridge, Ditzingen, Schlaich Bergermann Partner, 2018

The TRUMPF footbridge (Figure 17.12) crosses Gerlinger Strasse in Ditzingen. The design of this bridge combines engineering excellence with architectural vision and technology to create a state-of-the-art structure. The 28 m span bridge with 2.2 m wide running area is a lightweight shell construction, formed from one continuous, doubly curved stainless steel sheet, which, owing to the high efficiency of the supporting structure, is only 20 mm thick. To stabilise the free edges of the shell, they are folded downwards by 200 mm. These collars merge at four main supports, forming a stiff triangular section. The membrane forces from the shell are supported by foundation sockets of reinforced concrete in pairs, connected in the transverse direction by concrete ties. Micro-piles are used to transfer the loads. Abutments provide vertical and horizontal support to the deck, while allowing it to move longitudinally.



Figure 17.12 Shallow arch effect with glass parapet providing minimal visual impact (© sbp/Andreas)

Pedestrians walk directly on the steel shell, which is coated over the walking area to prevent slipping. Holes corresponding to the flow of force were cut into the shell using TRUMPF laser machines. The size and density of the apertures depend on the degree of utilisation of the supporting structure. In the area of the walking surface, the lasered apertures are replaced by approximately 14 300 smaller holes filled with glass plugs (Figure 17.13).

The bridge was welded together on-site from six individual parts and lifted into its final position using a heavy-duty crane. The lightness of the bridge is underlined by the very transparent and antireflective all-glass railings.

Geometrically non-linear finite-element analysis, considering deviations of up to 100 mm from the desired geometry, showed that the shell construction is very stable; owing to the double

curvature and high eigenfrequencies, pedestrians should not experience any vibration.

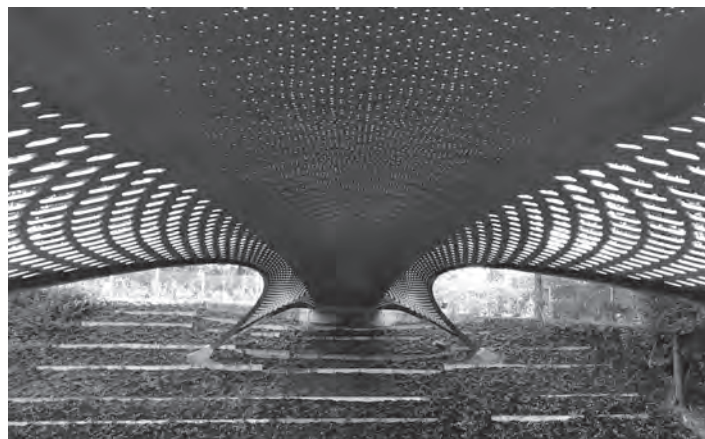


Figure 17.13 View from underneath, showing the holes (© Schlaich Bergermann Partner)

Taplow Riverside Footbridge, Taplow, UK Knight Architects/Cowi, 2018

This 35 m long elegant pedestrian–cycle bridge (Figure 17.14), crossing the River Thames at a location of natural beauty with access constraints, is an example of excellent collaboration between architect and engineer. Its slender and shallow sculptural form has made the bridge a popular local landmark. Access for construction was extremely limited; this required buildability to be considered from the outset in the design and installation of the bridge.



Figure 17.14 View showing complete bridge (© Lance McNulty)

A steel structure was designed with structural efficiency in mind, while developing a clear architectural identity comprising three key features: the arch, the deck and the flat plate hangers. The arches support a slender composite steel–concrete deck formed by a steel tray comprising the edge beams and bottom plate, which was filled with in situ concrete after the bridge was installed. Transverse stiffeners are revealed below (Figure 17.15) and extend outwards to form the hangers. The flat plate hangers, with pinned joints at their ends, provide lateral stability to the arch, achieving greater slenderness. This composite construction improves structural behaviour, particularly in relation to the dynamic response and acoustics, as well as facilitating construction (Figure 17.16).



Figure 17.15 Underside of the deck, showing transverse stiffeners (© Lance McNulty)



Figure 17.16 View showing key features of the bridge (© Knight Architects)

The parapets comprise closely spaced thin steel uprights welded to the deck supporting timber handrails. The dark grey of the parapet and the deck create a contrast with the white arch, accentuating the visual slenderness of the bridge.

The bridge was fabricated off-site and had a trial assembly to ensure a more efficient installation on-site. With the only viable access route being the river, the structure was fabricated in three parts and transported by road to an assembly yard downstream of the bridge location, where it was assembled. The entire structure was then floated upriver on a pontoon (Figure 17.17) and installed in a single operation.



Figure 17.17 Entire bridge transported for installation (© Knight Architects)

London Wall Place development, London, WSP/Make, 2018

As part of the London Wall Place development, a number of footbridges and walkways were required to replace some of the heavy 1960s high walks and provide stylish walkways and footbridges. The development provides two landmark commercial buildings and an acre of landscaped public realm. This open space creates a unique destination for the area through a series of new gardens set among historic landmarks, including the Roman City Wall and the ruins of St Alphage's church tower.

The six new footbridges and walkways, including a cable-stayed footbridge, a partially suspended walkway and a horizontally curved footbridge (Figure 17.18), provide the necessary functionality, while enhancing the area and blending within the landscaped area.



Figure 17.18 Curved footbridge, showing how the structure blends within its environment (© Brookfield Properties)

Several constraints and limitations made the design and construction more challenging. For example, the cable bridge could only be supported at the new London Wall Place building structure and not at the existing building across the road. This led to the design of a slim steel box deck supported by cables, which improved the headroom beneath compared with the original concrete bridge (Figure 17.19).



Figure 17.19a Cable-stayed bridge supported from the building frame (© WSP)

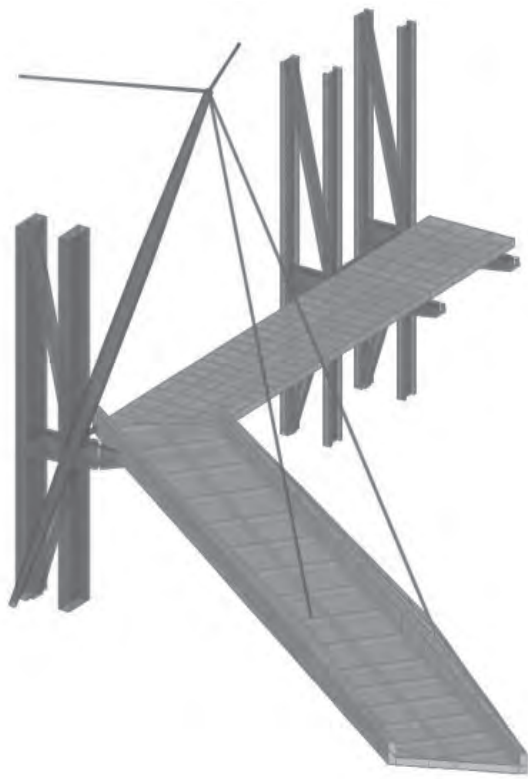


Figure 17.19b Model used for analysis of the cable-stayed bridge (© WSP)

Other restrictions included the presence of a basement area under the road immediately below the cable bridge, which had to be considered in the positioning and use of a crane to lift this bridge into its position.

Plasht School Footbridge, London, Birds Portchmouth/Russum/Techniker, Ltd, 2000

Former grammar and secondary modern schools are linked to form one comprehensive school by a footbridge over a busy urban road (Figure 17.20). The design recreates a school cloister



Figure 17.20 Covered steel bridge of sculpted plate and rolled sections (photograph courtesy of Techniker).

and curves on plan to avoid mature trees. The main girders are standard rolled steel sections, cold-rolled to curve horizontally and heat-bent to a vertical profile. A drainpipe set on the top flange makes up the required balustrade depth. The supporting piers are built up from steel plate and split tubular sections. The asymmetrical forms (Figure 17.21), with unstiffened edges, required a careful check for local buckling instabilities. The Teflon-coated glass-fibre membrane canopies are tensioned at regular intervals by expanding rings of punched steel plate.



Figure 17.21 Variety of fabrication techniques, machining and simple blacksmithing used to form distinctive components (photograph courtesy of Techniker)

Selfridges Footbridge, Birmingham, UK, Future Systems/Engineer, Arup, 2003

This is a 37 m span footbridge linking the Selfridges building in Birmingham to an adjacent car park. The bridge deck is 21 m above pavement level and constructed as a steel box girder, elegantly curved both on plan and in elevation (Figure 17.22). Further support is given by cable stays, which are tied back



Figure 17.22 Sky bridge connecting Selfridges to multi-storey car park (photograph courtesy of Arup)

to the Selfridges building frame at roof level (Figure 17.23). Three-dimensional modelling was used to demonstrate that the deck could be fabricated entirely from warped but developable steel plates and segments of singularly bent tubes. The bridge is covered by a polycarbonate canopy, supported from a series of T-section steel arches at varying angles of inclination and restrained laterally out of plane by connection to a continuous handrail.

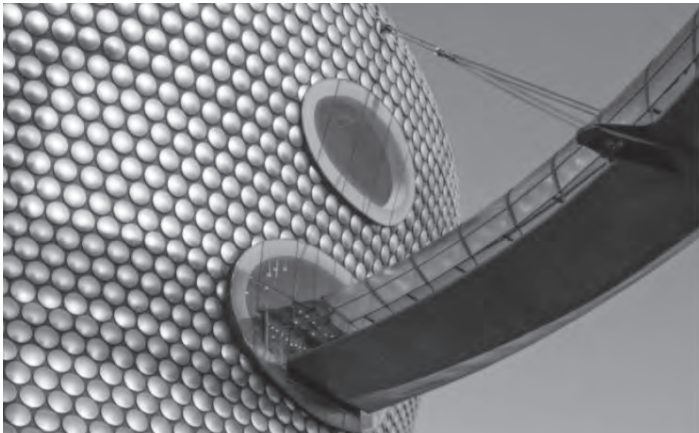


Figure 17.23 Cable supporting the sky bridge from the building (photograph courtesy of Arup)

The Peace Bridge, Derry, Wilkinson Eyre/AECOM, 2011

The Peace Bridge is a foot–cycle bridge across the River Foyle in Derry, Northern Ireland. The asymmetrical bridge, which is a self-anchored suspension bridge with a total length of 235 m, a longest span of 101 m and a width of 4 m, is supported by two sloping pylons (Figures 17.24 and 17.25) and symbolises a coming together of two communities. Its curved footpath and cycleway link the Guildhall, in the centre of the city, with Ebrington Square in the Waterside area and St Columb's Park.

The bridge, which is intended to improve access between the east and west banks as part of a wider regeneration plans, also provides a crossing over a railway line approaching Waterside station.

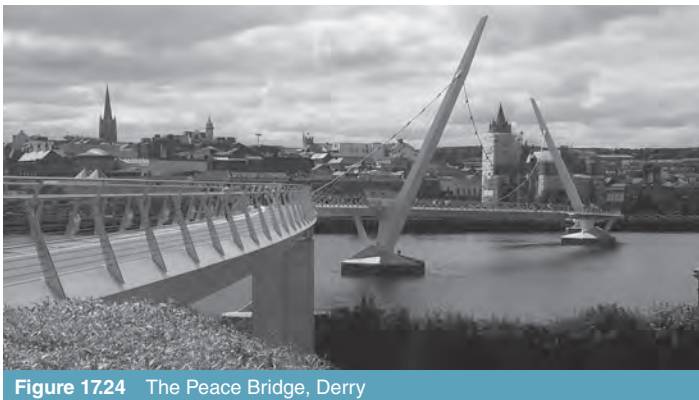


Figure 17.24 The Peace Bridge, Derry



Figure 17.25 The two sloping pylons supporting the deck

Merchant Square Lifting Footbridge, London Knight Architects/AKT II, 2014

This 3 m wide cantilevered structure spans the Grand Union Canal and is raised using hydraulic jacks in a manner reminiscent of a traditional Japanese hand fan (Figure 17.26). Combining and optimising geometric, structural, mechanical and electrical parameters, an elegant and efficient solution has been achieved.



Figure 17.26 Bridge in its closed position, allowing pedestrian access (© Peter Cook)

The bridge comprises five fabricated beams opening in sequence, with the first rising to 70° and the last resulting in the required clearance of 2.5 m high by 5.5 m wide (Figure 17.27). The hydraulic mechanism is assisted with shaped counterweights, reducing the amount of energy required to lift the beams. The bridge balustrades are formed from twin rows of inclined stainless steel rods, with the handrails housing a continuous low-energy LED downlight that illuminates the walking surface and the edges.



Figure 17.27 Bridge during opening (© Edmund Sumner)

Abbey Wood Station and Footbridge, Fereday Pollard/WSP, 2020

This is an example of footbridge combined with lift, providing access for all across the platforms within a newly designed and constructed railway station (Figure 17.28).

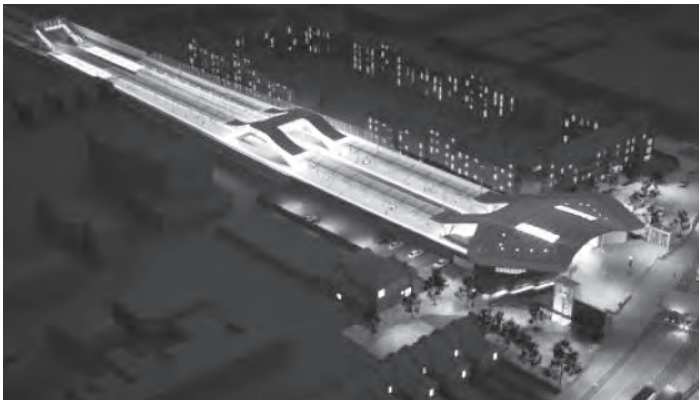


Figure 17.28 Abbey Wood Station (© Fereday Pollard)

The modern style footbridge with glass enclosures provides safe and secure access for passengers (Figure 17.29).



Figure 17.29 Abbey Wood Station, Fereday Pollard/WSP

Lille Langebro, Copenhagen Wilkinson Eyre/Buro Happold, 2019

This 160m span horizontal swing bridge, crossing the inner harbour, follows an elegant curve in plan and further enhances cyclist and pedestrian access in a city well known for its walking and cycling facilities (Figures 17.30 and 17.31). The structure is arranged as a triangular wing on either side of the bridge that dips below the deck at the abutments and gradually soars up at midspan in a sinuous curve before falling again.



Figure 17.30 Lille Langebro, Copenhagen (© Rasmus Hjortshøj-Coast)



Figure 17.31 Lille Langebro, Copenhagen (© Rasmus Hjortshøj-Coast)

Future considerations

The ten examples used show the variety of footbridges designed and constructed in recent times. The potential of using different forms and materials continues to grow and provide further opportunities for the engineer and the architect to explore and develop new ideas.

Advances in analysis and design software continue to enhance the ability to design complex forms, and provide unlimited opportunities. While the general analysis and design of structural elements is very advanced, detailing and interaction between the designer and fabricator remains an area that can be improved. This is often an area that can create challenges during fabrication and construction.

Sustainability continues to be an important factor in the design and construction of footbridges; consideration of the environmental, social and economic aspects of each project must be incorporated into the project activities from the outset. Equally, buildability will be an essential part of the design, and

early consideration of maintainability and decommissioning is required. The use of footbridges as part of public realms or elevated walkways within urban areas continues to rise, providing excellent opportunities for creativity and leading to enhanced infrastructure.

Pedestrian bridge design, as a specific activity, is relevant in the wider relationship of architecture, urban design and engineering, showing that there are now a multitude of solutions for most problems and that the design process is informed by a wide spectrum of concerns, of which engineering is only one discipline. New planning requirements and the influence of pressure groups, design panels, regulatory groups, urban review panels and other stakeholder groups can only enrich the design of individual bridges.

We enter the third decade of the twenty-first century, with a considerable range of improving materials: high-strength concrete, stainless steel, engineered timber products, composites and plastics, leading lightweight bridge design into new areas, where dynamic instability must be fully accounted for. All the tools, digital analysis and materials understanding are available from the aerospace and automotive industries to handle these innovations and statistical accounting for real-life loads will enable safety factors to be refined as a result of better analysis. A generation of more elegant bridges should appear, which could include hybrid structures both in form and material.

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Chapter 18

Modern developments of fibre-reinforced polymers used in bridge engineering

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The first half of this chapter introduces the polymers, fibres and fibre-reinforced polymer (FRP) composites used in bridge engineering, from the point of view of the basic components of composites, the manufacturing techniques of composites and the mechanical and in-service properties of the composite materials. The second half of the chapter concentrates on the areas of bridge engineering where these materials are utilised; their advantages over the more conventional civil engineering materials will be discussed, together with disadvantages and challenges.

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Introduction

The legacy of a lack of investment in research and development in the construction industry for more than 30 years after the end of the Second World War is clearly illustrated by the lack of progress in construction methods and in construction materials. Consequently, no interest was shown by potential material investors in civil engineering and the technological revolution in materials and their processing techniques, utilised in other sectors of the manufacturing industry, bypassed the construction industry. The Latham Report (Latham, 1994), regarded the construction industry as low-technology, low-skilled and labour-intensive compared with most other industries. It related the mismatch between research investment and construction expenditure to an inadequate understanding of many aspects of civil engineering, such as the deterioration mechanism of structures, and this has resulted in a lack of due allowance being made for practical repair and maintenance. However, notwithstanding this report, over the past 20 years there is evidence of a transition, by the construction industry, from conventional civil engineering materials to more advanced materials. However, there is still much room for improvement; the Reinventing Construction report (McKinsey, 2017) described construction as having poor productivity in comparison with all other industries, partly owing to a lack of standardisation and modularisation; this is the case for a significant amount of bridge construction.

One of the promising advanced materials to enter the civil engineering industry some 40 years ago was fibre-reinforced polymer (FRP). The developments associated with FRP

materials, during this period, have been considerable and the requirements that have initiated this state have revolutionised the manufacturing and fabrication techniques for the production of FRP materials. The aerospace, defence and renewable energy industries have been utilising FRP materials for some considerable time and the manufacturing techniques that are currently used in construction have been developed from those industries. It will be demonstrated in this chapter that one area of the civil engineering industry that is realising great benefits from the use of FRPs is that of bridge engineering. Fibre-reinforced polymer materials have started to overcome the construction industry productivity challenge by enabling off-site fabrication of large bridge components.

Before discussing the subject of FRP materials associated with bridge engineering in detail, it is essential for the reader to have a clear understanding of the meaning of that material. A useful wide-ranging definition for FRP materials is provided in Holloway and Head (2001).

Composite materials consist normally of two discrete phases, a continuous matrix which is often a resin, surrounding a fibrous reinforcing structure. The reinforcement has high strength and stiffness whilst the matrix binds the fibres together, allowing stress to be transferred from one fibre to other producing consolidated structures. In advanced or high performance composites, high strength and stiffness fibres are used in relatively high volume fractions whilst the orientation of the fibres is controlled to enable high mechanical stresses to be carried safely. In the anisotropic nature of these materials lies their major advantage. The reinforcement can be tailored and oriented to follow the stress patterns in the component leading to much greater design economy than can be achieved with traditional isotropic materials.

The reinforcements are typically glass, carbon or aramid fibres in the form of continuous filament, tow or woven fabrics. The resins which confer distinctive properties such as heat or chemical resistance may be chosen from a wide spectrum of thermosetting or thermoplastic synthetic materials, but those commonly used are the thermoset resins, the epoxy, the phenolic and sometimes the polyester. More advanced heat resisting types such as bismaleimides are gaining usages in high performance applications and advanced carbon fibre/thermoplastic composites are well into a market development phase.

It will be seen in this chapter that the main fibres used in bridge engineering are the carbon and glass fibres. These names are generic fibre names and a number of grades and formulations are available within each fibre category. This will become clear in a later section, where the various fibres and their forms are discussed. The manufacturing techniques for FRPs, which affect their physical and mechanical properties, are described in a further section.

The term *FRP* encompasses a wide range of fibre–matrix materials, each with its own unique characteristics. It is not the intention in this chapter to discuss the whole family of FRPs; reference should be made to Hollaway (1993), Hollaway and Head (2001) and Matthews and Rawlings (2000) for further information on this topic. In this chapter, the relevant FRPs and adhesive materials used in bridges and their specific manufacturing techniques will be discussed, thus enabling the most efficient and cost-effective solution to the design problem to be made. The chapter will then continue by describing the utilisation of the material in conjunction with the more conventional civil engineering materials used in bridge structures and as ‘all-FRP’ bridge constructions. There are UK design guides and specifications, European design guides and the ACI, Canadian and Japanese design codes for FRPs; these are given in an appendix.

The second half of the chapter will focus on the utilisation of FRP composites in bridge engineering under 11 topics:

- new bridge structures, fabricated primarily from FRP composite material
- FRP–concrete beam construction
- bridge decks, manufactured from FRP composite material
- steel-free bridge decks
- bridge enclosures
- rehabilitation of existing bridges
- seismic retrofit
- FRP confining of concrete columns
- internal reinforcement to concrete members
- intelligent structures.

Reinforcement mechanism of fibre-reinforced polymer composites

Introduction

Fibre-reinforced composite materials are made by the controlled distribution of two materials; these are (a) the continuous matrix phase and (b) the fibre reinforcement phase. In addition, there is the boundary between the matrix and the reinforcement (the interface area), which also controls the properties of the given pair of materials.

The continuous matrix phase (the polymer)

The matrix phase (the polymer) is an organic material composed of molecules that are repeats of similar but simpler units called the monomer. The requirements of the matrix phase are (a) to bind and to maintain the fibres in position, (b) to protect the surfaces of the fibres from external influences, environmental degradation and abrasion and (c) to transfer stresses to the fibres by adhesion or friction, or both; the adhesion to the fibres must be coupled with adequate matrix shear strength. In addition, the matrix must maintain chemical and thermal compatibility with the fibre over the lifespan of the composite. Furthermore, during the manufacturing process, complete wet-out of the fibres by the matrix must be achieved.

The three major types of matrix (polymer) that are used in bridge engineering are the thermosetting, the thermoplastic and the elastomeric; each requires a different procedure for its manufacture. Their mechanical and in-service properties will be different; these are discussed later.

It is noted that the matrix polymer material requires two components, the resin and the curing agent (hardener). In bridge engineering, there are two types of curing procedure; one uses a cold-cure resin and the other a hot-cure resin. The former would be used in situ in the field to form either the composite or the adhesive, owing to difficulties in environmental control and heating; the latter is typically used in a factory environment. The cure temperature of the site would determine the length of the polymerisation period. Cold-cure resins are often developed to avoid the need for significant heating or further post-curing at higher temperatures, and are typically cured at temperatures of approximately 20°C or lower. The hot-cured resin products (typically cured at approximately 150°C) would be manufactured by automated techniques, such as pultrusion or fibre pre-impregnation, in a factory under controlled conditions of temperature and pressure; the manufacturing and cure

procedures are discussed later. Resins that cure at intermediate temperatures have also been developed to allow a wide range of manufacturing methods to be used.

Thermoplastic polymers

The thermoplastic polymers are composed of long chain molecules, which are held together by relatively weak van der Waals forces, but the chemical valency bond along the chains is extremely strong; they are generally of a semi-crystalline form. Their strength and stiffness will be derived from the very high molecular weight and from the inherent properties of the monomer units.

The polyolefins and polyesters are the main thermoplastic polymers utilised in bridge engineering for geotechnical applications. These materials are formed into geotextiles, geolinear elements and geomembranes; geosynthetics are not discussed in this chapter but references can be found in Fannin (2004), Hollaway and Head (2001) and Ingold and Miller (1988). If the thermoplastic polymer polyacrylonitrile (PAN) is placed under a tensile load, the material behaves viscoelastically, with a gradually decreasing slope of the stress–strain relationship. At a limiting load value, the material will be unable to support further load and will continue to elongate excessively; the slope of the stress–strain relationship will become zero. During this elongation the molecules will align themselves with the direction of the applied load – the greater the load, the greater the elongation – and in so doing form a fibre. This fibre is the precursor for the manufacture of many types of carbon fibre.

Thermosetting polymers

The thermosetting polymers consist of long chain molecules, which are cross-linked in a curing reaction, producing covalent bonds between them. This chemical reaction is known as polycondensation, polymerisation or curing. This network and the length and density of the molecular units are a function of the chemicals used in the manufacture of the polymers, and the cross-linking is a function of the degree of cure of the polymer. Both the network and the cross-linking will have an influence on the mechanical and in-service properties of the material. Furthermore, the degree of cure is a function of the temperature and the length of the polymerisation period. The main thermosetting polymers used for structural components in bridge engineering are the epoxies, the vinyl esters and the unsaturated polyesters.

Elastomers

A third member of the polymer family of interest to the bridge engineer is the elastomer, which is of similar form to natural rubber; the elastomeric material can be either reinforced or unreinforced. The material is composed of long chain molecules, which are coiled and twisted in a random manner, similar in form to that of the thermoplastic polymer; owing to the molecular flexibility of the material, it is able to undergo

very large deformations. The molecules are cross-linked by a curing process known as vulcanisation; it is a similar procedure to the curing of the thermosetting polymer. The main use of this polymer in bridge engineering is for elastomeric bridge bearings, which are discussed in more detail in other chapters in this book.

Biological sources for the manufacture of thermoplastic and thermosetting resins (known as bio-resins) are now also used, as well as petrochemical sources, and blends are also used (Netcomposites, 2014). The use of bio-resins is expected to increase in the future, owing to the environmental and sustainability benefits.

Thermosetting polymers

Thermosetting polymers are the most common polymer type used for structural FRP components in bridge construction, and include epoxy, vinyl ester and polyester resins.

Epoxies

The large family of epoxy resins represents some of the highest performance resins of those available at this time. Epoxies generally outperform most other resin types in terms of mechanical properties and resistance to environmental degradation; this has led to their almost exclusive use in aircraft components and a sizeable use in the construction industry. The most important epoxy resins for the bridge engineer are the low molecular weight polymers (oligomers), which are produced from the reaction of bisphenol A and epichlorohydrin. They range from the medium-viscosity liquids through to high melting point solids. Epoxies used in bridge engineering have the following properties.

- They can be used for different composite manufacturing techniques, where polymerisation can take place at room temperature (cold-cure resins) or elevated temperatures (hot-cure resins); thus, two different resin and curing systems (i.e. the cold- or the hot-cure systems) will be required for these manufacturing methods. Consideration must be given to the *glass transition temperature* (T_g) of cold-cure epoxy polymers (see later).
- They have high specific strengths and dimensional stability.
- They have higher operating temperatures than the polyesters, owing to their superior toughness.
- They exhibit low shrinkage during polymerisation and good adhesion to many substrates. This allows high-quality mouldings with good dimensional tolerance to be manufactured.
- They use hot-cured polymer systems, which have a high temperature resistance and can be used at temperatures up to 200°C. Some epoxies have a maximum temperature range of above 300°C.

Since the amine molecules ‘co-react’ with the epoxy molecules in a fixed ratio, it is essential that the correct mix ratio is obtained between the epoxy resin and the curing agent, to ensure that a complete reaction does take place. If the amine

and epoxy are not mixed in the correct ratios, unreacted resin or curing agent will remain within the matrix, which will affect the final properties after curing. To assist with the accurate mixing of the resin and curing agent (particularly for on-site fabrication), manufacturers usually formulate the two component parts to give a simple mix ratio that is readily achieved by measuring out the two components by weight or volume. Alternatively, the resin and curing agent are provided, by the manufacturers, in exact amounts in two separate containers; the resin container would be of sufficient volume to pour the curing agent into it and to mix by mechanical paddle.

Vinyl esters

The vinyl ester is a hybrid form of polyester resin that has been toughened with epoxy molecules within the main molecular structure. Its molecule features fewer ester groups than the polyester resin; as these ester groups are susceptible to water degradation by hydrolysis, vinyl esters exhibit better resistance to water and many other chemicals than do their polyester counterparts. Furthermore, as vinyl esters have unsaturated esters of epoxy resins, they have similar mechanical and in-service properties to those of the epoxies, but they have similar processing techniques to those of the polyesters. Generally, the vinyl esters have

- a processing method at both room (cold-cure resins) and elevated (hot-cure resins) temperatures; the same consideration, regarding the glass transition temperature (T_g), is given to the use of this polymer as that stated for the epoxy polymer
- good wetting characteristics and bond well to glass fibres
- good resistance to strong acids and strong alkalis
- reduced water absorption and shrinkage properties, as well as enhanced chemical resistance, compared with those of the polyester.

Unsaturated polyesters

Polyesters are sometimes used for the manufacture of bridge structural components although they generally have lower resistance to chemical degradation than vinyl ester or epoxy resins. For instance, they are used in the production of automated pultrusion components. They are also readily processed and cured at ambient temperatures using a wet lay-up procedure; these ambient-cured polymers may be post-cured. Styrene, in amounts of up to 50%, is the most commonly used diluent. However, it should be noted that, during the wet lay-up (hand-laminated) production method, large amounts of styrene and trace quantities of styrene-7,8-oxide (SO) are released, which will cause a health risk unless safety precautions are taken. There are three types of commonly used polyester resin; these are the orthophthalic, isophthalic and bisphenol A resins. The first resin mentioned has low thermal stability and chemical resistance, and is generally not suitable for bridge components, which have very long design life requirements. The isophthalic

resins, which contain isophthalic acids as an essential ingredient, are of superior quality, with better thermal and chemical properties. The bisphenol A resin is of superior quality to both orthophthalic and isophthalic resins, with a higher degree of chemical, thermal and hardness resistance and has some degree of flame resistance. The shrinkage at cure of the polyester resin tends to be high, particularly if styrene in monomeric form is used as a reactive diluent in the resin. This will cause (a) an increase in hydrophobicity, thereby effectively decreasing the level of moisture absorption, and (b) an increase in shrinkage to levels of 5%–19% by volume. This can result in significant microcracking in resin-rich areas and large residual stresses in composites with high fibre volume fractions.

The polymerisation of thermosetting polymers must be completed before the polymer (or the FRP composite) is used, as the long-term performance properties of the composite will be influenced by the degree of cure of the resin, particularly for ambient (cold) cured systems. The various methods for the manufacture of composites are discussed later in the next chapter, and it will be seen that materials that are manually fabricated in the field, together with adhesive polymers (if these are employed during fabrication), use cold-cure systems at ambient temperatures. Composites, which are manufactured under factory conditions, use hot-curing polymers and are cured at the operating temperature of the manufacturing process; the production temperature values of these processes are of the order of 120°C to 135°C; the actual value will depend on the system being used.

Ideally, all cold-cure adhesive polymers and polymer composites *should be post-cured* at an elevated temperature for a certain length of time; the higher the temperature, the shorter the post-cure period. The initial cure of these polymers will reach only about 90% of the full polymerisation value after some 10 days (depending on the polymer system and ambient cure temperature). Cain *et al.* (2006) investigated the post-cure effects on E-glass/vinyl ester fibre-reinforced polymer composites manufactured using the vacuum-assisted resin transfer moulding (VARTM) method. The composites were differentiated by varying levels of post-cure temperature and duration, and examined for the effects of advancing cure at various time points after the post-cure period. In parallel, the matrix polymer was inspected using Fourier transform infrared spectroscopy to assess the degree of conversion directly. The results of this investigation suggested that the degree of conversion is limited to 80% for the vinyl ester oligomer and 90%–95% for styrene following a post-cure of 93°C. It was observed that after 300 days of ambient temperature storage, the non-post-cured samples approached the degree of conversion exhibited by those post-cured at 93°C. Cain *et al.* (2006) concluded that resin-dominated quasistatic properties are greatly affected by the degree of cure, whereas fibre dominated properties are not affected so much. As mentioned, this under-cure will affect the long-term mechanical property of the polymer, with the result that its exposure to the variety of adverse and sometimes harsh environmental conditions met in construction will degrade the polymer and the FRP composite

material, and thus alter their mechanical performance. A further concern regarding the ambient-cured systems is that their relatively low cure temperature will affect their glass transition temperature (T_g), which will generally be about 15°C to 20°C above the cure temperature. This point is discussed in the section on dimensional stability.

Mechanical and in-service properties of thermosetting polymers

Mechanical properties

The important mechanical properties of the three polymers used in bridge engineering are given in Table 18.1. Although these mechanical properties are important to the bridge engineer, it should be remembered that the polymer composite derives its mechanical characteristics primarily from those of the fibre. The most important properties of the polymer are its in-service properties, given next.

Material	Density: kg/m ³	Ultimate tensile strength: MPa	Modulus of elasticity in tension: GPa	Coefficient of linear expansion: 10 ⁻⁶ /°C
Polyester	1200–1500	45–90	2.5–4.0	100–110
Vinyl ester	1100–1400	90	4.0	80
Epoxy	1100–1400	90–110	3.5	45–65

Table 18.1 Typical mechanical properties of the three thermosetting polymers used in bridge engineering

In-service properties

The most important in-service properties of polymers are listed.

The stiffness of thermosetting polymers

As stated earlier the stiffness is influenced by the degree of cross-linking of the polymer. The process of cross-linking produces a tightly bound three-dimensional network of polymer chains. This network, the chain length and the density of the molecular units, are a function of the chemicals used in the manufacture of the polymer, and the cross-linking is a function of the degree of cure of the polymer.

The strength of thermosetting polymers

The short-term strength of a thermosetting polymer depends on the bonding, length, density and degree of cross-linking of the molecular structure of the polymer. It will also depend on the type of loading applied to that polymer. The long-term strength of the polymer will depend on its durability in the environment in which it is placed. The durability of composites, which depends largely on the durability of the polymer, is discussed later.

Thermal conductivity of all polymers

The thermal conductivity of polymers is low; consequently, they are good heat insulators. The thermal conductivity of a polymer can be changed by incorporating metallic fillers at the time of manufacture of the polymer.

Coefficient of thermal expansion of thermosetting polymers

This characteristic is extremely important in design when considering joining FRP composites to a dissimilar material. The coefficient of thermal expansion of a polymer is an order higher than that of the more conventional civil engineering materials. In bridge engineering practice, this situation only occurs when an adhesive material is used to join two dissimilar materials, for example, when FRP composite material is bonded to other FRP, reinforced concrete, timber or steel bridge structural members to strengthen/stiffen them. This will be discussed later in the chapter.

Toughness of a thermosetting polymer

The toughness of a brittle thermosetting polymer can be improved by blend filling or copolymerising it with a tough but lower-stiffness one. However, an increase in the toughness of the polymer will tend to decrease its stiffness; consequently, a compromise between toughness and stiffness of a thermosetting polymer must be made.

Permeability

Polymers with high crystallinity or density or a high degree of cross-linking will generally have low permeability; thus, gases and other small particles will not readily permeate through it. Hackman and Hollaway (2006), Haque and Shamsuzzoha (2003) and Liu *et al.* (2005a) have shown that moisture will permeate polymers over time, particularly if the polymer (composite) is permanently immersed in water or salt solution, or is exposed to de-icing salt solutions.

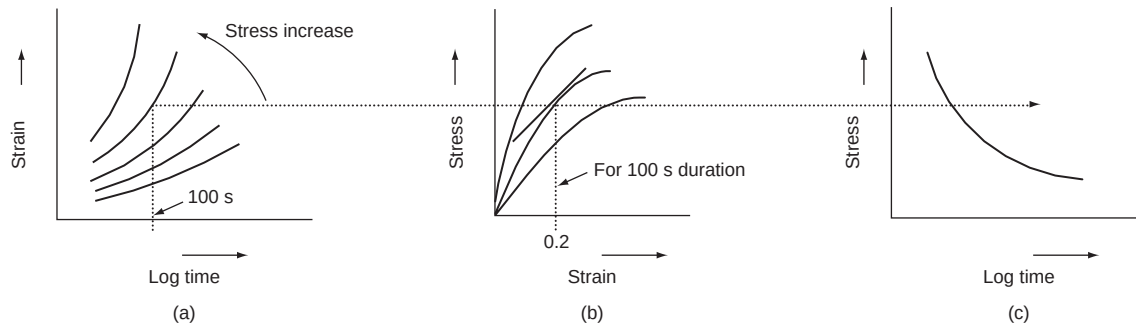
Chemical resistance

The chemical resistance of a thermosetting polymer depends on the chemical composition and bonding in the monomer; generally, the resistance is good.

Creep characteristics of polymers

Polymers are classified as viscoelastic materials: they have both elastic solid and viscous fluid characteristics. In assessing the creep of a polymer material, it is important to know

- the temperature and moisture environments into which it is placed
- the loading histories and the nature of the applied load



Note: For fibre-matrix composite materials

As the fibre-matrix composite contains a viscoelastic polymer, this material will creep; however, the fibres do not creep and therefore stabilise the polymer, causing the composite to creep less than that of the pure polymer. Similar forms of creep curves to those above for polymers will be formed for composites.

Figure 18.1 Creep curves – isochronous stress–strain curves (adapted from BS 4618 1970): (a) isostress creep curve for polymers (experimental or field values); (b) 100 s isochronous stress–strain curve for polymers (values derived from (a)); (c) isostrain creep curve for polymers (values derived from (a))

- the time-dependent nature of the microdamage in the composite material subjected to stress.

To minimise creep, it is necessary to ensure that the service temperatures do not approach the glass transition temperature of the polymer. Figure 18.1 shows a family of creep curves, consisting of isostress creep curves (Figure 18.1(a)), 100 s isochronous stress–strain creep curves (Figure 18.1(b)) and the isostrain creep curve (Figure 18.1(c)). These latter curves have been produced by cross-plotting from the isostress creep curves at constant times.

The glass transition temperature of polymers

The glass transition temperature (T_g) of an *amorphous* material (the polymer) is the temperature at which it changes between a brittle or vitreous state and a plastic state; this change takes place over a temperature range, the midpoint of which is taken as T_g , as shown in Figure 18.2. The value of T_g depends on the detailed chemical structure of the polymer, which, in turn, depends on the cross-linking of the molecules for strength. Below T_g , all polymers are rigid and frequently brittle; they therefore have both stiffness and strength. Above T_g , the *amorphous* polymers are soft elastomers or viscous liquids; consequently, they have essentially no stiffness or strength. The epoxies, vinyl esters and polyesters used in construction would generally be in the *amorphous* state. The numerical value of T_g may be quoted with slightly different values, depending on the testing technique used; these methods are dynamic mechanical analysis (DMA) and differential scanning calorimetry (DSC). The drastic changes that any polymer will experience at T_g are shown in Figure 18.2 as typical mechanical or physical properties.

From this discussion, it will be clear that polymer composite structural members should not be exposed to temperatures above the value of T_g of the matrix material. Hot-cured thermosetting polymers would normally have a higher value of T_g than the room temperature cured ('cold-cure') thermo-

setting materials; their actual values would generally be about 15°C–20°C above the curing temperature. The value of T_g of *some* low-temperature (ambient-cured) moulded composites, can be increased by further post-curing the polymer at a higher temperature than that of the original cure, but there is a maximum value of T_g of all polymers, irrespective of the post-cure temperature.

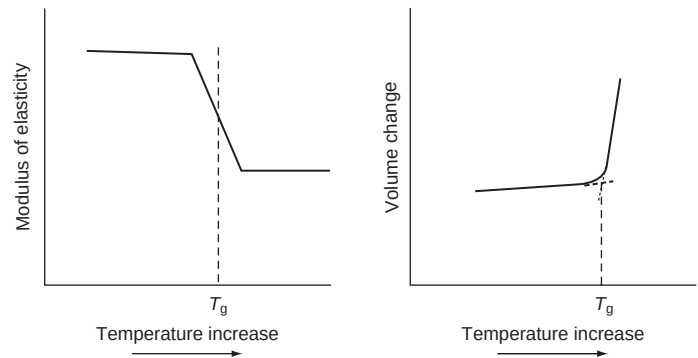


Figure 18.2 Change in properties at the glass transition temperature of thermosetting polymers: (a) modulus of elasticity (as a function of temperature); (b) hardness (volume as a function of temperature)

Dimensional stability

The thermosetting polymers used in bridge engineering are semi-crystalline, semi-rigid and frequently brittle in nature and will lose their dimensional stability above the glass transition temperature (T_g); therefore, the strength and stiffness will be much reduced. If polymers with a high crystallinity are used, they will have a region of acceptable dimensional stability above T_g . The post-curing procedures of thermosetting polymers are vitally important, particularly the ambient-cured ones, as in this case the polymerisation will only be about 90% complete after about 7 to 10 days, depending on the ambient temperature during the initial cure.

The fibres

The fibres that are mainly used in bridge engineering for structural applications are the carbon and glass fibres and for geotechnical applications those made from one of the polyolefin family of thermoplastic polymers (Aboutaha *et al.*, 2002; Hollaway and Head, 2001; Liu, 2003). Other fibres such as aramid and basalt fibres are also available commercially. Research has been undertaken on the use of bio-fibres in bridge construction, with a small number of bridge applications (Mathijssen, 2020) in the Netherlands using flax fibres. Although less is known regarding the long-term durability of bio-fibres (which absorb moisture more readily than synthetic fibres), there are clear sustainability benefits in the use of such materials and further research and application in bridges is expected.

Glass fibres

Glass is the common name given to a number of mutually soluble oxides that can be supercooled without crystallising; they are all clear amorphous solids when cooled. Several grades of glass fibre are produced commercially, of which E-glass fibre is the most important one used in bridge engineering. Other types of glass fibre are also used.

- *E-glass fibre* has a low alkali content, of the order of 2%. It is used for general-purpose structural applications and is the major fibre used in the construction industry; it also has good heat and electrical resistance.
- *S-glass fibre* is a stronger (typically 40% stronger at room temperature) and stiffer fibre with a greater corrosion resistance than the E-glass fibre. It has good heat resistance. The S-2-glass fibre has the same glass composition as S-glass but differs in its coating. The S-2-glass fibre has good resistance to acids, such as hydrochloric, nitric and sulphuric acids.
- *E-CR-glass fibre* has good resistance to acids and bases and has chemical stability in chemically corrosive environments.
- *R-glass fibre* has a higher tensile strength and tensile modulus and greater resistance to fatigue, ageing and temperature corrosion than E-glass.
- *Cem-FIL* (or *AR-glass*) *fibre* is specifically used for resisting the alkali in Portland cement and is used as the reinforcement for glass-fibre-reinforced concrete (GRFC).
- *Advantex fibre* is a proprietary fibre that exhibits similar mechanical properties to E-glass fibre and chemical resistance properties of E-CR-glass fibre, but with less environmental impact during manufacture.

Table 18.2 shows the chemical composition of E-glass and S-glass.

Oxides used	CaO	Al ₂ O ₃	MgO	B ₂ O ₃	SiO ₂
E-glass	17.5	14.0	4.5	10	54
S-glass	–	25	10	–	65

Table 18.2 Chemical composition of E-glass and S-glass (after Phillips (1989))

The commercial manufacturing technique for glass fibres is undertaken by drawing swiftly and continuously fine filaments from the molten glass, which is at a temperature of 1200–1400°C. These series of filaments have exceptionally high specific stiffness and strength and range in diameter from 3 to 24 µm. During this production stage, a surface treatment or sizing is applied before the fibres are gathered into strands, each consisting of 200 individual filaments, and wound onto a drum. The manufacturing technique has been discussed in Hollaway and Head (2001). The mechanical properties of the E and S-2 types of glass fibre are given in Table 18.3.

Material	Fibre	Elastic modulus: GPa	Tensile strength: MPa	Ultimate strain: %
Glass fibre	E	72.4	2400	3.5
	S-2	88.0	4600	5.7
Carbon fibre				
<i>PAN based fibre</i>				
Hysol Grafil Apollo	IM ^a	300	5200	1.73
	HM ^b	450	3500	0.77
	HS ^c	260	5020	2.00
BASF Celion	G-40-700	300	4960	1.66
	Gy 80	572	1860	0.33
Torayca	T300	234	3530	1.51
<i>Pitch based fibre</i>				
Union carbide	P120	827	2200	0.27
	P100	724	2200	0.31
	P75-S	520	2100	0.40
	P55-S	380	1900	0.50
	P55-W	160	1400	0.90
Aramid fibre	49	125	2760	2.4
	29	83	2750	4.0
Basalt fibre	Basaltex	89	3100	3.4

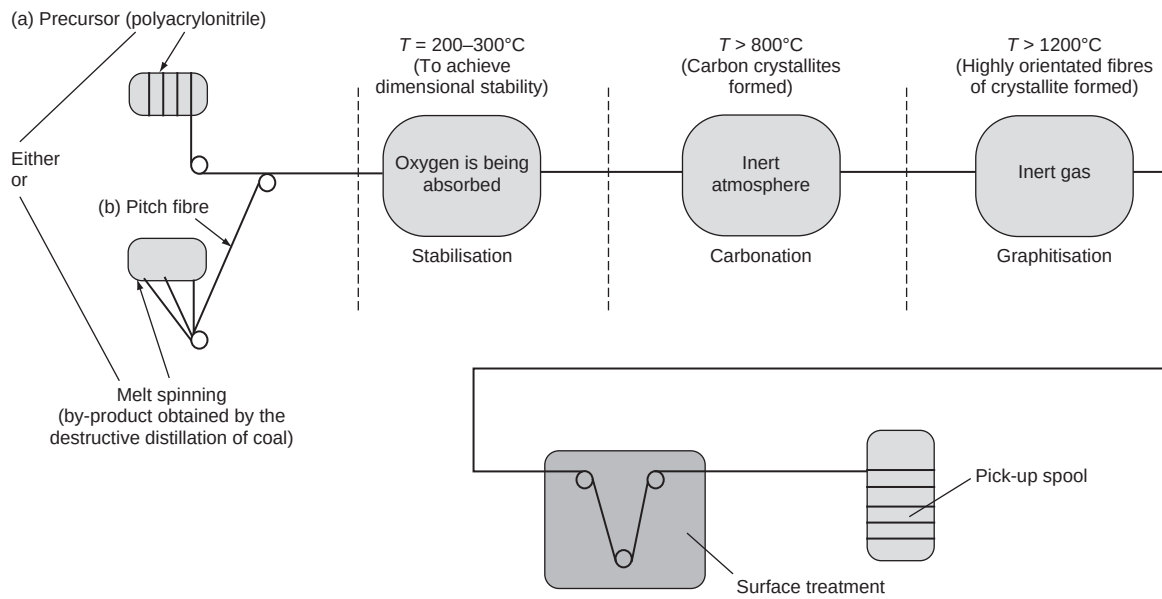
^a High modulus (intermediate modulus)

^b Ultrahigh modulus (high modulus) ^c High strain

Table 18.3 Typical mechanical properties of glass, carbon, basalt and aramid fibres (adapted from Hollaway and Head (2001))

Carbon fibres

The two high performance carbon fibres used in Europe, North America and Japan for bridge engineering are defined as high



Precursors to the carbon fibre used in civil engineering.

(a) Polyacrylonitrile (PAN) fibre (used in the manufacture of high-modulus carbon fibres)

(b) Pitch fibre produced from the distillation of coal (used for the manufacture of ultra-high-modulus carbon fibres)

Figure 18.3 Manufacture of carbon fibre (adapted from Hollaway and Head (2001))

modulus (HM) and ultrahigh modulus (UHM) carbon fibres (in North America, Japan and some Asian countries, these two definitions are referred to as intermediate modulus and high modulus carbon fibres, respectively). The basic manufacturing techniques for the HM and the UHM fibres are the same but the heat treatment temperature will be greater, the higher the modulus of the fibres; thus, a more highly orientated fibre of crystallites will be formed for the UHM fibre. Typical mechanical properties of carbon fibres are given in Table 18.3. Hollaway and Head (2001) state that carbon fibres are manufactured by controlled pyrolysis and crystallisation of certain organic precursors; the manufacturing process consists of sequences of procedures that include (a) stabilisation, (b) carbonisation, (c) graphitisation and, finally, (d) surface treatment; the manufacturing technique of the carbon fibre is illustrated in Figure 18.3.

The precursor fibres that are used for the production of carbon fibres, are (a) polyacrylonitrile (PAN) and (b) pitch fibres. The first fibre precursor is manufactured by spinning to produce a round cross-section fibre; the yield is only 50% of the original precursor fibre. The PAN precursor carbon fibres can also be manufactured by melt-assisted extrusion as part of the spinning operation. Both I-type and X-type rectangular cross-section carbon fibre composites are produced, with a closer fibre packing in the composite. The aerospace industry uses only these types of carbon fibre and they are also mainly used in bridge engineering. The pitch precursor fibres are derived from petroleum, asphalt, coal tar and polyvinyl chloride (PVC); the carbon yield is high but the uniformity of the fibre cross-sections is not constant from batch to batch.

This non-uniformity is acceptable to the construction industry and the pitch fibre precursor is invariably used when ultrahigh stiffness carbon fibres are required. As the modulus of elasticity of the pitch fibres is invariably much greater than that of the PAN carbon fibres, the strain to failure will be lower.

Typically the diameters of carbon fibre filaments are 5 and 8 μm and they are combined into *tows* containing 5000–12000 filaments. A common size of untwisted carbon fibre tow is called 12K, which contains 12000 filaments of carbon and typically ranges in stiffness from 230 GPa ('high modulus') to 640 GPa or even higher ('ultrahigh modulus'). The tows can be twisted into yarns and woven into fabrics, similar to glass fibre.

In addition to strong and high stiffness fibres, bridge engineering requires in-service resistance to withstand high temperature and aggressive environmental conditions; carbon fibres, in general, are not affected by moisture, solvents, bases or weak acids. Table 18.3 provides mechanical properties for a number of types of carbon fibre.

More recently, extensive research and development has been undertaken to develop lower cost precursors and manufacturing methods for carbon fibres (Holmes, 2017); cost has been the main barrier to the more widespread use of carbon fibres in civil engineering (and other industries) for a long time. This has led to research into the use of a variety of commonly available raw materials (e.g. low-grade textiles or lignin from paper waste, as described in Souto *et al.* (2018)), and optimised manufacturing methods (e.g. oxidation methods), which have significantly reduced cost while retaining acceptable mechanical properties and reducing environmental impact.

Aramid fibres

Aramid fibres are synthesised from the monomers 1,4-phenyldiamine (*para*-phenylenediamine) and terephthaloyl chloride. The result is a polymeric aromatic amide (aramid) with alternating benzene rings and amide groups; these polymer strands, when produced, are randomly oriented. To manufacture aramid fibres, the polymer strands are dissolved and spun at a temperature of between 50°C and 80°C and are then extruded into a hot cylinder, which is at a temperature of 200°C; this causes the solvent to evaporate, and the resulting fibre is wound onto a bobbin. The fibre then undergoes a stretching and drawing process to increase its strength and stiffness. The resulting commercial fibre has a high strength, which is unaffected by immersion in water, and is heat resistant. Two grades of stiffness are generally available: (a) the first has a modulus of elasticity of around 60 GPa and is used in applications such as cables and body armour, and (b) the second has a modulus of elasticity of 130 GPa and is used as reinforcement in FRP composites in construction. The aramid fibre is an anisotropic material and, as is the case with all fibres, gives higher strength and modulus values in its longitudinal direction than in its transverse direction. The value of the longitudinal strength is 2.7 GPa and the fibre is heat and cut resistant. It retains its mechanical properties from cryogenic temperatures up to 400°C, is resistant to fatigue, both static and dynamic, and is elastic in tension but behaves non-linearly in compression; in addition, it has a ductile compressive characteristic. The fibre possesses good toughness and damage tolerance properties. It does not rust or corrode and its strength is not affected by immersion in water. The main weakness of aramid fibre is that it decomposes under alkaline or chlorine environments. Typical mechanical properties of aramid fibres

and their stiffness characteristics are given in Table 18.3 and Figure 18.4, respectively.

There are various types of aramid fibres on the market, including Kevlar, which is made in the USA, and Technora, produced in Japan by Teijin, who also produce Twaron.

Basalt fibres

Basalt fibres are manufactured from volcanic basalt rock, which is composed of plagioclase, pyroxene and olivine minerals. Not all basalt rock is suitable for basalt fibre production; as it is a natural material, its composition and properties vary, depending on geographic locations and cooling rate in particular. The basalt rock is crushed and melted at approximately 1500°C and extruded through a bushing to form continuous basalt fibre that is wound onto bobbins (the melt-spinning process). Basalt fibres have good thermal and chemical resistance (generally better than that of glass fibre), and have mechanical properties intermediate of those for E-glass fibre and carbon fibre. The mechanical properties of basalt fibre are provided in Table 18.3.

Fibre-reinforced polymer composites

Fibre-reinforced polymer composites are manufactured by the impregnation of the fibre into the matrix material to form a composite system and, in so doing, obtain a wide range of composites with varying mechanical properties. The fibre-matrix composite can have the properties of anisotropy or in-plane isotropy, by virtue of the arrangement and direction of the fibres in the matrix.

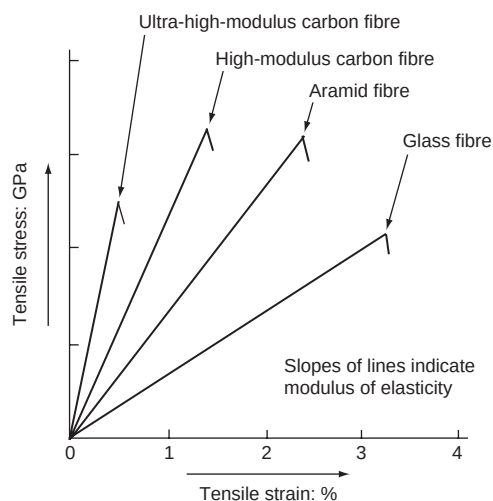


Figure 18.4 Typical comparative stress–strain characteristics of glass, aramid and carbon fibres

Mechanical properties of composites

The mechanical properties of the final fibre/polymer composite will depend on

- the method of manufacture
- the mechanical properties of the component parts (the various fibres and polymers)
- the relative proportions of the polymer and fibre (fibre volume fraction)
- the orientation of the fibre; namely, unidirectional (anisotropic composite), bidirectional aligned (orthotropic composite) or randomly oriented (quasi-isotropic composite))
- the long-term durability of the composite system (which will generally involve a reduction in property values of the composite system from those of its pristine condition)
- the creep of the composite.

The method of manufacture of the composite

There are a number of techniques for the manufacture of FRP composites, all of which have an influence on the mechanical properties of the final composite. As would be expected, the automated fabrication method tends to give higher values of strength and stiffness than the manual fabricated techniques because of the greater degree of process control, compaction and curing in the former techniques, thus providing a higher fibre volume fraction.

The various manufacturing processes given here are all relevant to bridge engineering and can be divided into three major divisions: (a) the manual production process, (b) the semi-automated process and (c) the automated process. A short discussion of these processes and an indication of when specific applications are used in practice is now presented.

Manual techniques

The manual processes used currently are a variation of the general wet lay-up method. The wet lay-up technique consists of in situ wetting of dry fibres in the form of sheets or fabrics impregnated in situ with a resin polymer. The sheets or fabrics are wrapped around the structural member or placed on the mould; their size will depend on the size of the member or mould, but fibre reinforcements are generally of widths varying between 150 and 1500 mm.

The *REPLARK method* was developed by the Mitsubishi Chemical Corporation, and uses a prepreg carbon fibre sheet, where the matrix material is an epoxy resin (Epotherm) and the fibres are Mitsubishi-manufactured fibres, unidirectionally oriented. The sheet has a paper backing, which serves to keep the fibres in position; the paper backing is removed when the sheet is positioned on the mould structure. The procedure employed to apply the carbon fibre reinforced polymer (CFRP) sheet is as follows.

- The first layer of impregnating polymer is applied to the prepared surface of the mould or structural member.
- The CFRP sheet, cut to size, is adhered to the structural member using the first laminating layer of polymer; it is pressed down on the surface using a roller to expel entrapped air between the fibres and the polymer.
- After the paper backing is removed from the carbon fibre sheet, the second layer of impregnating polymer is applied.
- The last two steps, application of polymer and sheets, are repeated in the case of a multi-ply system.
- The system is then allowed to cure.

The *Dupont method* involves using Kevlar fibres, and is marketed as a repair system for concrete structures. The application of the material to the surface to be retrofitted is similar to the REPLARK method.

The *Tonen Forca method* uses a unidirectional carbon fibre

sheet in an epoxy laminated system marketed in the UK by Kyokuto Boeki Kaisha Ltd. The system was originally developed by the Mitsubishi Chemical Corporation and is therefore similar to the REPLARK system.

These methods have been used in bridge engineering to confine columns or for strengthening or stiffening bridge beams. Further details on these methods may be obtained from Hollaway and Head (2001). All the wet lay-up methods either involve wetting a dry fibre fabric with resin followed by application onto the substrate or mould, or applying resin onto the substrate or mould and then dry fabric, repeated to the required design.

Other manual methods include the use of preformed FRP shells that are placed around, typically circular, columns or piers and then the annulus grouted – this method is particularly used for bridge or jetty piers over or in water.

Semi-automated processes

The semi-automated processes used currently are

- the XXsys Technologies method
- the resin transfer moulding (RTM) process
- the resin infusion under flexible tooling (RIFT) process
- the hot-melt factory-made pre-impregnated fibre (prepreg) method.

The *XXsys Technologies method* was developed in the USA by XXsys Technologies, Inc. San Diego, California, as a semi-automated process for seismic retrofitting and strength restoration of concrete columns using continuous carbon fibre. The XXsys carbon fibre composite jackets are installed using a fully automated machine called a Robo-Wrapper and a portable oven for curing. The technology associated with the technique is based on the filament winding of prepreg carbon fibre tows around the structural unit, thus forming a carbon fibre jacket; currently, the structural unit to be upgraded would be a column. The polymer is then cured by a controlled elevated temperature oven and can, if desired, be coated with a resin to match the existing structure. An advantage of this automated process is that the carbon fibre prepreg is impregnated with the polymer under factory-controlled conditions, providing good quality control and, as a consequence, a high strength-to-weight ratio. The equipment is erected on-site with minimum disturbance to traffic and the whole operation is undertaken in minimum time; the latter will, however, depend on the size of the job. The carbon fibre jacket that is eventually formed around the column will increase the axial and shear capacity of the column and will confine the concrete and greatly enhance its ductility in the flexural plastic hinge region. Furthermore, it will provide lap splice clamping and will prevent local buckling of the vertical reinforcement. For corrosion-damaged columns, the jacket can restore shear and axial capacity and will prevent spalling of the cover concrete.

Resin transfer moulding is a low-pressure, closed mould semi-mechanised process. In the RTM process, several layers of

dry continuous strand mat, woven roving or cloth are placed in the bottom half of a two-part closed mould and a low-viscosity catalysed resin is injected under pressure into the mould cavity, and cured. Flat reinforcing layers, such as a continuous strand mat, or a *preform* that has already been shaped to the desired product can be used as the starting material in the RTM process. The potential advantages of RTM are the rapid manufacture of large, complex, high performance structures with good surface finish on both sides, the design flexibility and the capability of integrating a large number of components in one part. This method can be used to form large components for all-composite bridge units but it is not often used; however, it is used to produce bridge street furniture.

There are three main industrial methods, all of which are a variation of the general RTM method; these are the Seemann composite resin infusion manufacturing (SCRIMP) method, vacuum-assisted RTM (VARTM) and thermal expansion RTM (TERTM).

The *resin infusion under flexible tooling (RIFT) process* is similar to the resin transfer moulding process but, rather than using two mould faces, the FRP component is applied to a single mould surface and then sealed using a flexible vacuum bag. Resin is then drawn and infused throughout the fibre fabric using a vacuum. Once the fabric is fully wetted by resin and the infusion is complete, the component is cured for the required duration and temperature (typically, room temperature cure for large components). Successful use of this system requires careful control of resin flow (e.g. through choice of resin, and the number and placement of resin injection lines). Many similar processes have been developed, all on the same basis of resin infusion of a fibre pack to form an FRP component (Williams *et al.*, 1996).

In the final method, the *hot-melt factory-made pre-impregnated fibre (prepreg)* is cured in either the factory, for the production of precast plates, or on-site if the prepreg composite is to be fabricated onto a bridge member. In the latter case, a compatible film adhesive is used and the adhesive and the prepreg components are cured in one operation under an elevated

temperature of 65°C applied for 16 h or 80°C applied for 4 h; a vacuum-assisted pressure of 1 bar is applied for simultaneous compaction of the composite and the film adhesive (Hollaway *et al.*, 2006).

Automated processes

The automated processes that are available to the construction industry are

- the pultrusion technique
- filament winding.

The *pultrusion technique* is used quite extensively in the construction industry and a large percentage of this use is associated with bridges. Sections are manufactured in a factory using a hot-cure resin; the dies operate at temperatures between 120°C and 135°C. The sections produced will generally be fully cured but this does depend on their size; large sections may require post-curing. Flat plates and various geometrical cross-sections can be produced; they are generally straight in the longitudinal direction, although products can be manufactured that are curved in plan. Care must be taken to ensure that (a) the fibres are well compacted into any bends in the cross-section (thus preventing voids forming), (b) there is complete wetting of the fibres in the pultruded unit (again, preventing voids forming) and (c) the fibres are well distributed in all cross-sections. It is not usual to have a fibre weight fraction (f.w.f.) greater than 60%, although uncomplicated sections, such as small-diameter rods, have been manufactured with 70% f.w.f. To provide resistance to hostile environments, a resin-rich exterior surface to the pultruded section can be fabricated using a surface veil, which is incorporated in the structural component at the time of manufacture. The limiting factor on the size of the unit and the complexity of the cross-section is the pull force required to draw the pultruded section through the die; the more complex the section, the greater the force due to friction of the unit within the die. Hydraulic

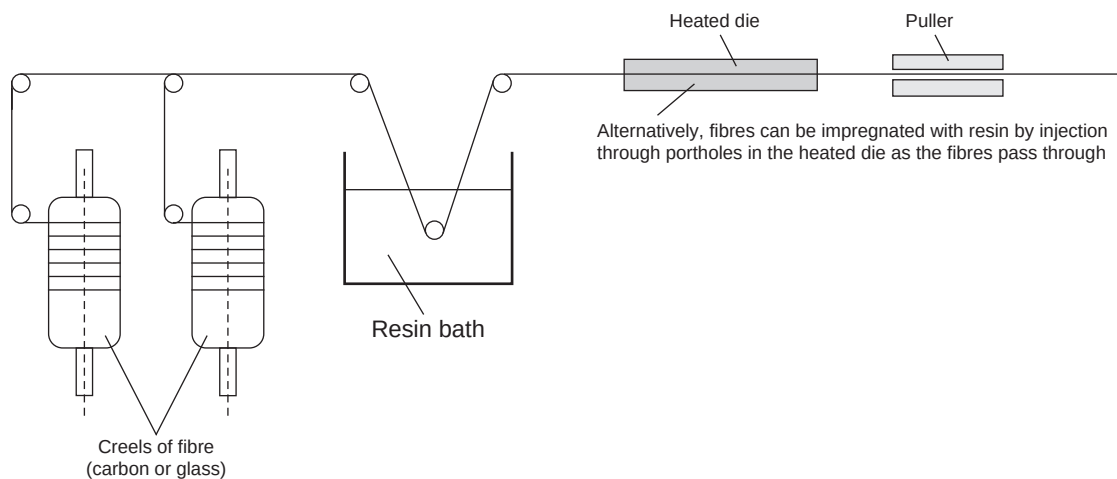


Figure 18.5 Pultrusion machine (adapted from Hollaway and Head (2001))

pulling force systems up to 50t on sections of 2.5m by 275mm thick have been in operation in the USA. Carbon, aramid and glass fibres and epoxy, vinyl ester and polyester materials have all been used for the production of pultruded units. The epoxy polymers are probably the most difficult to pultrude but do have low shrinkage during polymerisation (3%–4%). The vinyl ester polymers have a shrinkage value of (6%–10%) and the polyester polymers have a large shrinkage during polymerisation (12%–19%). Further information may be obtained from Hollaway and Head (2001) and Starr (2000). Figure 18.5 illustrates the pultrusion technique.

Some companies providing details of geometries and mechanical properties of pultruded profiles are Fiberline (Europe), Creative Pultrusions, Strongwell and Bedford Reinforced Plastics (USA).

The pultrusion technique is utilised in bridge engineering to produce plates and sections to upgrade and retrofit structural beams, the manufacture of ‘all-composite’ bridges, bridge decks, near-surface-mounted rods, internal reinforcement to concrete and bridge enclosures and fairings.

The *filament winding technique* is used to manufacture pressure pipes and to undertake wrapping of columns. There are two different winding methods: (a) wet winding and (b) pre-impregnated winding. The wet winding method uses continuous strands or roving of dry fibres from a series of creels, which are passed through a bath of cold-cure resin, cold-curing agent, pigments and UV retardants on to a ‘pay-out eye’, which is mounted on a moving carriage along the length of a constant speed rotating mandrel. The roving fibre delivery system reciprocates along the length of the mandrel and is controlled relative to the rotation of the mandrel to give the required fibre orientation. The speed of reciprocation and rotation are synchronised to hold a preset winding angle, typically 7° to 90° . The machine has the ability to lay fibres in any direction and to employ as many permutations of movements as is required by the structural design. After winding, the filament-wound mandrel is subjected to curing and post-curing operations, during which the mandrel is continuously rotated to maintain uniformity

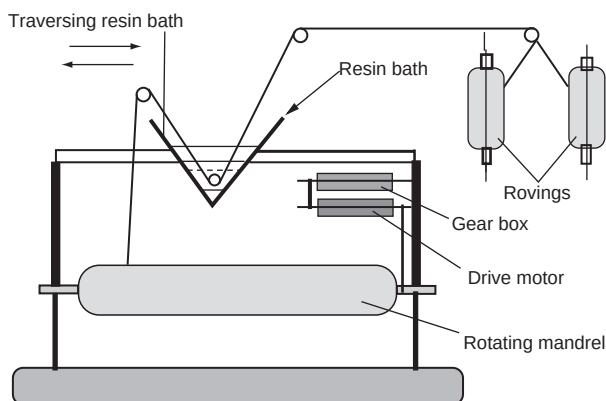


Figure 18.6 Diagrammatic representation of a filament winding machine (adapted from Hollaway and Head (2001))

of resin content around the circumference. After curing, the product is removed from the mandrel by either hydraulic or mechanical extraction. The pre-impregnated winding involves passing pre-impregnated fibres over a hot roller until they are tacky to the touch and they are then wound on to the rotating mandrel. Wet winding is the method usually used and has several advantages over pre-impregnated winding. These are low material cost, short winding time and the easy of varying the resin formulation to meet specific requirements. Figure 18.6 illustrates the filament winding technique. The XXsys Technology method, which is a site procedure for wrapping concrete bridge columns, is a filament winding technique; an example is shown in Figure 18.7



Figure 18.7 Column wrapping using XXsys technique (by kind permission of XXsys Technologies, Inc., San Diego, CA, USA)

Other automated manufacturing methods have been developed as the use of robotics has increased in the twenty-first century. These include such methods as automated tape-laying, where robotic arms lay tapes of material (typically carbon fibre) in a preconfigured pattern. However, these methods are not currently used for the manufacture of FRP bridge components.

Mechanical properties of the component parts of the composite

The properties of the composite greatly depend on the type of fibre used. The three main fibres used in bridge engineering, carbon, aramid and glass fibres, all have different stiffnesses; the higher the stiffness of the fibre (Table 18.4), the larger the stiffness of the composite. The orientation of the fibre also plays a large part in the stiffness of the composite. The strength of a composite also depends mainly on fibre type and orientation; however, a good bond between fibres and resin, with minimal defects (such as air voids or inclusions), is also important, as it delays the onset of delamination failure modes, which directly affect overall strength.

Material	Specific weight	Tensile strength: MPa	Tensile modulus: GPa	Flexural strength: MPa	Flexural modulus: GPa
E-glass	1.90	760–1030	41.0	1448	41.0
S-2 glass	1.80	1690.0	52.0	–	–
Aramid 58	1.45	1150–1380	70–107	–	–
Carbon (PAN)	1.60	2689–1930	130–172	1593	110.0
Carbon (pitch)	1.80	1380–1480	331–440	–	–
Basalt	2.10	>1500	>60	–	–
Flax	1.30	>250	40.0	–	–

Table 18.4 Typical tensile mechanical properties of long directionally aligned fibre-reinforced composites (f.w.f., 65%) manufactured by the automated process (adapted from Hollaway and Head (2001))

The relative proportions of the fibre and matrix

The fibre is the load-carrying component of the composite material, therefore, the greater the fibre volume fraction the stronger the composite. However, the maximum fibre volume fraction is typically limited to approximately 70% as poor wetting and bond between resin and fibres limits any further strength gain. More typical values vary from 45–65% depending on the manufacturing method, resin and fibre/fabric types.

The orientation of the fibre

The direction of the fibres, namely, unidirectional (anisotropic composite properties) or bidirectional (orthotropic (a special type of anisotropy) composite properties) angle ply or randomly oriented (quasi-isotropic), will determine the strength and stiffness of the composite. Figure 18.8 shows the relationship between the stiffness at angle β to the major axis (0° direction) for the different types of polymer fibre composite. It will be observed that, for the unidirectional fibre composite, the major axis (0° direction) of the composite gives the greatest stiffness and the 90° axis direction gives the least stiffness. Conversely, the randomly oriented fibre composite gives equal properties in all directions, from 0° to 90° .

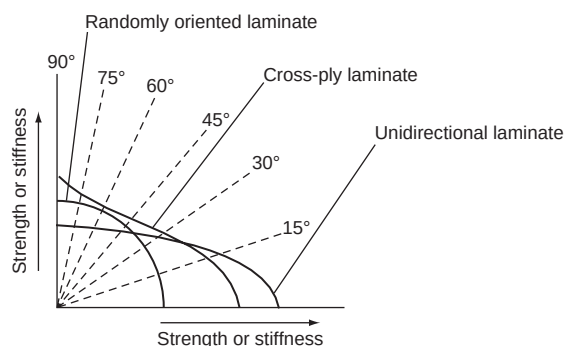


Figure 18.8 Relationship between the stiffness–strength at angle to the major axes (0° direction) for the different types of polymer fibre composites

The long-term durability of the composite

The civil engineering environments that cause the most concern for the durability of FRP composites are

- moisture and aqueous solutions (e.g. alkaline environments)
- fire
- thermal effects
- ultraviolet radiation.

Furthermore, the long-term loading conditions under which the material may have to function are

- fatigue
- creep.

Moisture, aqueous and chemical solutions

The durability of FRP composites is one area that does require further research to be undertaken to enable a greater understanding of the characteristics of the material over long periods of time. All engineering materials are sensitive to environmental changes in different ways, which may lead to their degradation over time. The FRP materials are no exception to this rule but they do offer some significant advantages over the more conventional construction materials. One such advantage is associated with their use as external reinforcement for RC or metallic bridge members, as they have good resistance to corrosion. A further example is the use of FRP rebars in RC members. One of the problems in acquiring data relating to the durability properties of any material is the length of time involved in gathering the relevant information. This is particularly difficult with respect to construction polymer composites, as there are many different polymers on the market, some of which have been modified by chemists over the years to improve their performance and, as a consequence, it would be hoped, their durability performance. Moreover, modern structural FRP composites have been used in bridge engineering construction only for some 30 years. The majority of those that are used will have additives, incorporated to reduce the length of cure time or to improve some specific mechanical or physical property. In effect, the addition of additives to composites is equivalent to adding impurities to the polymer; these additives tend to reduce other mechanical properties. Consequently, many of the FRP composite bridge components that are being, or have recently been, erected are monitored for any changes in their stiffness characteristics or any signs of degradation. Indeed, bridge structures are visually inspected at regular intervals and any degradation that has occurred will be reported and rectified accordingly.

To determine the durability characteristics of a composite as quickly as possible, test specimens are sometimes exposed to an accelerated test regime, which generally involves the

specimens being subjected to an environment many times more severe than that which would be experienced in practice. In addition, these test samples are exposed to an elevated temperature to further increase degradation. This method of accelerating the degradation of polymer composites should be used with caution. High temperatures are not relevant for most civil engineering polymer composites that operate at normal environmental conditions; furthermore, the degradation mechanisms in FRP materials at high temperatures are different from those under the lower practical temperatures. It will also be realised that, as the environmental temperature rises towards the glass transition temperature of the polymer, the latter will lose some of its stiffness and strength, with the result that the investigation will not be analysing the original material. The accelerated test procedures for obtaining durability data under one environmental situation will generally not be equivalent or relevant to the more gradual degradation effect had the environment been applied in a less rigorous manner. Moreover, materials used in construction would normally be exposed to many different environments acting simultaneously but in a less harsh way, and each environment possibly having an effect on the other.

Monitored field tests are the most relevant tests for establishing the durability characteristics of polymer composites used in bridge construction but the disadvantage of this method of obtaining information is the length of time involved. Hollaway (2007) discusses many field surveys of bridges and other structures that have been and are being undertaken throughout the world; the results from these tests have revealed or are revealing interesting discoveries regarding the resistance of FRP composites to natural or specific environments. An interesting example is associated with the degradation of glass FRP (GFRP) rebars in concrete. Accelerated laboratory test results of GFRP in a simulated concrete pore water solution of high pH and at elevated temperatures, up to 80°C, indicate that there is a decrease in the tensile, shear and bond strengths for that material (Bank and Gentry, 1995; Bank *et al.*, 1998; Sen *et al.*, 2002). Uomoto (2000) concludes that these results would suggest that there is a case for not using GFRP rebars in concrete. However, Tomosawa and Nakatsuji (1997) show that after 12 months exposure to alkaline solutions at a temperature between 20°C and 30°C there is no material or physical deterioration to the GFRP composite and Clarke and Sheard (1998) show likewise; after 2 years exposure to a tropical climate on a test platform off the Japanese coast, there has been no material or physical deterioration. Furthermore, the overall conclusions of the EUROCRETE project (Sheard *et al.*, 1997) were that GFRP is suitable in a concrete environment. Later in this chapter, a discussion is given of tests undertaken by ISIS Canada Research Network on the reliability of five GFRP reinforced concrete structures to provide information on the reliability of GFRP materials.

Owing to the increased use of GFRP reinforcement bars in concrete, a number of national standard specifications have been produced (e.g. ASTM D7957), which include test require-

ments for various chemical environments, including the retention of tensile strength after exposure to a specified alkaline solution and moisture absorption limits.

Methods to improve the durability of FRP composite materials in the civil infrastructure are discussed in Hollaway (2007), and will not be repeated here.

Fire behaviour of polymer composites

A major concern of the engineer in using polymers in the construction industry is the problem associated with fire. The polymer component of the composite is an organic material and is composed of carbon, hydrogen and nitrogen atoms; they are flammable to varying degrees. It is possible, however, to incorporate additives into the resin formulations, to add nano-clay particles to the pristine polymer (Hackman and Hollaway, 2006) or to alter the chemical structure of the polymer, thereby modifying the burning behaviour and producing a composite with a much enhanced fire property. A large number of intumescent and ceramified coatings are also available and used in various industries to provide the required fire performance; indeed, there are examples of the use of FRP materials for tunnel linings (e.g. in the Pajares Base Tunnel in Spain, see Mason (2018)), which have very severe fire performance requirements. Nevertheless, in extreme circumstances, fire can damage composites, and indeed all civil engineering materials. Many composites used in bridge engineering have high fibre volume fractions, hence the progress of the fire is reduced considerably, as the fibre does not burn and the penetration progress of the fire into the interior of the composite through the burning of the polymer is much reduced.

Fibre-reinforced polymer composites are widely used in the rehabilitation of bridge beams and bridge columns, which are constructed from either reinforced concrete or metallic materials. The potentially harmful effects of fire and high temperature on FRP-strengthened reinforced concrete and metallic bridge structures are well recognised in the literature, although few studies have been undertaken on this topic. For FRP-strengthened elements that rely primarily on good bond characteristics between the substrate and the FRP material, an organic adhesive is invariably the weakest link, as it usually has a relatively low glass transition temperature. The first studies to investigate FRP composites bonded to RC beams were conducted by Deuring (1994) and Blontrock *et al.* (2001) and limited laboratory experimental work has been undertaken on wrapped cylindrical columns after exposure to elevated temperatures (Saafi and Romine, 2002). Bisby *et al.* (2004) conducted a numerical and experimental programme to investigate the fire performance of FRP-wrapped full-scale reinforced concrete columns, in order to provide fire-design guidance. Numerical modelling is capable of predicting the thermal and structural response of an FRP-wrapped concrete column under exposure to a standard fire.

If fire resistance of FRP-strengthened elements is necessary, this will typically mean that additional coating or protection

to the FRP materials will be required, in particular, to delay and slow the increase in temperature of the adhesive material. Evidence of the adequacy of such fire protection systems must be obtained through testing.

Thermal effects

Coefficient of thermal expansion

The glass, aramid and carbon fibre components of fibre–matrix composites have a much lower coefficient of thermal expansion (CTE) in the longitudinal direction than the matrix material, and unidirectional FRP composite materials will have CTEs in the region of $10 \times 10^{-6}/^{\circ}\text{C}$, $-2 \times 10^{-6}/^{\circ}\text{C}$ and $-0.9 \times 10^{-6}/^{\circ}\text{C}$, respectively, depending on the type of the specific fibre; values for the matrix materials are given in Table 18.1. The fibres stabilise the composite system and reduce its coefficient of thermal expansion to a value nearer to that of conventional bridge construction materials, such as steel and concrete; the actual value will depend on the fibre volume fraction and fibre orientations. In addition, the value of the coefficient of thermal expansion will vary with the environmental temperature range in which the composite is placed. Furthermore, the degree of cross-linking of the polymer will also influence the rate of thermal expansion. The low CTE for unidirectional FRP materials, compared with that of most metals, is an important consideration for the adhesive bond, as stresses caused by the restraint of thermal expansion or contraction can be greater than those caused by traffic or similar load effects.

The low CTE and high modulus of elasticity of unidirectional CFRP material, in combination with GFRP bridge deck systems, opens up the interesting possibility of low or near-zero CTE bridges. This cannot be achieved with other bridge construction materials, and is particularly useful in reducing the size and cost of foundations, which otherwise have to resist high thermal effects and earth pressures (Bloodworth and Banks, 2018), with consequential cost savings and faster construction. The use of FRP materials for low or near-zero CTE bridges also removes the severe span and skew limitations that usually exist in national bridge design standards and guidance (Highways England, 2001) for steel or concrete integral bridge forms.

Thermal conductivity

This property is important when FRP composites are used in the production of FRP bridge decks. The thermal conductivity of polymers is low; consequently, they are good heat insulators. Thermal stress analyses have been undertaken by Alnahhal *et al.* (2006), utilising the finite-element method to predict the failure mechanisms and the ‘fire resistance limit’ of a superstructure of a bridge under extreme thermal loading conditions. The results were verified by using field results provided by the New York State Department of Transportation. In addition, damage simulations of the FRP deck as a result of

snow and ice clearance by a snow-plough were conducted to investigate any possibility of bridge failure after damage had occurred. Thermal simulations showed that FRP bridge decks are highly sensitive to the effect of elevated temperatures. The FRP deck approached the fire resistance limit at an early stage of the fire incident under all fire scenarios although for other FRP bridge decks this would depend on the existence of fire protection measures. The simulations of damage caused by a snow-plough showed minimal bridge failure under the worst-case damage scenario. These results provide an insight into the safety and reliability of FRP systems after the stipulated damage scenarios are considered. Alnahhal *et al.* (2006) discuss the recommended immediate actions necessary to repair the damaged region of the FRP deck panels.

The creep of the composite

The mechanics of creep in fibre–polymer composite materials are related to

1. the deformation characteristics of the creep of the viscoelastic matrix
2. the near-zero creep of the fibres
3. the stabilising effect of the fibres on the polymer, as a result of items 1 and 2
4. progressive changes in the internal balance of forces within the materials, resulting from the behaviour of the matrix, the fibre, the adhesion and the load transfer at the fibre–matrix interface.

The deformational behaviour of an FRP material is a function of the resin type, the fibre type and its architecture, fibre volume fraction, direction of heat flow and service temperature. The mechanical behaviour of composite materials can be established by applying constant loads over long periods of time; these investigations may be defined as creep tests. The tests produce curves of elongation against time at different stress levels; although they are not able to produce data that may be converted directly into stress–strain curves, constant time sections through families of such creep curves have been used to produce isochronous stress–strain curves (see earlier in this chapter).

Tensile and compressive properties of polymer composites

Tensile properties of composites

Table 18.4 gives typical properties of composites manufactured using long directionally aligned fibre reinforcement of glass, aramid, carbon, basalt and flax fibres with a fibre–matrix ratio by weight of 65%. Typical tensile mechanical properties of glass fibre composites manufactured by different techniques are given in Table 18.5; the effects that the methods of fabrication have on the properties is clearly shown. The variation in

Method of manufacture	Tensile strength: MPa	Tensile modulus: GPa	Flexural strength: MPa	Flexural modulus: GPa
Hand lay-up	62–344	4–31	110–550	6–28
Spray-up	35–124	6–12	83–190	5–9
RTM	138–193	3–10	207–310	8–15
Resin infusion (RIFT)	250–1000	15–40	300–1200	15–40
Filament winding	550–1380	30–50	690–1725	34–48
Pultrusion	275–1240	21–41	517–1448	21–41

Table 18.5 Typical tensile mechanical properties of E-glass fibre composites manufactured by different fabrication methods (adapted from Hollaway and Head (2001))

the composite tensile properties when the fibre–matrix ratio is changed but the method of manufacture and component parts of the composite remain constant is shown in Table 18.6.

Fibre/matrix ratio: %	Specific weight	Flexural strength: MPa	Flexural modulus: GPa	Tensile strength: MPa	Tensile modulus: GPa
67	1.84–1.90	483	17.9	269	19.3
65	1.75	406	15.1	214	15.8
50	1.80	332	15.3	166	15.8

Table 18.6 Typical tensile mechanical properties of glass fibre–vinyl ester polymer, with compression moulding and randomly oriented fibres (adapted from Hollaway and Head (2001))

Composites under compressive loads

The integrity of both component parts of the composite is far more important under a compressive load than under a tensile one. Furthermore, local resin and interface damage caused by compressive loading leads to fibre instability, which is more severe than the fibre isolation mode that occurs in tensile loading. The mode of failure for FRP composites subjected to longitudinal compression will depend on the type of fibre, the fibre volume fraction and the type of resin, and may include fibre microbuckling, transverse tensile failure or shear failure. The compressive strengths of CFRP and GFRP composite materials increase as the tensile strengths increase, but the aramid fibres in aramid fibre-reinforced polymer (AFRP) composites exhibit non-linear behaviour in compression at a relatively low level of stress.

The compressive modulus of elasticity of FRP materials is generally less than the tensile modulus. Test samples containing 55%–60% by weight of continuous fibres in a vinyl ester resin have compressive moduli of elasticity of approximately 80%, 100% (but the compressive strength is very low) and 85% of the tensile value for GFRP, AFRP and CFRP, respectively. Subramaniyan *et al.* (2003) show that the compressive strengths of GFRP composites can be improved by adding nanoclays to the polymer.

Adhesives

Structural adhesives used in bridge strengthening, in particular for bonding FRP materials to metal and concrete substrates, are usually epoxy systems and can be a two-part cold cure, a two-part hot cure or an adhesive film; the latter two require heat for polymerisation. A considerable amount of bonding, particularly when upgrading bridge structures, must be undertaken on-site and therefore cold-cure adhesives are generally used; the adhesive should have a T_g that is typically at least 15°C above the maximum operating temperature (typically a T_g of at least 60°C is acceptable in most bridge applications). If the bonding operation is undertaken in a factory environment, the hot-cure adhesive system can be used; the T_g then depends on the heat of polymerisation. The cross-linked chemical structure of the adhesive renders the polymer insoluble and infusible and these characteristics greatly reduce the creep of the adhesive. To increase the toughness of an adhesive polymer, it is possible to blend, fill or copolymerise it with a tough polymer. This will cause a reduction in the strength of the adhesive; in this situation, it is necessary to compromise between strength and stiffness.

A wider range of structural adhesive formulations are used for bonding different FRP components, and include polyurethane and methacrylate adhesives in addition to epoxy adhesives. Polyurethane and methacrylate adhesives typically have a lower elastic modulus (typically around 1 GPa or less) than epoxy adhesives with higher strain to failure, with the associated benefit of reduced adhesive stress concentrations at the edges of bonded areas.

Further information on adhesive bonding may be obtained from Mays and Hutchinson (1992).

Core materials

Core materials are used with FRP materials to form sandwich panels; this is an efficient method of improving the overall stiffness and strength of an FRP component. Core materials are combined with FRP face materials to form sandwich panels, and are often used to form lightweight bridge deck systems. The main core type used in bridge construction is rigid PVC foam core, although many other types, such as polymethacrylimide (PMI), polyetherimide (PEI) and styrene acrylonitrile (SAN). Balsa is also used as a structural core material in bridge components. The primary requirements for a structural foam core material are light weight, a good tensile bond strength to FRP face materials and adequate strength and stiffness to enable efficient transfer of in-plane forces between the FRP face materials, as well as to prevent buckling or wrinkling of the FRP face materials. Foam core materials can also be used in resin-infused bridge components as a type of internal mould to control the placement and spacing of FRP materials during the manufacturing process; in such cases, they may serve no permanent structural function and can have a density as low as approximately 50 kg/m³.

Honeycomb core materials are also available but are not commonly used for manufacturing FRP bridge components.

New bridge structures

The 'all-FRP composite' bridge

For FRP bridge systems to be successful, components should be modular and assembly should be rapid, simple and have reliable connections; the material should be durable. Any construction and long maintenance period causes disruption to the flow of traffic and is extremely expensive. Because FRP materials are durable and lightweight, they fulfil these requirements, provided the initial design of the basic building modular system is properly undertaken and the material properly installed.

During the early 1990s, three FRP composite bridges manufactured from modular components were the Aberfeldy footbridge, Bond's Mill Bridge, in Stonehouse, UK, and the Public Works Research Institute (PWRI) USA Composite Cable-Stayed Demonstration Bridge. This latter example used only mechanical fasteners in the form of (GFRP) bolts, with assembly and connection concepts similar to those of steel structures, whereas the composite units of the two former structures were adhesively bonded. The composite material used in these two bridges was GFRP and although this material is strong in tension it has a low modulus of elasticity relative to steel. If possible, the stiffness should be increased by shaping the modular system to give it enhanced flexural rigidity. These projects clearly showed the feasibility and potential for FRP bridges and constituted the pioneering work in this field; all three bridges, however, were demonstration projects. Although significant advantages for construction can be derived from the light weight and high strength-to-weight ratio of FRP composite bridge materials, the durability and long-term performance issues, particularly for the resin or adhesive polymers, still require further research and data development. It is essential that design, installation and management information and best practice are promulgated, based on sound research data, to ensure that the materials are used appropriately to achieve their maximum benefit and can be specified confidently. Long-term life-cycle cost models do not yet exist for these types of structure, but are being developed as further applications are implemented. A problem with the 'all-composite' structural systems is that, currently, they tend to be more expensive than bridge structures built from more conventional materials. However, the transportation, installation and disruption benefits derived from fast installation of the lighter-weight material do financially compensate the greater cost of the material. Furthermore, as the manufacturing procedures become more sophisticated and volumes increase, the cost will reduce.

The FRP experimental pedestrian bridge project features a three-span cable-stay bridge with an 11 m main span and 4.5 m side spans. The width of the walkway is 2 m. The total weight of the bridge is 4.4 t, resulting in a dead load/live load ratio of 0.3. The bridge is built on conventional reinforced concrete

foundations and anchored with steel anchor bolts. Pylons and deck are manufactured as pultruded GFRP sections, strengthened in some areas with carbon fibre sheets. The longitudinal girders are supported by transverse beams, which, in turn, are supported by CFRP cable stays. Both Leadline and Tokyo Rope carbon cables of different sizes are used as stay cables.

The Aberfeldy footbridge and Bond's Mill Bridge were manufactured from the first major UK, all-composite, versatile automated modular system, produced by Maunsell Structural Plastics (now AECOM), Beckenham, UK. This system was introduced to the construction industry as the Advanced Composite Construction system (ACCS) and was first used in modular form as the bridge enclosure to the A19 Tees Viaduct at Middlesbrough. Following this development, it was also used to form the deck and towers of the Aberfeldy footbridge and also fabricated into an all-FRP composite box beam, which formed the deck component of Bond's Mill Bridge. The modular ACCS relies on cold-cure adhesive bonding with an epoxy adhesive and a mechanical toggle system to join the individual plank units. It is extremely versatile in use, extending from bridges (including walkways) to building structures and to the Modispine cable support system, which was developed specifically for use in tunnels. Figure 18.9 shows a single Maunsell plank (the unit from which the Aberfeldy bridge was built) and ten of these planks fabricated into a box beam (used in Bond's Mill Bridge). The system was initially manufactured in the UK by GEC Plastics (now Exel Composites) by the pultrusion technique using isophthalic polyester resin and unidirectional, bidirectional and chopped strand mat glass fibre reinforcement for the main structural members. Strongwell, in Bristol, Virginia, and Chatfield, Minnesota, USA, now hold the manufacturing licence for the plank and produce similar panels under the trade name of COMPOSOLITE. In this form, it has continued to be used for lifting bridges, and footbridges in particular, with applications such as Chamberlain Bridge (a 9 m span bascule lifting bridge in Barbados, constructed in 2005), Wilcott footbridge (a 51 m span suspension footbridge with FRP deck, constructed in 2007) and St Austell footbridge (a 25 m length three-span footbridge over the railway, also constructed in 2007). The system was also used at Launder Aqueduct (a 36 m length three-span structure over the railway)

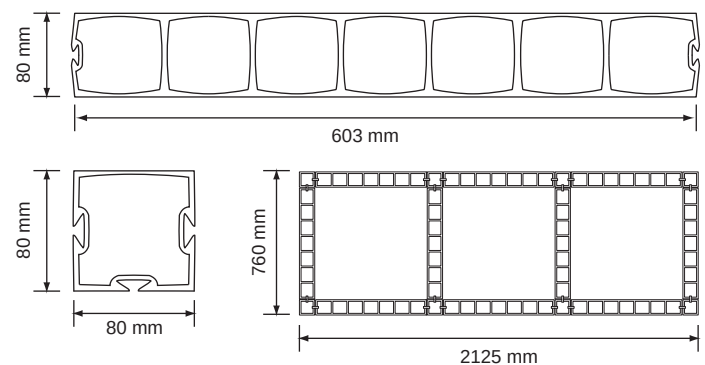


Figure 18.9 Maunsell plank and a box beam fabricated from ten Maunsell planks

and for other bridge enclosures. Further information on the COMPOSOLITE system can be obtained from Hollaway and Head (2001) and Strongwell, Bristol, Virginia, USA.

The single bascule lift bridge at Bond's Mill is shown in Figure 18.10. It consists of two epoxy bonded ACCS multi-cell box beams (Figure 18.9), which were in-filled with slow foaming epoxy foam of density 90 kg/m^3 (manufactured by Ciba Polymers) in the compressive flanges and webs of the $80 \text{ mm} \times 80 \text{ mm} \times 9 \text{ m}$ long cells of the ACCS modules. This material provided uniform support to the thin walls of the ACCS units, allowing load transfer without high local bending stresses. The bridge weighed 4.5 t. The running surface of the polymer composite bridge was made from ACME panels (a proprietary system of epoxy-coated panels with grit bonded or embedded in it), which were bolted to the top flange of the GFRP box beam.



Figure 18.10 Opening ceremony of the Bond's Mill lift bridge (span 8.2 m, width 4.27 m), Gloucestershire, UK, May 1994 (courtesy of Faber Maunsell)

Figure 18.11 shows the Wilcott footbridge in Shropshire, UK, over the A5 Nesscliffe bypass. It is a similar construction to the previous two bridges but was constructed from Strongwell's COMPOSOLITE panels, which were connected by three-way connectors and toggles; the panels were adhesively bonded to each other. The bridge is 50 m long and has a width



Figure 18.11 The Wilcott footbridge, Shropshire, UK, constructed from COMPOSOLITE (Strongwell, USA), a patented Maunsell plank (courtesy of Faber Maunsell)

of 2.25 m; it was built in three sections and spliced to fabricate the full length. It was opened in March 2003.

The production and material content of the ACCS or COMPOSOLITE planks have been optimised to provide highly durable and versatile components; in addition, structures can be formed quickly from a small number of standard components. As the material is lightweight, transportation and erection on-site is efficient.

A number of pultruded modularised sections suitable for bridge constructions are currently on the market, manufactured throughout Europe and North America; the manufacturers of these units include Creative Pultrusions, USA, Strongwell COMPOSOLITE, USA, Martin Marietta Materials, USA, Raleigh, North Carolina, USA, and Fiberline in Europe.

Many examples of the use of pultruded FRP bridge deck sections for bridges in the UK are described in Canning and Luke (2010), including footbridges, aqueducts, highway bridges and railway decking.

Biocomposites are also now starting to be used in bridge construction, primarily driven by developments in the automotive industry, where biocomposites are gaining interest owing to low carbon footprint and good recyclability. A 66 m long moving bridge in the Netherlands was constructed in 2019 using flax fibres and a resin blend incorporating epoxy (66%) and bio-resin (34%), with balsa as a core material (Vrieswijk and Herrema, 2020). Further research and applications on biocomposite materials, particularly regarding durability and creep, will enable increased use in bridges, to take full advantage of the material's recyclability and carbon footprint benefits.

The FRP–concrete beam construction (the duplex beam)

The procedures used to upgrade structures were the forerunners of a technique to combine FRP composites and concrete. This innovative idea was first introduced by Triantafyllou and Meier (1992); the basic concept of this construction has been developed independently by a number of researchers (Canning *et al.*, 2000; Hulatt *et al.*, 2003, 2004; Van Upr *et al.*, 2003). It currently involves placing the bulk concrete in the compression part of a rectangular or T-section beam and the FRP composite in the tension zone, this latter to resist flexural and shear forces on the beam. As the FRP composite can only be used in relatively thin plate form, the interior of the tension part of the beams may require a 'permanent shuttering' constructed from, say, a foamed polymer. The advantages of this new concept are a considerable reduction in beam weight, high load-carrying capacity and good fatigue behaviour. Figure 18.12 illustrates the technique used to form the duplex beam. The material manufacture and fabrication are described earlier in the chapter. Researchers at the University of Southern Queensland, Australia, have expanded the development into the construction industry, where two or three bridges have been built using this technique. From research work on the

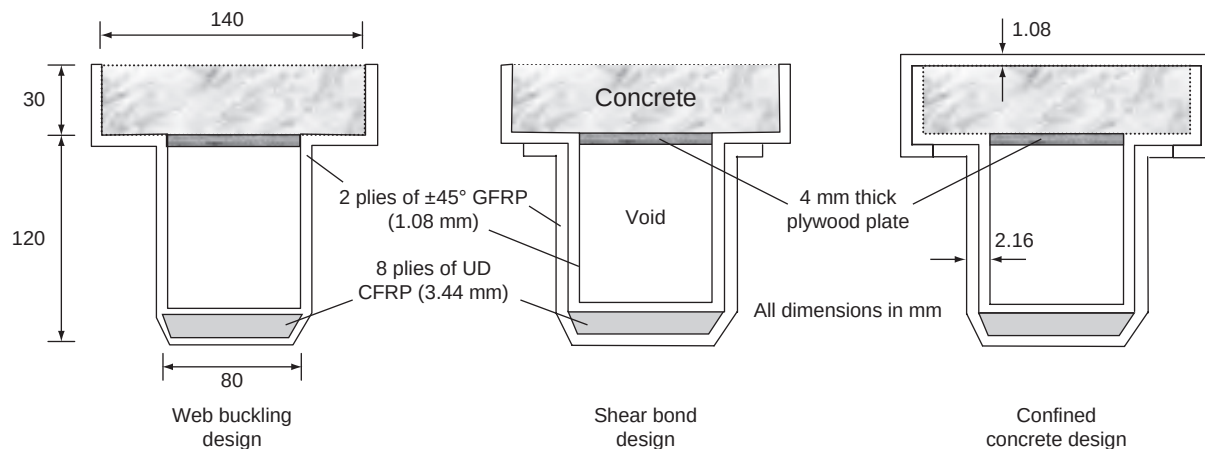


Figure 18.12 Cross-section of possible duplex beams developed at the University of Surrey (by permission of Thomas Telford)

combined FRP composite–concrete member (duplex system) undertaken at the University of Surrey, UK, an example of a bridge constructed using this system has been erected at Asturias Airport, Avilés, on the northern coast of Spain. The contractors were Necso Entrecanales Cubiertas, Madrid, Spain, who used material from the Advanced Composites Group Ltd, Heanor, Derbyshire, UK. The beam element utilises the high compressive strength of concrete and the high tensile strength of carbon fibre; the manufacturing method utilises hot-melt factory-made pre-impregnated FRP technology, using a cure temperature of 65°C for 16h and a vacuum-assisted pressure of 1 bar. The benefits of this system are the significant cost savings provided, owing to the lower weight, and the reduced through life cost of the beam. Another example of this form (FRP providing tension and shear capacity with compression capacity from another bulk construction material, such as concrete) is at Moss Canal Bridge in Rochdale, UK (Clapham *et al.*, 2015). An 8 m span highway bridge with a concrete deck in poor condition was replaced with a new deck comprising cellular FRP sections with a compression flange of polymer concrete, allowing reuse of the existing substructure.

A similar example of using a combination of FRP and other materials to their full advantage is the Hillman Composite Beam (HCB) system (Hillman, 2017). This system is used to form beams (either box or T cross-section) comprising a U-shaped GFRP shell, with galvanised prestressing wire inside the base of the GFRP shell, and an internal arch formed from self-compacting concrete. The HCB system has been used to construct road and rail bridges in USA since the first application in 2007.

FRP bridge decks

Deterioration of bridge decks is of critical importance and is the result of bridge decks reaching the end of their service life and degradation, owing to lack of proper maintenance, environmental conditions or poor initial construction. The

use of road salt for de-icing typically severely reduces the life of a conventional reinforced concrete bridge deck. Repair or replacement is then required and can be as much as 75% to 90% of the total annual maintenance cost of the structure (Karbhari *et al.*, 2001). When repair or replacement is imminent, there is not only the associated cost of materials and labour but also the cost of losses owing to delays and detours. Bridge decks of GFRP are a viable alternative that resolve many of these problems.

The modular use of FRP composite material is a possible solution for the rehabilitation of existing bridges and the construction of new ones. The deck part of the Aberfeldy footbridge and the Bond's Mill road bridge cannot be described as examples of FRP composite bridge decks, as these decks are an integral part of the longitudinal beams of the overall 'all-composite' bridge and cannot be removed from their superstructure. The FRP bridge deck is a component part of the bridge but can be removed from the primary component, namely, the main structural beams (the superstructure), which can be manufactured from either the more conventional civil engineering material or from fabricated FRP composites. The FRP bridge deck provides low weight, corrosion resistance and rapid installation, with minimum traffic disruption. The ratio of high strength to low weight enables the bridge deck to carry the existing designed traffic loads with little or no upgrading of the superstructure. The dead load of the bridge deck is about 20% of the weight of an equivalent size of a concrete deck and the deck can be erected much more quickly than concrete decks; FRP composite bridge decks have been used in the USA since the mid 1990s and have also been used for many new bridges and replacement bridge decks in the Netherlands (primarily resin-infused GFRP decks with rigid foam core), in particular since the 2000s (Smit, 2016). A recent interesting example of a footbridge that takes full advantage of the lightweight benefits and geometric freedom that FRP materials can provide is the Pont Y Ddraig (or Dragon's bridge) in Rhyl, Wales. The double bascule lifting bridge has two 36m spans, each with a 10m wide footpath, with a central caisson incorporating

the lifting mast (Hobbs, 2014). The FRP bridge comprises resin-infused glass fibre-epoxy skins with foam core and the local use of CFRP material for stiffening. Another example of resin-infused footbridges is the pair of identical 'twin' resin-infused 28 m span footbridges over the railway in Lancashire, UK. Each footbridge replaced an older tubular steel footbridge; lightweight FRP materials were chosen as they enable reuse of the existing substructure while minimising the need for expensive maintenance within railway closures (Canning and Halpin, 2019). These footbridges were a further development from a similar structural form used at the River Chor Aqueduct in Chorley, UK (Canning and Halpin, 2015), where an FRP structure was chosen for quick installation and good durability in a wet environment. A resin-infused FRP structure was also used to replace a wrought iron aqueduct in poor condition at the end of the Standedge Tunnel in the UK in 2018, again chosen for the light weight and durability.

The first FRP composite highway bridge deck (with the primary structure also manufactured from FRP beams) on a European public highway network spans the River Cole, at Westmill, in Oxfordshire, UK, which was opened in 2002. The span of the bridge is 10 m with a width of 6.8 m to carry two lanes of traffic and footpaths. The new bridge is composed of reinforced concrete spread foundations and abutments with brickwork facings, supporting four longitudinal beams and the bridge deck. The longitudinal beams were manufactured from pultruded GFRP composite box beams with pultruded unidirectional CFRP composite flanges to provide the required global flexural rigidity. The deck consists of a series of adhesively bonded GFRP ASSET or FB600 profiles (Luke *et al.*, 2002) spanning transversely, onto which the polymer concrete surfacing and epoxy wearing course for the carriageway are located. Figure 18.13 shows the completed bridge and its cross-section. This FRP deck system has also been used for Standen Hey bridge, a 10 m span replacement highway bridge over the railway in Lancashire, UK, but in this case with two bonded layers of the FBD600 profiles spanning longitudinally (Dawson and Farmer, 2009). A further optimisation of this form was used for the 8 m span Church Road bridge in Frampton Cotterell, Gloucestershire, UK, but with reduced fabrication and construction cost by using the FBD600 system supported on GFRP box sections with low-cost CFRP stiffening plates (Sebastian and Johnson, 2018). This bridge was particularly lightweight, with a total FRP weight of only 20 t for a deck area of approximately 100 m², which enabled simple transport and installation of the fully fabricated bridge. It is also the first ever instance where a FRP bridge structure was designed with a lower coefficient of thermal expansion (CTE) (approximately $7 \times 10^{-6}/^{\circ}\text{C}$) than that of

conventional steel or concrete bridges (typically assumed as $12 \times 10^{-6}/^{\circ}\text{C}$ in the UK), owing to the effective use of CFRP plates with negative CTE, as discussed earlier in this chapter.

In 2002, the New York State department constructed an FRP bridge deck as an experimental project. The aim of the project was to improve the load rating of a 50 year-old truss bridge located in Wellsburg, New York (Alampalli and Kunin, 2002). The FRP deck weighed approximately 80% less than the deteriorated concrete bridge deck it replaced. Reducing the dead load allowed the allowable live load capacity of the bridge to be increased without significant repair work to the existing superstructure, thus lengthening the bridge's service life. Load testing was undertaken after installation of the FRP deck to allow a study to be made of the design technique used and the assumptions made on the composite action between the deck and the superstructure, and to examine the effectiveness of joints in load transfer. The results indicated that the design was conservative. The design assumed no composite action between the deck and the superstructure, and the experimental data confirm that assumption. The study also shows that the joints are only partially effective in load transfer between panels. Peak strains under the test loads were only a very small fraction of the ultimate strength of the FRP deck.

If FRP bridge decks are to compete with concrete decks in many situations, the FRP decks must contribute to the load-carrying capacity of the top chords of the main girders; concrete decks are generally built with complete composite action with the main girders. This construction increases the stiffness of the girder; this is achieved using shear studs or stirrup connectors. However, FRP bridge decks constructed compositely with the main girders are generally joined by adhesive bonding. The stiffness of the FRP decks in the longitudinal axis of the bridge is not as great as that in the concrete construction; this has



Figure 18.13 The GFRP deck and GFRP–CFRP main beam (span, 10 m) construction of the bridge over the River Cole, Oxfordshire, UK; the main beams were manufactured from pultruded GFRP composites and strengthened and stiffened with CFRP composite plates on the tension and compressive flanges of the beams. (photograph courtesy of Mouchel Parkman)

been shown by Keller and Gürtler (2005). As a general rule, the contribution of the FRP deck to the load-carrying capacity of the top chord can only be achieved with spans less than 20 m; as the span of the bridge increases beyond this value, the contribution decreases to about 5%–10% of the total girder stiffness (Keller and Gürtler, 2005). Consequently, the current FRP bridge decks, which contribute to the load-carrying capacity of the bridge, are suitable for the smaller-span bridges up to about 20 m. If, however, the decks are to provide only transverse load-carrying functions of the bridge, span values are not limited.

Examples of the use of FRP bridge decks spanning transversely on fabricated steel girders can be seen at Mount Pleasant Bridge over the M6 motorway in England (Canning, 2008), comprising two 25 m spans, and Friedberg Bridge in Hessen, Germany (Knippers and Gabler, 2008), comprising a single 21 m span. The fabrication of Mount Pleasant Bridge was undertaken in controlled environmental conditions within enclosures on the hard shoulders of the motorway and the fully fabricated spans, each weighing approximately 100 t, were craned into position during an overnight motorway closure. The combination of FRP deck spanning transversely on steelwork girders provides a cost-effective use of materials to form a lightweight bridge for spans of up to 50 m, particularly where weathering steel girders are used to reduce maintenance requirements.

The features and benefits of FRP bridge decks are

- durability (highly resistant to corrosion and fatigue)
- light weight
- high strength
- rapid installation
- lower or competitive life-cycle cost
- high-quality manufacturing processes under controlled environments
- reduced carbon footprint owing to minimal material use and reduced traffic disruption.

Steel-free bridge deck

The steel-free deck system is an important bridge engineering technology developed in recent decades. By eliminating the internal steel reinforcement, the steel-free deck eliminates major bridge deck deterioration, particularly owing to the use of de-icing salts, and results in savings for bridge agencies. Eliminating corrosion makes concrete deck slabs virtually maintenance-free, which makes life-cycle costs of steel-free concrete decks much lower than reinforced concrete decks.

Shear connectors connect the steel-free concrete deck composite with the supporting steel girders. Owing to compressive membrane action, loads on the concrete deck between girders

cause arching and transverse thrust at the girder top flanges. External transverse steel straps are placed at the top flanges of the girders, thus resisting the thrust from arching action within the concrete deck. The width and distribution of cracks within the concrete deck are controlled by incorporating FRP crack control grids of either CFRP or GFRP (basalt-fibre-reinforced polymer (BFRP) is also now commercially available) near the bottom surface of the slab; this is shown in Figure 18.14. The ultimate load can be greater than the load at which the same deck would fail if it were reinforced conventionally. The tension capacity of the steel straps in the steel-free deck replaces conventional reinforcing steel. The external steel straps can be inspected and maintained in a similar fashion to steel girders.

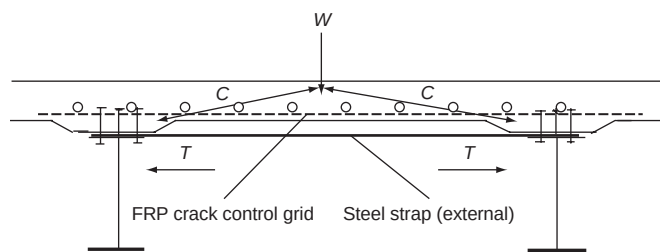


Figure 18.14 Position of FRP crack control grid in a steel-free concrete deck

In conventional slab-on-girder design, the longitudinal steel reinforcement in the deck contributes to the resistance of negative bending moments created at the internal piers of continuous bridges. The steel-free bridge deck is devoid of all internal steel reinforcement and hence requires an alternative design approach. A key aspect of this approach is the recommended use of FRP reinforcement bars to control cracking of the deck over the intermediate supports. Limiting these crack widths is essential to the durability performance of the concrete, particularly in freeze–thaw environments.

Reinforcement bars of FRP can also be used within the edge or parapet beams, as these elements are also otherwise commonly vulnerable to deterioration, owing to frequent splashing with water contaminated with chlorides from the passage of traffic.

Although not an FRP material, another material that has been used for bridge decks and complete bridge structures is recycled polymer or plastic. Using a proprietary blend of thermoplastic waste polymer materials, girders and decking units can be moulded and fabricated to form a variety of structures. This technology has been used to construct the world's first recycled plastic highway bridges, in 2009, in Fort Bragg, North Carolina, USA (Chandra and Kim, 2011). Following these successful applications, the same thermoplastic materials and components were used to replace two degraded timber railway bridges in Fort Eustis, Virginia, in 2010. Recycled thermoplastic HDPE components were also used for the construction of a bridge over the River Tweed in Scotland in 2011 (Lampo *et al.*, 2017), saving approximately 50 t of waste

material and showing the environmental benefits that can be achieved. The design of such structures uses relatively bulky components to limit permanent stresses and deflections, owing to significant creep and the relatively low elastic modulus of thermoplastic polymers. The high coefficient of thermal expansion for thermoplastic materials, such as HDPE, is typically up to approximately $100 \times 10^{-6}/^{\circ}\text{C}$, and hence thermal movements and restraint must be considered in any such bridge design.

Bridge enclosures and fairings

The need for regular inspection and maintenance of bridge structures is causing increasing concern because of the disruption caused to travellers if closure is required to maintain a bridge. Furthermore, the cost of closure will be extremely high. Moreover, stringent standards are increasing costs of maintenance work over or beside busy roads and railways. Many bridges designed in the past do not have good access for inspection and in Northern Europe and North America deterioration caused by de-icing salts is creating an increasing maintenance workload.

The concept of a bridge enclosure was developed jointly by the Transport Research Laboratory (TRL, formerly TRRL) and Maunsell (now AECOM), Beckenham, UK, in 1982 to provide a solution to these problems. The function of these enclosures is to erect a 'floor' underneath the girder of a steel composite bridge to provide inspection and maintenance access. In addition to providing these structural requirements, enclosures allow greater freedom of aesthetic expression, independently of the structural bridge requirements. The floor is sealed to the underside of the edge girders to enclose the steelwork and protect it from further corrosion. Research work undertaken at the TRL (McKenzie, 1991, 1993) showed that, once the enclosures are erected, the rate of corrosion of uncoated steel in the protected environment within the enclosure is 2%–10% of that of painted steel in the open. It should be emphasised that no dehumidifying equipment is needed to prevent corrosion. Although this enclosure space has a high humidity, chloride and sulphur pollutants are excluded by seals so that when condensation does occur (as in steel girders) the water drops onto the enclosure floor, which is set below the steel girders, and escapes through small drainage holes. The floor and fixings are non-corrosive and no water is able to pond against the steel; hence corrosion, of the latter material, is prevented.

The other major benefit of bridge enclosures is safety and ease of access for bridge inspectors. The use of a bridge enclosure avoids the need for other types of access, such as movable inspection gantries, roped access or scaffolding.

Enclosures will undoubtedly have even more important implications for the future design of long-span bridges. Currently, steel box girders are often used for the deck girders of such bridges in order to provide an aerodynamic shape to minimise exposed steel areas and to give adequate torsional stiffness. However, the development of cable-stayed bridges

and the reduction in the fabrication costs of steel girders compared with labour-intensive steel boxes has resulted in a recent increase in the use of plate girders for long-span bridges. The addition of FRP enclosures around such structures not only enables maintenance costs to be greatly reduced, but also enables the shape of the cross-section to be optimised by extending the enclosure into a fairing to give minimum drag, consistent with aerodynamic stability. Seven of the 24 new structures on the Second Severn Crossing incorporated FRP enclosure systems. When one of the structures was inspected by a Building Research Establishment (BRE) engineer in September 2001, the interior was found to be very dry and clean. The system was operating very satisfactorily with no sign of water ingress, damp, build-up of debris or corrosion. An important aspect to the system was that some ventilation was allowed and that it was not designed to be hermetic; thus, any condensation formed was removed. The exterior of the composite was clean, in spite of the dirt and spray from passing traffic and was aesthetically very pleasing. The structure of the approach road to the Second Severn Crossing (Figure 18.15) is one example where the GFRP enclosure is extended into a fairing. The construction period of this bridge was between 1992 and 1996.



Figure 18.15 Approach road to the Second Severn Crossing, showing the Maunsell composite enclosure and fairings (by kind permission of Maunsell Structural Plastics, Beckenham, Kent, UK)

Fibre-reinforced polymer composites are ideal materials from which to manufacture enclosure floors because they add little weight to the bridge and are highly durable, particularly as the FRP, being under the bridge soffit, is protected from direct ultraviolet light. This form of degradation, however, is no longer the problem that it was formerly, owing to improved resin formulations and the possibility of incorporating ultraviolet additives to the resins. Most bridge enclosures that have been erected in the UK have utilised FRP materials. The first major example of this technique was in 1988–1989, when the A19 Tees Viaduct at Middlesbrough (Figure 18.16) was fitted with the Maunsell Caretaker system. This was followed by further retrofit projects, one at Botley, Oxford (1990), where the hand lay-up GFRP method was used, and Neville's Cross

(1990), near Durham, where the pultruded GFRP system was fitted to an existing bridge over the main east coast railway line. Two new bridges were then built with enclosures, one at Bromley in south-east London (1992), which utilised the Maunsell Caretaker system; the other was in 1993, at Winterbrook. This bridge carries the A4130 Wallingford bypass over the River Thames and was designed by the Bridge Department of Oxford County Council; the enclosure was designed by Mouchel (now Jacobs). The structural steelwork of the bridge was enclosed by a number of GFRP panels and, as a curved profile of the bridge was required, the panel offsets from the plate girders were varied. Each panel was manufactured from single laminates, major ribs on the panel perimeter, minor ribs elsewhere and stainless steel inserts.



Figure 18.16 Maunsell Caretaker system used on the A19 Tees Viaduct at Middlesbrough (by kind permission of Maunsell Structural Plastics, Beckenham, Kent, UK)

Fibre-reinforced polymer bridge enclosures can also provide improved inspection access and durability for bridges over railways (where access for inspection can otherwise be difficult, disruptive and expensive), an example being Chislehurst Road Bridge where a pultruded GFRP enclosure (incorporating the COMPOSOLITE system) was installed in 2012.

In 1996, the Highways Agency, UK, published the design standard for bridge enclosures – BD67/96; the requirements for wind loading are covered by Eurocode 1 (BS EN 1991-1-4:2005) and its UK National Annex. When enclosures are placed under railway bridges, aerodynamic pressure caused by the displacement of air by the passage of a train is significant; therefore, the allowable deflection and the design of the fixings for the enclosure must be carefully considered.

The rehabilitation of the civil infrastructure – strengthening or stiffening of existing bridges

Introduction

Many of the bridges of the world are either structurally deficient or functionally obsolete. The definitions of these two terms are as follows.

- A structurally deficient bridge is one whose components may have deteriorated or been damaged, resulting in restrictions on its use.
- A functionally obsolete bridge refers to the geometrical characteristics of the bridge in terms of its load-carrying capacity. For instance, a bridge that was designed long ago for lower load levels, traffic volume or under- or over-clearance and that now requires restrictions to be imposed on its use is functionally obsolete, in spite of its good structural condition.

The US Federal Highway Administration (FHWA) has estimated that the federal government alone has invested over US \$1 trillion in the nation's highway system. Of concern, over 40% of the nation's bridges are structurally or functionally deficient. In California alone, over US \$3.5 billion is required for the seismic retrofitting of bridges. Consequently, the FHWA is stressing the use of asset management systems that will target the most economical allocation of resources in upgrading the nation's transportation system.

The corrosion of metallic structures has had a significant impact on the US economy, including infrastructure, transportation, manufacture and government. A study funded by the FHWA (Koch *et al.*, 2002) estimates the annual direct cost of corrosion for highway bridges to be US \$6.43 billion to US \$10.15 billion. This included \$3.79 billion to replace structurally deficient bridges over the next 10 years and US \$1.07 billion to US \$2.93 billion to cover maintenance and the cost of capital for concrete bridge decks. Furthermore, the life-cycle analysis undertaken estimated that, in addition to these direct costs, indirect costs, owing to traffic delays and thus loss of productivity, amounts to more than 10× the direct cost of corrosion. In the UK, steel–concrete composite bridges on the national trunk road network managed by Highways England are generally newer, but a large number of old (>100 years) wrought iron and early steel structures are to be found on the railway and canal network. In addition to corrosion, other problems relate to fatigue-sensitive details, the need to increase the service load and a historical lack of maintenance.

The deterioration of concrete infrastructure is a major civil engineering challenge facing the developed world. In the USA alone, it has been reported that 600 000 concrete road bridges are scheduled for repair at an estimated cost of US \$200 billion; four times the cost of their original construction.

In Europe, the situation varies depending on the country. For example, in France about 50% of more than 20 000 bridges located along the national road network need to be repaired. In the UK, concrete highway bridges were first constructed at the beginning of the twentieth century. The oldest reinforced concrete bridges and prestressed concrete bridges in the UK are over 100 years and over 56 years age, respectively. Surveys in the UK estimate that the cost of repairs, replacement or rehabilitation of deteriorated structures, mostly from the use of de-icing salts, will cost billions of pounds. In addition, a special construction programme in the UK (Bridgeward, see Bridle and Sims (2009)) has focused on the strengthening of a large number of existing bridges to allow the use of unrestricted heavy goods vehicles throughout Europe, and has continued for over 50 years into the twenty-first century.

Therefore, bridge engineers are faced with unique challenges as a result of severely deteriorating infrastructure and insufficient funding. However, great opportunities are presented for the utilisation of FRP composites for new bridge structures, bridge decks, strengthening of bridges and non-metallic rebars for reinforcing concrete using a material that is corrosion-resistant and lightweight and that can be rapidly installed.

Preparation of substrate surfaces for bonding like and dissimilar adherends

Before the rehabilitation and retrofitting of reinforced concrete and steel structures using the various FRP composite materials is undertaken, the surfaces of the adherends to be bonded must be prepared.

A clean surface is necessary for adhesion but it is not sufficient for bond durability. Most structural adhesion is the result of the formation of chemical bonds between the adherend surface atoms and the compounds constituting the adhesive. These chemical links provide the load transfer between the adherends. Solvent degreasing is important, because it removes contaminants, which inhibit the formation of the chemical bonds (Kinloch, 1987). Consequently, it is necessary to pretreat the substrate of the adherends to enable the required surface properties to be achieved. *This treatment will be different for different adherends*; the FRP composite materials are highly polar and hence very receptive to adhesive bonding, whereas the metals and aluminium adherends will require methods ranging from physical to chemical. Physical methods include solvent degreasing, abrasion and grit blasting, while chemical pretreatment includes etching and anodising procedures, by definition, causing chemical modification to the surfaces involved. Sometimes a primer solution is painted onto the surface of the substrate.

For most civil engineering structures, it is necessary for the adhesive to perform satisfactorily in service over a number of decades; this implies a careful selection of polymers, fabrication methods and a pretreatment of the adherends. The main environments that cause serious loss of joint strength in service are moisture and salt spray and these must be guarded against.

Concrete substrates

Concrete substrates are prepared by grit blasting; in the UK, angular chilled iron grit (Guyson, 1989) of nominal 0.18 mm particle size is commonly used. The grit is applied at a blasting pressure of 80 psi (550 kPa). With this operation, the surface cement layer (the laitance) must be removed, providing uniform exposure of the underlying aggregate. Before the adhesive polymer is applied to the surface, all traces of dust must be removed by air jet or similar and solvent cleaned. *The performance of the adhesive joint is directly related to the successful application of the pretreatment*, which in turn depends on the quality of the surface characteristics of the substrate in terms of topography and chemistry.

Steel substrates

Steel substrates require a clean rough surface on which to bond the FRP composite. To achieve this and a greater durability of the joint, solvent degreasing and grit blasting (e.g. to the Swedish code SA 2½) is often used. Silanes have also been shown to enhance the durability of bonded steel structures, but compatibility between the adherend and the silane must be achieved. Gettings and Kinlock (1972) found that premixing with γ -glycidoxypropyltrimethoxysilane (γ -GPS) considerably improved the durability of grit-blasted steel joints, whereas mixing with two other silanes did not. The substrate would then be finally solvent degreased again immediately before applying the adhesive. A primer coating is often used to prevent 'rust blooming' prior to bonding of adherends.

Polymer composites

Two of the most widely used preparation techniques for FRP materials are the abrasion method followed by solvent cleaning and the peel-ply method. Abrasion removes weak surface layers and contamination and increases the apparent surface energy and the rate of spreading of the adhesive. Although the degree of abrasion prior to bonding is known to affect subsequent bond strength and durability, the strength of bonded FRP joints depends on the roughness of the surfaces and the level of contaminants present. Consequently, contamination-free surfaces are the factor of overwhelming importance but exaggerated surface roughness might reduce joint strength, owing to the entrapment of air.

The FRP surface can be abraded using microfibre cloth, sand or silica carbide (SiC) paper or pumice; SiC paper followed by cleaning is convenient and will give high-strength joints.

Light grit blasting is an alternative technique. It can be applied to contoured surfaces but might cause loose grit handling problems if a recycled system is not used. It has also been found that, even with a low blast pressure and short treatment times, fibre damage is evident with most carbon and glass fibre-reinforced composites. The technique is not normally recommended for the preparation of FRP surfaces.

Peel-ply composites are adapted from the manufacture of

Property	Epoxy	Polyester ^a	Polyurethane
Tensile strength, MPa (ASTM D638): psi	28–90 4000–13 000	4–90 600–13 000	1.2–70 175–10 000
Tensile elongation (ASTM D638): %	3–6	2–6	100–1000
Compressive strength: MPa (ASTM D695): psi	105–175 15 000–25 000	90–205 13 000–30 000	140 20 000
Compressive modulus: 10 ³ MPa (ASTM D695): psi	–	2–3 300–400	0.07–0.7 10–100
Heat deflection temperature (ASTM D648): °C	45–260	60–200	–
Coefficient of thermal expansion (ASTM D696): 10 ^{–6} /°C	45–65	55–100	100–200

^a Rarely used in adhesives for civil engineering structures

Table 18.7 Typical physical properties of common adhesives used with concrete (Hollaway, 2008)

multilayer laminates built from glass and carbon fibre preregs. Peel-ply layers can also be applied to pultrusion composites; they are attached to the pultruded units during the manufacturing procedure. A peel-ply is a layer of nylon or polyester fabric incorporated onto the surface of the composite during manufacture. The peel-ply is stripped from the pultruded surface immediately prior to bonding to the adherend to give a clean, textured surface to the composite unit. The success of this procedure depends on the clean removal of the peel-ply without plucking fibres from, or leaving remains of the peel-ply on, the composite matrix. As such, most peel-plys are coated with a release agent to ensure that their removal does not damage the underlying plies. A great advantage of the peel-ply is that it protects the surface of the composite against contaminants and, on removal from the composite, provides a clean and roughened composite surface onto which the adhesive is applied immediately before the plate is offered up to the reinforced concrete beam.

Hollaway and Leeming (1999) recommend the peel-ply method, particularly when long-span beams (e.g. 18m span beams) are to be upgraded using strips of CFRP composite manufactured by the pultrusion technique. The peel-ply can be attached to one or both sides of the CFRP plate strip during the manufacturing operation. The CFRP plate is then rolled into a coil of 2–3m diameter, depending on the thickness of the plate, for transportation to the site. As the bonding operation progresses, the peel-ply is stripped from one surface of the composite; this surface is then ready to receive the adhesive.

For wet lay-up systems, no direct surface preparation is required, as the fibre sheets are laid onto the prepared surface of the adherend and impregnated in place. The application procedure for a wet lay-up method (e.g. REPLARK) was given previously in this chapter.

Further advice on surface preparation methods for bonding can be found in Cadei *et al.* (2004) and Darby *et al.* (2012).

Adhesive bonding of polymer composites to concrete surfaces

In discussing this topic, it is assumed that the surface of the concrete has been correctly prepared, grit-blasted and cleaned.

Solvent-free adhesive polymers, such as epoxies and their hybrids (e.g., epoxy polysulphides, epoxy urethanes) are used for bonding FRP composites to concrete surfaces; the physical properties of these adhesives are given in Table 18.7. These adhesives are generally formulated from epoxy resin, an amine or polyamide curing agent, reactive diluents and organic fillers and thixotropic agents.

The strength of an adhesive depends on

- the cohesive strength of the adhesive material
- the cohesive strength of the substrate materials; the tensile strength of the concrete is weaker than that of the adhesive polymer and therefore the concrete will generally be the critical failure point
- the adhesion of the adhesive to the substrate material (bond strength).

Adhesive bonding of polymer composites to steel adherends

In discussing this topic it is assumed that the surface of the steel has been correctly prepared, grit-blasted and cleaned.

The failure of a double strap joint specimen made from rigid CFRP composite plate or steel adherends using a two-part adhesive system would vary between a cohesive, an interlaminar and an interfacial mode. For a similar joint specimen made from a hot-melt pre-impregnated CFRP composite (prepreg) and compatible film adhesive bonded to a steel adherend, the failure mode is likely to be either interfacial or strain failure of the CFRP composite; in either case, the CFRP prepreg composite and compatible film adhesive would give a higher test failure result than the two-part cold setting adhesive (Photiou *et al.*, 2006a; Zhao and Zhang, 2007). The reason for the variation of the failure modes between these two methods of bonding is that a better compacted joint and an elevated temperature cure provides a better formed joint. Theoretically, the adhesive layer should not be the weak link in the joint; wherever possible, the joint should be designed to ensure that the CFRP adherend fails before the bond layer. However, in a steel–CFRP composite joint, the adhesive can be much weaker than the FRP composite or steel adherends, owing to the thickness of the adherends

compared with the adhesive; consequently, the bond stresses become relatively large and failure occurs. In a well-bonded steel–FRP composite joint, failure is likely to occur within the adhesive (cohesive failure) or within the adherend (FRP interlaminar failure).

Plate bonding

Plate bonding was pioneered using steel plates but there are several disadvantages to the use of this material as external reinforcement, namely

- the possibility of corrosion, which could adversely affect the bond strength
- uncertainties concerning durability and the effects of corrosion
- difficulty in shaping the plate
- difficulty in transportation and handling on-site
- the dead weight of the structure, which can be increased significantly
- the requirement for elaborate and expensive falsework
- the necessity to have flatness tolerance to prevent stresses being introduced normal to the bondline during curing.

To overcome some of these shortcomings, it was proposed in the mid 1980s that FRP plates could prove advantageous over steel plates in strengthening or stiffening applications (Kaiser, 1989; Meier, 1987; Meier and Kaiser 1991); FRP composites are unaffected by electrochemical deterioration and can resist the corrosion effects of alkalis, salts and similar aggressive materials at a wide range of temperatures (Hollaway and Head, 2001). The advantages of FRP composites over those for steel plate bonding are as follows.

- They have high specific strengths and stiffnesses in the fibre direction at a fraction of the weight of steel and therefore do not add significant loads to the structure after installation.
- They are easier to transport, handle and install on-site and therefore cause less site disruption, allowing faster and more economical strengthening.
- They require less falsework.
- They can be used in areas of limited access.
- They are non-magnetic and non-conductive.

As the FRP composites have low bending stiffness, continuous lengths can be delivered to the site in rolls; the inclusion of joints during installation is thus avoided. Generally, CFRP composites have excellent fatigue and creep properties and require less energy per kilogram to produce and transport to the site than is the case for steel plates.

There are some drawbacks to the use of FRP composite materials in the rehabilitating technique: (a) the intolerance to uneven bonding surfaces which may cause peeling of the plate;

(b) the possibility of brittle failure modes (Swamy and Mukhopadhyaya, 1995); (c) the higher material cost. However, in a rehabilitation project, where the material costs rarely exceed 15% of the overall project cost, and with the labour and equipment costs reduced, construction times will be shorter and durability of the overall system will be improved, the installation savings will offset the higher material costs.

Rehabilitating reinforced concrete bridge structures

Reinforced concrete bridge structures, for a variety of reasons, may be found to be unsatisfactory. In the design and construction phase, causes of deficiency include marginal design or detailing errors causing inadequate factors of safety, the use of inferior materials or poor construction workmanship or management, causing the design strengths not to be achieved. In service, changes in the use of a structure include

- *increased live load* – increased traffic on a bridge or changes in use of a structure, resulting in greater imposed live loads
- *increased dead load* – additional load on the structure, owing to new construction
- *increased dead and live loads* – widening a bridge to add an extra lane of traffic
- *structural modification* – cutting of voids in slabs and walls for services.

In addition, the load-carrying capacity of a member may be compromised by material deterioration, such as corrosion of internal reinforcement, particularly in marine or industrial environments, carbonation of the concrete, alkali–silica reaction or structural damage caused by fire, impact, explosion, earthquake or overloading. On highway structures, corrosion of the internal reinforcement is exacerbated by the application of de-icing salts. For prestressed concrete beams, strengthening measures may be required to prevent further loss of prestress.

There are two possible alternatives to restore a deficient structure to the required standards:

- complete or partial demolition and rebuild
- starting a programme of strengthening; in this context, strengthening is defined as rehabilitation to restore the original structural performance or upgrading to attain higher strengths or stiffness requirements.

The choice between strengthening and demolition depends on many factors, such as material and labour costs, time during which the structure is out of commission and the distribution of other facilities. However, the financial benefits of strengthening, as opposed to demolition, can often be considerable, particularly if a simple, quick strengthening technique is available. In addition, if the structure in question has historical importance, the possibility of demolition may be precluded.

Strengthening can be carried out using several techniques.

- The deficient member can be enlarged through the provision of additional reinforced or prestressed concrete layers, using stapling and pressure grouting.
- Additional supports, beams or stringers can be introduced, or non-structural toppings can be replaced with structural toppings or lighter materials or polymer impregnation. For bridge structures, traffic management measures may be imposed to relieve loading on weak members.
- High stiffness plates or bars can be externally bonded or mechanically fixed onto the soffit of the bridge in situ, effectively increasing the area of reinforcement provided. The plates (or bars) then act in conjunction with the original member, producing a section with improved flexural strength and stiffness. The success of adhesively bonded strengthening methods depends critically on the performance of the adhesive used.

The costs of the strengthening methods vary considerably, depending on the size of the structure, the extent of the strengthening work required and, in the case of bridges, the volume of traffic carried over and under it. In techniques where additional material is applied to the original member, the main problem is that of ensuring adequate connection and composite action between the reinforcing element and the existing structure. External post-tensioning by means of high-strength strands or bars has been successfully used to increase the strength of beams in existing bridges (such as at the Hammersmith Flyover in London, see Jackson and Moore (2017)). However, this method does present some difficulties in providing anchorage for the post-tensioning strands, maintaining the lateral stability of the girders during post-tensioning and protecting the strands against corrosion.

The practical techniques used for plate or bar reinforcement are

- flexural and shear strengthening or stiffening of RC, steel, timber and aluminium beams and RC slabs, by the utilisation of high-strength or stiffness plates, either steel or FRP composite
- near-surface-mounted (NSM) FRP rods for strengthening or stiffening RC, timber and masonry beams
- column confinement, using FRP jackets.

To upgrade structural beams made from RC, the FRP composite material used is either CFRP, AFRP or GFRP and these composites will be fabricated by one of three methods, namely

- the pultrusion technique, where the factory-made rigid precast FRP plate will be bonded to the degraded member
- bonding the factory-made rigid fully cured FRP pre-impregnated plate to the degraded member
- the wet lay-up process.

Currently, the pultrusion or pre-impregnation techniques are used for placements on the soffits of beams, with the following implications.

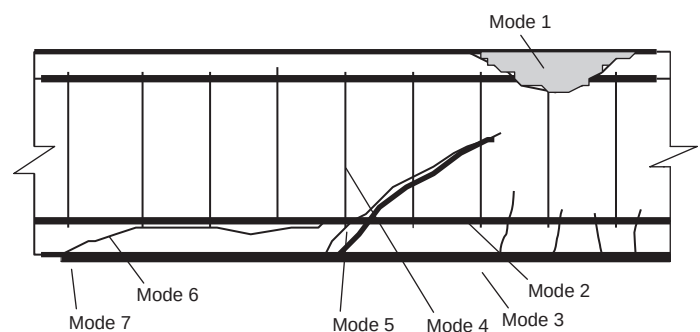
- The material cannot be reformed to cope with the geometries of the bridge member.
- A two-part, cold-cure, epoxy adhesive will be used to bond the plate to the substrate.

In these two cases, the effectiveness of the strengthening scheme depends highly on the bond between the composite and the concrete substrate and the condition of the cover concrete. Four areas relating to this topic require particular attention:

- the bond line thickness, which is difficult to control and can vary considerably over the length of the plate because of the undulating nature of the soffit of the concrete beams
- the long-term bonding characteristics and durability of the adhesive
- the durability of the FRP composite plate
- the polymerisation of the cold-cure adhesive; if not undertaken with care regarding the post-cure procedure, the polymer will not cure completely, with a resulting low T_g .

Figure 18.17 shows ten failure areas of a RC beam upgraded with an unstressed FRP plate. The following description has been developed from Hollaway and Leeming (1999).

- For an over-reinforced RC beam in an *unstrengthened* situation, flexural failure occurs as a concrete compression failure at the top flange (mode 1).
- For an under-reinforced RC beam, the initial failure occurs at yield of the steel tensile reinforcement (mode 2). With an increasing deflection but without any additional load-carrying capacity, the beam fails in concrete compression in the top flange (mode 1), owing to excessive deflection.
- For an originally under-reinforced beam, if the beam remains under-reinforced when strengthened with an FRP plate, the failure mode could be a tensile rupture of the laminate (mode 3).



Failure mode 1	Concrete compression failure
Failure mode 2	Yield of rebars
Failure mode 3	Tensile failure of FRP plate
Failure mode 4	Shear failure
Failure mode 5	Peel due to vertical movement at shear crack
Failure mode 6	Anchorage peel/shear in cover zone
Failure mode 7	Peel failure
Failure mode 8	Adhesive failure at concrete-adhesive interface
Failure mode 9	Adhesive failure at adhesive-FRP interface
Failure mode 10	Interlaminar shear within the FRP plate

Figure 18.17 Typical failure modes for strengthened beams (adapted from Hollaway and Head (2001))

- For a beam overstrengthened after plate bonding, flexural failure occurs as a concrete compression failure in the top flange (mode 1). Yielding of the steel reinforcement is likely to occur before either the concrete or the CFRP plate fails; while this may contribute to the ultimate failure of the beam, it is not the prime cause of failure. At the termination of the plate (the plate's free end), there are large stresses normal to the plate; these will cause the plate to peel off towards the centre of the beam. This is known as end anchorage peel (modes 6 and 7).
- For upgraded beams, there is also a peel failure mode at a shear crack (modes 4, 5 and 8), where there is a possible complex mechanism of debonding, owing to strain redistribution in the plate at the crack or the formation of a step in the soffit of the beam, causing shear peel. The delamination can then propagate towards the end of the plate. Whether modes 5 or 8 occur depends on the structure of the shear reinforcement in the unstrengthened beam.

A number of other possible but unlikely modes of failure have been identified in the literature, such as delamination of the composite plate or of the area within the glue line, but these have not generally been experienced; the strength of these materials is greater than that of concrete and the failures will only happen if the installation has been poorly performed or there is a defect in the manufacture of the plate.

Further information on the technique and analysis of retrofitting FRP composites to reinforced concrete may be obtained from Hollaway and Leeming (1999), Oehlers and Seracino (2004) and Teng *et al.* (2002).

There are several design guides for the design calculations for retrofitting of FRP composites to reinforced concrete structures, used through the world; these are given in the appendix; FRP strengthening and upgrading systems are now widely used on major bridge structures around the world, such as at West Gate Bridge in Australia (Williams *et al.*, 2010).

Fibre composite tendons for prestressing concrete

A number of studies have been undertaken of the rehabilitation of prestressed concrete (PC) and cable-stay bridges utilising prestressing bars and tendons made from FRP composites (Arockiasamy *et al.*, 1996; Balázs *et al.*, 2000; Burke and Dolan, 2001; Fam *et al.*, 1997; Grace, 1999, 2000; Lu *et al.*, 2000; Saadatmanesh and Tannous, 1999). Nordin (2004) reports on a number of existing concrete bridge girders that have been rehabilitated and strengthened with external FRP tendons. The advantages of using FRP reinforcement compared with steel are its non-corrosive, non-magnetic, high strength and lightweight properties. However, many designers have been deterred from utilising FRP as tendons more widely because of their initial high cost and the increased quality control required during their manufacture. The cost of materials used during the rehabilitation of bridges is generally only a small percentage of the overall cost; when the whole-life costing for the upgrading of the bridge, including durability, is calculated, it would be shown that rehabilitation using composites would be cheaper

than that for conventional materials. Rizkalla *et al.* (1998) and Shehata *et al.* (1997) report on some experimental precast concrete bridges built in Canada that have been prestressed with FRP composite tendons; these illustrate the advantages of the prestressing systems. A number of design guidelines (ACI, 1996, 2002a, 2002b; BRI, 1995; CSA, 2000; ISIS Canada, 2001) have been developed for structural concrete reinforced with FRP composites. It seems unlikely that, in the immediate future, FRP tendons will gain widespread acceptance in construction without an initial economic incentive to use them.

Some properties of FRP composite and steel tendons are given in ACI (1996) and Schupack (2001). Strength, stress-strain, creep and fatigue are the main properties affecting the performance of prestressed tendons. In addition, it is important to consider the size effects of the tendons, as the strength of FRP reduces as size increases. This effect is attributed to shear lag as bond stresses are transferred to the core of the tendon through internal shear stresses (ACI, 1996; Ganga Rao and Faza, 1992; Nikolic-Brzev and Pantazopoulou, 1995). The failure of FRP composite tendons is generally through fibre failure.

To prevent stress corrosion (creep rupture), GFRP composite tendons must not be loaded to a value greater than 20% of their ultimate strength; CFRP composite tendons have the best creep-rupture performance of the three composite tendons (GFRP, AFRP and CFRP), with a value of about 95% of the ultimate strength. The draping of the tendons that takes place in a prestressed girder also places a limit on the FRP tendon stress.

Both CFRP and AFRP composite tendons show superior fatigue performance to that of steel (Schwartz, 1992). In addition, the FRP composite tendons have a good chemical stability in hostile environments and CFRP composite tendons have superior durability performance in moisture, alkaline and acidic environments, compared with steel tendons.

Three systems are commercially available; these are Polystal, Arapree and Parafil, described next.

Parafil ropes

Parafil ropes consist of a closely packed, parallel-laid, high-strength, synthetic core protected by an abrasion-resistant polymeric sheath. There are three types of Parafil, depending on the type of fibre used; these are polyester, standard modulus aramid (Kevlar 29) and high modulus (Kevlar 49). Four types of sheath are available: polyethylene, polyethylene-EVA copolymer, polyester elastomer and flame retardant. The Parafil product range is summarised in Table 18.8 (Parafil is a registered trademark).

The most suitable rope for use in prestressing tendons is the Type G rope, which contains Kevlar 49 as its core yarn. The elastic modulus is about 120 GPa, with a short-term ultimate tensile strength of 1930 MPa. The rope will creep to failure at high loads. Extrapolation from short-term tests, combined with predictions based on the reaction rate theories of chemical processes, predicts that a Parafil rope will sustain a load of 50%

Yarn	Sheath			
	Polyethylene	Polyethylene-EVA copolymer	Polyester elastomer	Flame retardant
Polyester	Type A	Type A/C	Type A/H	Type A/X
Standard modulus aramid	Type F	Type F/C	Type F/H	Type F/X
High modulus aramid	Type G	Type G/C	Type G/H	Type G/X

Table 18.8 Three standard types of Parafil rope (from Linear Composites Ltd, 2021a)

of the short-term strength for 100 years (Chambers, 1988). Measurements made on Parafil Type G ropes have produced the following creep coefficient equation (Burgoyne and Guimaraes, 1992):

$$\varepsilon_t = (0.012 \pm 0.003) \log_{10} t \quad (18.1)$$

where $\varepsilon_t = \{\varepsilon_0(t)\} / \varepsilon_0$ is the creep coefficient, $\varepsilon_0(t)$ is the creep strain at time t and ε_0 is the initial strain.

Observations from strain rupture work and creep data analysis show that Type G has a limiting creep strain, irrespective of the initial stress, between 0.10% and 0.12%, as described in Linear Composites Ltd (2021b).

The stress relaxation of Type G Parafil can be given in the following relationship (Chambers, 1986).

$$r = 1.82 + 0.0403f + 0.67 \log_{10}(t - 100) \quad (18.2)$$

where r is the stress relaxation, expressed as a percentage of the normal break load (NBL), f is the initial stress, expressed as a percentage of the NBL and t is the time (in hours).

At, say, 60% NBL, the relaxation over 100 years is 8.2% NBL. This equates to a relaxation of 13.6% of the initial stress.

Tendons used in cable-stay bridges are expected to have high durability in normal environments. Kevlar degrades in ultraviolet light, but the fibre is shielded by the sheath; therefore, this type of degradation does not occur. In addition, Kevlar fibres suffer from hydrolytic attack by strong acids and alkalis, but the tendons would not be bonded directly to the concrete and are shielded; again, they will not suffer an attack for this cause.

Corrosion resistance

As Parafil ropes are manufactured from materials that possess a high degree of mechanical toughness and are inert chemically, they have a strong resistance to the corrosion action of salt water, most inorganic salts and acids. Furthermore, there is poor adhesion between ice and the smooth water-repellent surface of the Parafil ropes.

Anchorage

In post-tensioning applications of FRP tendons, there are three main types of anchor – the mechanical gripping, the bond-type anchor and the solid conical cone – which hold the separate tendons. The first holds the tendons by external radial pressure,

the second through a bonding material and interface shear and the third through a type of mechanical gripping, where a number of tendons can be gripped using a conical solid core, which must be well forced into an outer conical socket. The socket contains both the conical core and the tendons, to give greater shear resistance; the outer parts are sealed with resin to prevent moisture uptake (Burgoyne, 1998; Erki and Rizkalla, 1993). The wedges used in the first and second anchor types are generally manufactured from steel material but can be made from polymers. Reda Taha and Shrive (2003) have developed a new anchor concept, in which the outer barrel and the four-piece wedges are made from non-metallic material. When mechanical gripping steel wedges are used, premature failure of the FRP composite might occur at the anchorage position.

These ropes are not strictly reinforced polymer composites because the reinforcing fibres are not embedded in a polymer matrix. The sheath is used merely to protect the fibres from abrasion and weathering.

Polystal

Polystal tendons consist of bundles of bars or rods, each containing E-type glass fibre filaments in an unsaturated polymer resin. The diameter of a typical bar would be 7.5 mm and it would have a fibre volume fraction of 68%. Nineteen of these bars would be grouped together to give a tendon working load capacity of 600 kN. Polystal is produced by Bayer AG in association with Strabag AG, Germany.

It should be noted that glass fibres, under long-term loading of magnitude greater than 20% of the ultimate value would suffer from stress corrosion (or stress ageing), in which cracks develop at the surfaces of flaws and propagate through the fibre thickness.

Polystal tendons would have to be protected against overheating, particularly in the anchorage zone. This would be undertaken by structural means, such as increased concrete cover. Table 18.9 gives values of residual strengths of various prestressing tendons.

Materials	Residual strengths: %			
	150°C	200°C	300°C	400°C
Prestressing strand	100	90	70	50
Arapree ^a	100	100	100	80
Polystal	95	90	80	55
FRP rebar	90			

^aHeated for one half hour

Table 18.9 Comparison of residual strengths at elevated temperatures (values taken from Clarke (1993a))

Arapree

Arapree consists of aramid filaments embedded in an epoxy resin. Although aramid is very strong and can resist hard treatment, it has been shown that effective use could be improved

by impregnating the bare fibre bundle in resin to facilitate handling, improve alkali resistance and activate the real tension strength of the material. Arapree, therefore, is manufactured from a pultrusion of the aramid fibre Twaron in an epoxy resin. It was developed by AKZO in association with Hollandsche Beton Groep (HBG) in the Netherlands and is now produced by Sireg S.p.A. in Italy. The tendons rely on the bond between the concrete and the pultrusion resin; this bond is provided by silica particles bonded to the surface of the composite. The properties of Twaron are very similar to those of Kevlar, including the relaxation figures, and it gives a higher overall strength than Parafil; the local failure of some fibres would not cause its strength to be lost over the whole length of the tendon.

The standard types of Arapree element are circular and rectangular in cross-section. Both consist of up to 400 000 filaments of aramid. The ultimate tensile strength and modulus of elasticity of 200 000 filaments is 67 kN and 130 GPa, respectively; Table 18.10 gives the mechanical properties of the material.

Material property	Value		
Arapree rod size	5.5	7.5	10
Nominal diameter: mm	5.5	7.5	10
Tensile strength: MPa	1400	1400	1400
Tensile modulus: GPa	60	60	60
Ultimate deformation: %	2.4	2.4	2.4
Maximum axial load: N	32 000	62 000	109 000
Poisson's ratio	0.38	0.38	0.38

Table 18.10 Mechanical properties of Arapree rectangular strips at 20°C (derived from the manufacturer's data sheets)

Arapree tendons exhibit excellent resistance to chlorides and many other environments aggressive to steel. Specifically, the insensitivity to chlorides, such as de-icing salts, offers opportunities to overcome a range of existing deterioration problems in concrete structures.

As Arapree is an organic material, for service temperatures higher than 100°C the strength will start to decrease and at 150°C when loaded continuously for 103 hours it will have decreased to 90% of its initial value.

The first prestressed concrete bridge to be built using glass fibre-reinforced prestressing strand was a small footbridge in Düsseldorf, which was completed in 1980 (Weiser, 1983). This bridge was essentially designed as a reinforced concrete bridge, allowing some of the tendons to be removed for testing.

A number of bridges have been built worldwide utilising prestressed FRP cables but generally using FRP rebars for the unstressed concrete slabs. A total of five road bridges and footbridges have been built in Germany and Austria utilising Polystal glass fibre composite tendons (Wolff and Miessler, 1993). The first highway bridge, which was opened to traffic in 1986, was the Ulenbergstraße Bridge in Düsseldorf. The bridge is 15 m wide and has spans of 21.3 and 25.6 m. The slab was first post-tensioned with 59 Polystal prestressing tendons,

each made from 19 glass reinforced polymer rods of nominal diameter 7.5 mm. These tendons were anchored to a designed block and each tensioned to a working load of 60 kN; 4 t of glass reinforcement polymer prestressing tendons were used. This bridge has been monitored and test loaded periodically since it was opened. In Japan, where a total of ten bridges have been built since 1988, the emphasis has been on the development of carbon and aramid fibre tendons (Noritake *et al.*, 1993; Tsuji *et al.*, 1993). Carbon fibre has also been used on one bridge in Germany and aramid fibre tendons for a cantilevered roadway in Spain (Casas and Aparicio, 1990).

One bridge in North America, at South Dakota, has been stressed using glass and carbon fibre composite tendons (Iyer, 1993) and a bridge in Calgary, Canada, has been built using carbon fibre composite strands (Anon, 1993).

Fibre-reinforced polymer composites have also been suggested as alternative materials for bridge cables. In particular, CFRP cables provide excellent tensile strength, fatigue, creep and sag behaviour, and would enable increased spans for ultra-long-span applications (Meier, 2012) of over 4000 m.

Seismic retrofit of RC structures

Fibre-reinforced polymer composite jackets have been demonstrated to be an effective means of providing lateral confinement for the seismic retrofit and strengthening of reinforced concrete columns. The retrofit technique and application of the FRP composites to the degraded structure is similar to that applied for the rehabilitation of degraded structures and shows good environmental durability. The composite material can be applied to the columns in the form of a prepreg or by dry fibres, which are then impregnated with a resin (see the section on the method of manufacture of the composite for the description of the wet lay-up). An appropriate post-fabrication surface treatment is invariably applied by painting or rendering.

Near-surface-mounted (NSM) FRP rods

Recently, NSM FRP composite rods to strengthen the flexural and shear components of reinforced concrete beams have been utilised (De Lorenzis *et al.*, 2000a). In this application, pre-formed grooves are formed in the soffit of the RC beam to be rehabilitated using customised grooving tools, which form the groove in one operation. A bonding agent is then placed in the groove by gun application and the FRP rods are bonded into these grooves. This is undertaken in the longitudinal direction of the beam to the desired depth and width and involves minimal installation time, compared with that of the external plate bonding technique. There is no limit to the cross-sectional shape of the FRP reinforcement, which is manufactured by the pultrusion technique; however, different cross-sectional shapes will result in different bond performance and varying efficiencies of the system. The NSM bars are identical to those used for FRP rebars. In addition to the upgrading of reinforced concrete beams, NSM bars have been used for the strengthening of deficit timber members and masonry walls. Currently, there is

some discussion as to whether the free ends of the rods require further anchorage bonding; one school of thought suggests that the rods have sufficient bond length without additional anchorage, whereas the other school of thought suggests that the rods should be anchored by other means. Further investigative work on this problem is being conducted. Moreover, of vital importance is the understanding of the interaction between flexural or shear cracking and bond stresses. A section of an embedded NSM FRP rod is shown in Figure 18.18. This technique can only be undertaken satisfactorily if there is sufficient depth in the cover concrete to accommodate the groove size. If the cover is not sufficient, the groove might be placed in the vertical part of the RC beam but this position will, of course, not be as efficient as the case for soffit-mounted rods.

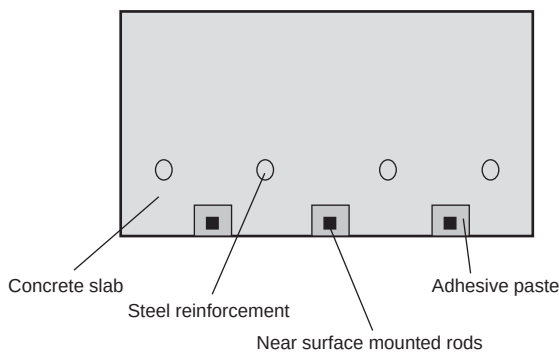


Figure 18.18 Reinforced concrete slab showing embedment of NSM rods

De Lorenzis *et al.* (2002) and El-Hacha and Rizkalla (2004) indicate that NSM reinforcement can significantly increase the flexural capacity of RC elements, compared with externally bonded laminates. El-Hacha and Rizkalla (2004), in their investigations on the strengthening of reinforced concrete T-beams using identical CFRP strips, both as NSM and externally bonded reinforcement, show that the NSM rods failed by tensile rupture of the fibre at a much higher load than that of the externally bonded strips. The latter failed by early debonding. Nevertheless, NSM rods can fail by debonding; this failure may be the limiting factor for the efficiency of this technology.

There are numerous applications where NSM rods have been employed, some examples are the strengthening of the Prairie Surf Studios, Oklahoma City, USA, in 1997–1998 (Emmons *et al.*, 2001; Hogue *et al.*, 1999).

The CFRP composite material would generally be used for strengthening concrete and steel members but for strengthening timber and masonry structural members GFRP composite material would be used. This is because the latter material has a much lower modulus of elasticity than the former and a comparable value to that of the material being rehabilitated. The failure mode of a timber beam upgraded with GFRP changes from a brittle-flexural failure in the plane timber specimen to a more ductile compression-flexural failure in the strengthened specimen. Furthermore, FRP rods have been used for the structural repair of masonry structures (De Lorenzis *et*

al., 2000b; Tinazzi *et al.*, 2000). Installations of FRP rods have also been undertaken for the repair of historical monuments.

The advantages of the use of the NSM rod systems lie in the fact that the rods are protected from the external environments, in that they are completely surrounded by adhesive paste. This implies that alkaline and other salts in the cements in concrete structures do not attack the paste. If the paste is not attacked, the rods will not be affected by alkaline-initiated corrosion in a concrete environment. The NSM rod systems are also useful in cases where loss of an externally bonded strengthening system could occur (e.g. over-height vehicles impacting bridge soffits, or 'bridge bashing').

Rehabilitation of steel structural members

The advanced polymer composite materials have not been utilised to upgrade metallic structures to the same extent as for reinforced concrete structures. Until recently, only a limited amount of research has been conducted on the application of these materials to metallic structures, but this situation is now changing (Luke and Canning, 2004, 2005; Mertz and Gillespie, 1996; Mosallam *et al.*, 1999; Photiou *et al.*, 2006b; Tavakkoli-zadeh and Saadatmanesh, 2003).

The early and modern steels and the cast iron metals have a relatively high bending stiffness; therefore, to upgrade these units, the CFRP composites will invariably be used. As high modulus (HM) CFRP composites have stiffnesses of the same order as that of the steels, substantial load transfer can only take place after the steel has yielded. Depending on the manufacturing heat treatment, ultrahigh modulus (UHM) CFRP composites can have stiffnesses in excess of 600 GPa. To upgrade steel beams, UHM pitch CFRP prepregs at 60% fibre volume fraction are used, with moduli of about 400 GPa. Consequently, the stiffness of this material could be twice as high as that of the steel and significant load transfer to the composite can take place before the steel has yielded. With the high stiffness moduli, the strain to failure of the UHM carbon fibres is very low (less than 0.4% strain; this value will depend on the modulus of elasticity).

There are three standard fabrication–adhesive bonding methods for upgrading metallic structural members:

- using a pultruded rigid plate and two-part cold setting epoxy adhesive
- using a pre-impregnated rigid plate and two-part cold setting epoxy adhesive
- the wet lay-up method, where the matrix material component of the composite also acts as the adhesive material of the upgrade.

A further upgrading system uses hot-melt factory-made pre-impregnated fibre with a compatible film adhesive; this is discussed earlier in this chapter. The test results of bonded double butt joint coupons (Photiou *et al.*, 2006a) using film

adhesive and the two-part cold-cure system have shown that the film adhesive fails at a higher ultimate load than the two-part system. The pre-impregnated CFRP composite with a compatible film adhesive has been used to fit CFRP composites to a historic building (Garden and Shahidi, 2002) but, to the knowledge of the author, it has yet to be used to upgrade a bridge structure.

Hollaway (2007) has reviewed the durability of some bridge and other structures that have been in a civil engineering environment for some 30 years. Furthermore, Hollaway *et al.* (2006) note that the near-zero coefficient of thermal expansion of the carbon fibre is an advantage in that there is a reduction in the internal stresses set up in the steel during the cooling period after polymerisation of the composite and film adhesive. Regardless, the effect of the differing coefficients of thermal expansion for metallic substrates and bonded unidirectional CFRP composite materials is of a similar magnitude to that of live load effects in CFRP-strengthened steel structures and causes high localised stresses at discontinuities in the adhesive bond (such as where bonded plates are curtailed). Methods to minimise such adhesive stresses include gradual tapering of the bonded plate and a gradual increase in the flexibility of the adhesive near the end of a bonded plate (by increasing adhesive thickness or using an adhesive with a lower elastic modulus).

Provided the FRP strengthening plate is bonded throughout the length of the steel beam and the high modulus CFRP composite plate is used to upgrade the beam, the failure criteria will be due to excessive deflection of the beam, provided the ultimate strain to failure of the carbon fibre is not exceeded. If, however, an ultrahigh modulus carbon fibre plate were used to stiffen or strengthen the beam, it is likely that the system would fail by the ultimate strain of the fibre being reached; the larger the carbon fibre modulus, the smaller the strain to failure of the composite and therefore of the beam.

Further information on the technique, analysis and design of the rehabilitation of FRP composites to metallic structures may be obtained from Cadei *et al.* (2004), Hill *et al.* (1999), Hollaway *et al.* (2006), Leonard (2002), Liu *et al.* (2001, 2005a), Moy (2001) and Photiou *et al.* (2006a, 2006b).

FRP confining of concrete

The external application of FRP composites to confine concrete bridge columns is an attractive and effective solution for retrofitting or for improving the axial compressive strength and ductility of existing columns. The technique is most efficient when applied to circular columns, albeit not as much as would be expected from predictions based on mechanical properties of the composite; it is much less effective, if at all, when applied to rectangular columns. Considerable research effort, coupled with field tests of FRP wraps as passive confinement to concrete columns, has been made, as has the development of models to calculate the strengthening effects. Two significant experimental observations are that: (a) the apparent average failure strains of the FRP wraps, which are 50%–80% of the

failure strains of tensile coupons made from the same material specification (Karbhari and Geo, 1997; Mirmiran *et al.*, 1998; Xiao and Wu, 2000) and (b) passive confinement of square and rectangular columns with sharp corners is less efficient than that of circular columns (Chaallal *et al.*, 2003; Karam and Tabbara, 2002; Mirmiran *et al.*, 1998; Pessiki *et al.*, 2001; Rochette and Labossière 2000; Yang *et al.*, 2001).

The FRP wrapping of concrete columns has two functions. These are (a) to cause an increase in the confined concrete peak stress compared with that of the unconfined concrete and (b) to increase the post-peak ductility and ultimate strength of the concrete column, thus developing a pseudoductile plateau, as illustrated in the axial compressive stress–axial compressive strain curve (Figure 18.19) for circular columns. The first function is caused by Poisson's lateral stresses, followed by non-linear dilation behaviour of the concrete, owing to pre-peak cracking. The second function is caused by the mechanism of FRP wraps restraining the movement of discrete concrete blocks after the localisation of concrete failure (Issa and Karam, 2004). Both of these effects are developed for circular columns. However, in rectangular columns, the first effect is negligible, as there is little or no increase in the concrete peak stress compared with the unconfined concrete, indicating that little or no confining stresses have been developed. Most of the improvement is observed post-peak, in the form of increased ductility and ultimate strength. The material around the corners and across the diagonals between opposite corners is confined to a certain extent, while the material along the sides of the flat portions of the rectangular section is confined to a minimum extent or not at all, depending on the curvature of the corners. Figure 18.20 shows the extent of confinement. There are methods of increasing the effectiveness of the FRP confinement for a rectangular column, one of which is to modify the column section into an elliptical one; this is illustrated in Figure 18.21.

Available stress–strain models for FRP-confined concrete have been reviewed and assessed using a test database built by Lam and Teng (2002), who have shown that most of the models

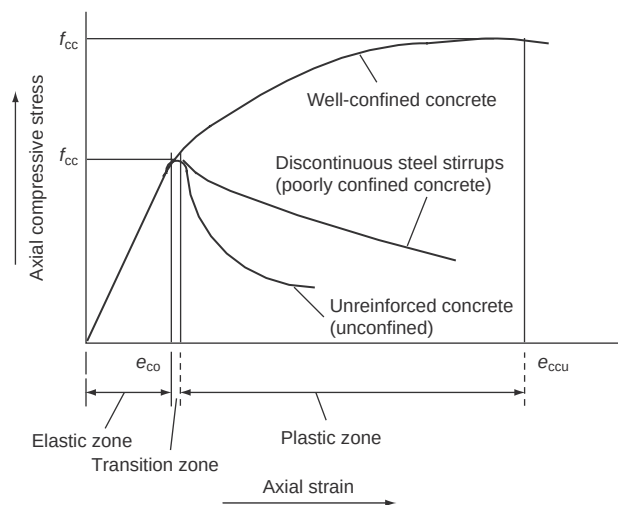


Figure 18.19 Drawing of qualitatives

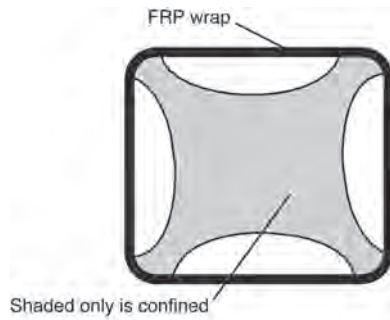


Figure 18.20 Confinement of square column

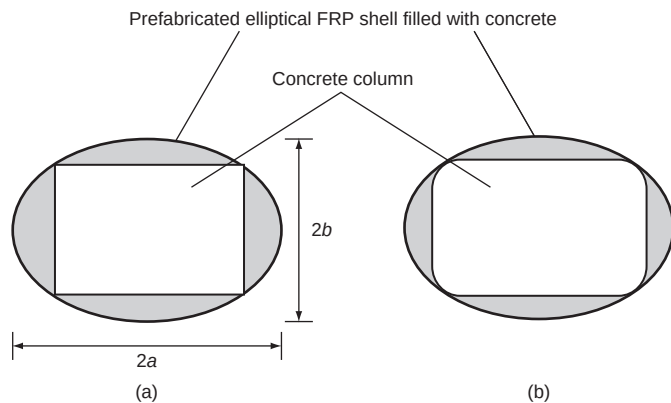


Figure 18.21 Strengthening of rectangular column by shape modification and FRP reinforcement: (a) shape modification without corner rounding; (b) shape modification with corner rounding

provide inaccurate predictions of the ultimate concrete strain or the shape of the stress–strain curve and that none of them accounts for the dependency of the ultimate concrete strain on the type of FRP. Lam and Teng (2003) have developed a model of the form

$$f'_{cc}/f'_{co} = 1 + k_1(f_1/f'_{co}) \quad (18.3)$$

where the closest experimental predictions give $k_1 = 2.15$, but this result was slightly on the unsafe side for the region of high degree of confinement. A new equation for the axial compressive strength of FRP-confined concrete

$$f'_{cc}/f'_{co} = 1 + 2(f_1/f'_{co}) \quad (18.4)$$

was therefore proposed for design use. It does, however, err on the conservative side, but Lam and Teng (2003) claim that it is simpler than all other existing models developed specifically for FRP-confined concrete and have recommended its use. The lower bound values of the ultimate concrete strain adopted are used for conservative predictions. The FRP tensile strength can be determined according to ASTM D 3039 (ASTM, 1995).

It must be emphasised that the strength and stress–strain models based on tests of circular specimens cannot be applied directly to columns with non-circular sections, such as square and rectangular sections. This requires an understanding of the behaviour of FRP confinement on elliptical columns. Teng

and Lam (2002) present and discuss the results of an experimental study on FRP-confined elliptical concrete columns and conclude that confinement becomes increasingly less effective as the section becomes more elliptical; however, substantial strength gains from confinement can still be achieved, even for highly elliptical sections. If rectangular column sections are not modified to an elliptical shape, the sharpness of the corners plays a role in the confinement effectiveness of the jacket, since stress concentrations at the corners can cause premature rupture of the FRP material. Consequently, for these column sections, the corners must be rounded. Furthermore, for square columns the GFRP composite jackets generally increase the ultimate axial stress and strain values more effectively than either the AFRP or CFRP composite jackets (Cole and Belarbi, 2001).

Chaallal *et al.* (2006) discussed and compare three design guidelines for strengthening circular concrete columns externally bonded with FRP composites. The ACI-440 committee (ACI 2002a, 2002b) provides design equations for axially loaded short circular columns. The gain in concrete strength owing to the FRP wrap depends on the passive confinement lateral pressure generated by the lateral FRP fibres. The fibres in the longitudinal direction are not considered to provide an increase in the load-carrying capacity. The axial load-carrying capacity of the strengthened column can be calculated as

$$\Phi P_n = k_c \phi [0.85 \psi_f f'_{cc} (A_g - A_{st}) + f_y A_{st}] \quad (18.5)$$

where k_c is the resistance factor ($=0.85$ for spiral reinforced columns and 0.80 for tie reinforced columns), P_n is the nominal axial load-carrying capacity, Φ is the strength reduction factor, ψ_f is the additional coefficient for FRP-wrapped columns ($=0.95$), f'_{cc} is the compressive strength of confined concrete, A_g is the cross-sectional area of the confined concrete, A_{st} is the longitudinal steel area and f_y is the steel yield strength.

The confined concrete strength is calculated after Mander *et al.* (1988) and depends on the confining pressure, that is

$$f'_{cc} = f'_c \left[2.25 \left(1 + 7.9 \left[f'_1/f'_c \right] \right)^{1/2} - 2 \left[f'_1/f'_c \right] - 1.25 \right] \quad (18.6)$$

$$f'_1 = [k_a \rho_f f_{te}] / 2 \quad (18.7)$$

where f'_c is the unconfined compressive concrete strength, f_1 is the lateral confining pressure, k_a is the efficiency coefficient ($=1.0$ for circular columns), ρ_f is the confining FRP volumetric ratio ($=4nt_f/d$), n is the number of FRP layers, d is the diameter of the circular column, f_{te} is the FRP tension strength ($=\varepsilon_{fe} E_f$), E_f is the modulus of elasticity of FRP, ε_{fe} is the FRP effective strain ($=\min(0.004, 0.75\varepsilon_{fu})$) and ε_{fu} is the ultimate FRP strain.

According to the Canadian Standards Association (CSA, 2002), the maximum load-carrying capacity of confined column is given by

$$P_{rmax} = k_c [\alpha_1 \phi_c f'_{cc} (A_g - A_{st}) + \phi_s f_y A_{st}] \quad (18.8)$$

where k_c is the resistance factor ($=0.85$ for columns with spiral transverse steel), ϕ_c and ϕ_s are the resistance factors for concrete and steel, respectively ($\phi_c = 0.6$ and $\phi_s = 0.85$), α_1 is the ratio of average compression stress to the concrete strength and $\alpha_1 = 0.85 - 0.0015f'_c \geq 0.67$.

The confined concrete strength f'_{cc} can be calculated as

$$f'_{cc} = 0.85f'_c + k_1 k_c f_i \quad (18.9)$$

$$k_1 = 6.7(k_c k_i)^{-0.17} \quad (18.10)$$

$$f_i = 2 t_j f_{ij} / D \quad (18.11)$$

where k_c is the confinement coefficient ($=1$ for circular columns), f_j is the lateral confinement pressure and t_j is the thickness of the FRP jacket.

The design guidelines provided by the Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures (ISIS, 2001) give the confined concrete strength as

$$F'_{cc} = f'_c (1 + \alpha_{pc} w_w) \quad (18.12)$$

$$w_w = 2(f_{iFRP} / \varphi_c f'_c) \quad (18.13)$$

$$f_{iFRP} = (2 N_b \varphi_{FRP} f_{FRPu} t_{FRP}) / D \quad (18.14)$$

where α_{pc} is the performance coefficient ($=1$ for circular columns), w is the volumetric strength ratio, f_{FRP} is the lateral confinement pressure, φ_c is the concrete resistance reduction factor, t_{FRP} is the thickness of one FRP layer, N_b is the number of FRP jacket layers, D is the diameter of the circular column, φ_{FRP} is the FRP resistance reduction factor and f_{FRPu} is the ultimate FRP tensile strength.

To ensure a certain amount of ductility and therefore provide an effective confinement, the ISIS guidelines impose a minimum confining pressure equal to 4 MPa, i.e.

$$F_{FRP} > 4 \text{ MPa} \quad (18.15)$$

In addition, a maximum confining pressure is prescribed to limit the axial compressive strains, that is

$$F_{FRP} \leq (0.29 f'_c) / \alpha_{pc} \quad (18.16)$$

Teng *et al.* (2009) has developed a model based on both FRP confinement strength and FRP confinement stiffness. This work has been used as part of the proposals and guidance in the Concrete Society's Technical Report 55 (Darby *et al.*, 2012), where further advice on design approaches can be found.

Fibre-reinforced polymer composite materials can also be used in the rehabilitation of bridge piers – a common application being in marine conditions, where reinforced concrete, steel, timber and cast iron can all experience severe deterioration. In these applications, the FRP composite is typically preformed as two halves of a cylinder to encapsulate the bridge pier, and the intervening annulus or gap filled with grout. In this form, the FRP material provides additional protection to the pier in a severe environment as well as additional strength, depending on the design philosophy. This technology was recently used at Burnett River rail bridge near Bundaberg, Australia, where investigations revealed deterioration to the cast iron piers (including graphitisation and some cracking). To provide protection to the cast iron piers, and to improve the flexural capacity against train braking effects in particular, conical and cylindrical GFRP cylinders were used to encapsulate and strengthen the piers, with epoxy grout placed within the annulus. Further information can be found in Kandadai *et al.* (2014).

FRP composite rebars used as internal reinforcement to concrete

Steel reinforcement has been widely used for conventional concrete structures. Generally, it is chemically protected by the high alkalinity (pH 12.5–13.5) of the concrete and physically protected by surrounding concrete cover against corrosion. However, for many structures exposed to aggressive environments (such as concrete bridges in a marine environment) or where concrete cracking is excessive, combinations of moisture, temperature and chlorides reduce the alkalinity of the concrete and result in the corrosion of steel reinforcement. This leads to concrete deterioration, followed by the eventual loss of structural serviceability. To overcome corrosion problems, a number of options to prevent corrosion agents from reaching the surface of the steel reinforcement have been investigated; these include (a) using epoxy coating, (b) providing cathodic protection to the steel reinforcement, (c) using sealers and membranes, (d) using a lower-permeability concrete and (e) using corrosion-inhibiting chemical admixtures. However, the potential of corrosion problems still remains with the constant presence of corrosive agents; therefore, the effectiveness of these options may vary considerably during the lifespan of the structure. Fibre-reinforced polymer rebar composites have become an alternative or competitor to reinforcing steel in certain applications.

FRP rebars, which have been produced for reinforcing concrete for over 30 years (Nanni, 1993b, ACI, 1996), have advantages and disadvantages in their material characteristics. These are:

■ Advantages

- High longitudinal strength.
- Corrosion resistance and electromagnetic neutrality.
- High fatigue endurance, particularly carbon and aramid fibre polymer composite.
- Light weight.
- Higher ratio of strength to self-weight (10–15 times greater than steel).
- Low axial coefficient of thermal conductivity, and electric conductivity.

■ Disadvantages

- No yielding before brittle rupture (lack of ductility).
- Low transverse strength.
- If manufactured by the pultrusion method, bars cannot be shaped into hooks or angled for end anchorage on-site but special bar shapes can be manufactured in a factory.
- Low durability of glass FRP composites in a moist environment. This is unlikely to take place when the rebar is placed in concrete with constant wetting and drying in the natural

environment but could be a problem if the concrete structure is under water. Carbon FRP composites do not suffer from low durability in this environment, and basalt FRP also shows better performance than glass FRP in this respect.

- Susceptibility to fire, depending on matrix type and concrete cover.
- As rebars are generally manufactured using a thermosetting resin, they cannot be reshaped once polymerised. It is therefore not possible to reform, shape or bend bars on-site.

Among some civil engineers there is a perception that some glass fibres exhibit low durability in an alkaline environment. This has occurred as a result of the interpretation of laboratory accelerated testing of polymer composites being incorrectly extrapolated to represent actual long-term site estimates. Field tests on the durability of structural concrete members reinforced with FRP rebars over decades have provided evidence that little or no chemical attack on the FRP rebars takes place.

There are several commercially available FRP rebars made from continuous aramid (AFRP), carbon (CFRP), basalt (BFRP) or glass (GFRP) fibres embedded in various thermosetting resin materials. These are generally manufactured by the pultrusion process and can be characterised by the type of surface finish of the rebar; the finish is to improve the bond characteristics between the concrete and the rebar. Several techniques are used; these are some of the more common ones.

- Ribbed bars are manufactured by a 'hybrid-pultrusion' process, a combination of pultrusion and compression moulding.
- Sand-blasted bars are manufactured by the pultrusion process and then sand-blasted to enhance the bond characteristics.
- Spirally wound and sand coated bars are manufactured by the pultrusion process; the surfaces of the bars are then spirally wound with a fibre tow and sand covered.

Rebars can be manufactured in various cross-sections; the circular cross-section is the most common, but square and rectangular sections can be produced.

The various types on the market worldwide have been described by Pilakoutas (2000) and more recently by Emparanza *et al.*, (2017).

As has been pointed out in the list of disadvantages, once polymerised, pultruded rebars cannot be shaped or reformed by the application of heat. Consequently, it is not possible to bend the bars on-site. There are thermoplastic polymer rebars that can be bent to 90° and 180° on-site by the application of heat, thus forming, in conjunction with the thermosetting rods, greater bond resistance at the free ends of the pultruded rebars. This fabrication will only be efficient if there is sufficient bond length between the thermosetting rods, the thermoplastic rebars and the concrete components. If it is necessary for thermosetting FRP rebars to be bent, they must be specially made at the manufacturer's plant. Bends can be produced to give any shape that can be obtained with steel rebar, although a more generous bend diameter will typically be required. Bank (2006) has given equations for calculating the bend radius

for FRP rebars. In any project where bending dimensions are critical, the supplier should be contacted to verify bending capability. The inside bend radii of the FRP stirrups tend to be greater than those of the steel stirrup rebars; consequently, the main (longitudinal) rebar nearest the free surface of the concrete is positioned farther from that surface of the beam than would be the case with a steel stirrup; consequently, the standard tables giving the number of longitudinal steel rebars permitted in a given width should be used with caution (Bank, 2006). Based on results from tests, approximately 50%–60% of the guaranteed design strength of a straight bar is retained after making a 90° bend.

The bond strength between the FRP rebar and the surrounding concrete will determine the transfer of forces. It might not always be possible to transfer all of the tensile force in the bar to the concrete for a given embedment length, as failure of the system might result before this happens. As shown by tests undertaken by Wambeke and Shield (2006), failure could result from

- splitting of the concrete surrounding the rebar
- pull-out of the rebar from the concrete.

Bank (2006) discusses various aspects of the design of FRP rebars, and has related these discussions to ACI codes. These design aspects include

- the maximum effective stress achievable in an FRP rebar, based on bond failure
- the development length of straight FRP rebars
- the development length of hooked FRP rebars
- the lap splices for FRP bars
- the design procedure to detail FRP rebars in a beam.

The FRP rebar is an anisotropic material; consequently, the mechanical properties are considerably more difficult to obtain than those of the isotropic steel rebar. The strength and stiffness of FRP rebars depend on the following.

- The method of manufacture, including the curing process and quality control.
- The fibre volume fraction (f.v.f.): the rebars are primarily longitudinally reinforced with f.v.f. values in the range 50%–65%.
- The size of the rebar. It has been reported by Faza and Ganga Rao (1993) that, owing to shear lag effects, there is a non-uniform loading across the section of the rebar. The larger the cross-sectional area, the greater will be this effect; indeed, for diameter increases of 9.5–20 mm, reductions in strength up to 40% have been recorded. Therefore, it is advisable to determine, experimentally, the tensile strength and stiffness of every type and diameter of rebar to be used; invariably, these will be given by the manufacturer of the bars. It is noted that a strength reduction of 40% still leaves a very high residual tensile strength.
- The type of fibre used (glass, aramid, basalt or carbon – or, in some cases, a hybrid of two fibres).

The advantages of the utilisation of FRP rebars lie in the fact that they do not corrode and are not susceptible to chloride- or carbonation-initiated corrosion in a concrete environment. This has been discussed earlier in this chapter. The performance of the composite system, as a whole, should be the primary consideration when operating in any hostile environment. Composites have been shown to exhibit superior performance and durability characteristics, in many hostile environments, to those of the more conventional materials. However, as discussed previously, the effects of an alkaline environment, which may cause degradation to the main constituents of the FRP composite, should be considered carefully. The concrete pore solution is a potential durability threat to FRP reinforcement. Glass fibres, in particular, could be susceptible to the high alkaline environment in concrete, in which the pore solution has a pH of 12.5–13.5. If the degradation of the rebar is of concern, it is possible to protect the fibres with a suitable thickness of appropriate resin to prevent rapid degradation. However, no such degradation was observed in field tests undertaken by ISIS Canada (see later). Saint Gobain Vetrotex has introduced Arcotex glass fibre, which exhibits good acid and alkaline resistance and has good stress-corrosion resistance; this material is suitable for reinforcing thermosetting resins to form rebars.

Although higher in cost than GFRP rebar composites, carbon, basalt and aramid fibre composites are considered to be inert to alkaline environment degradation and can be used in the most extreme cases. However, an appropriate resin must be selected to ensure good overall composite system performance, in order to ensure that alkali ingress into the polymer does not happen. The recent development of basalt reinforcement bars is particularly relevant, as they perform better in alkaline environments, with a marginal increase in cost compared with glass fibre, but minimal difference when considered in the final cost of a completed structure.

In 2004, in conjunction with ISIS Canada, Mufti *et al* (2005a, 2005b, 2005c) conducted field trials on five GFRP reinforced concrete structures located across Canada, from east to west, to obtain information on their reliability when exposed to civil engineering limits of temperature, moisture and salt solutions, in a wide coverage of varying natural environmental conditions. In these trials, core specimens of the GFRP reinforcement were removed from the structures. Examinations demonstrated that GFRP flexural tension reinforcement is durable and compatible with concrete; contrary to a belief commonly held by the engineering community, the test results for these structures revealed that the alkali in concrete bridge decks does not have any detrimental effect on GFRP composite. Monitoring of the reinforced concrete (FRP rebars) is continuing and it is likely that the performance will be as good over the longer term. Mufti *et al* (2005a, 2005b, 2005c) stated that, owing to the alkaline environment in the concrete, a possible degradation mechanism of the resin is alkali hydrolysis of the ester bonds in the structure of the polymer and thought it possible that some alkali hydrolysis might have taken place. However, infrared spectroscopy showed that almost no change in the spectra of

the specimens occurred and therefore no significant hydrolysis took place. In addition, energy dispersive X-ray (EDX) analysis indicated that no alkali ingress in the GFRP reinforcement had taken place. Furthermore, as the reduction in pH values of concrete pore solution reduces over the years, this form of attack is likely to reduce in magnitude. Further research and testing on GFRP and other FRP reinforcement in the alkaline concrete environment has been undertaken (e.g. Al Rifai *et al.*, 2020; Benmokrane and Ali, 2016; Wang *et al.*, 2017).

Design procedures are available in the appendix to this chapter; these include ACI 440, CSA-S806-02 and fib Bulletin 40.

Intelligent structures

Smart structures and materials have emerged during the past few years as one of the important technologies for the twenty-first century. The ideas are simple, although the technologies for obtaining the intelligent structure can be complicated. At the structural level, an integrated sensor system provides data on the structural loading and on the environment in which the structure is situated to a processing and control system, which incorporates signal integrated actuators to modify the properties of the structure in an appropriate way. Such systems can offer immense benefits to bridge engineering. The sensing and response functions are built into the material itself, possibly using a chemical or morphological structure to provide the response. A number of different disciplines must be involved in achieving a high level of sophistication in the art of intelligent structures before any meaningful activities in smart structures and materials can take place. Included in these disciplines are three of particular importance: material systems, adaptive control systems and artificial intelligence systems. The use and creation of materials has been an important human activity throughout history, while the use of adaptive control systems has only become of significance since the beginning of the Industrial Revolution, and the artificial intelligence depends on the development and availability of a computer.

A smart material can ‘sense’ changes in the environment and make a response by changing either its material properties or geometry or its mechanical or electromagnetic response. Both the sensor and actuator functions, which comprise the ‘brain’ of the material, must be integrated with the appropriate feedback. Piezoelectric ceramics have proved to be effective, both as sensors and actuators, for a wide variety of applications. Such materials can respond by either changing the stress or strain fields to a desirable value (active noise and vibration control, for example) or changing its surface stress or strain distribution, such as the external field, which itself could be a sensing signal.

The development of materials with built-in optical sensing systems constitutes a necessary phase in the evolution of smart structure technology. Structures from such materials could continuously monitor their internal strains, vibration, temperature and structural integrity. In the case of advanced composite materials, this intrinsic sensing system might also be capable of improving quality control during fabrication.

This clearly has both safety and economic aspects for the system and could lead to greater confidence in the use of advanced composite materials and material savings through the avoidance of overdesign.

Optical fibre methods, which have been directed towards the development of smart aerospace and hydrospace vehicle material evaluation and control during the past 15 years, could be applied, after modification, in the evaluation of some civil engineering structures. The advantages of optical fibre techniques for civil applications include the general robustness of the optical fibre and cable material under harsh environmental conditions and the general geometry of the fibre sensing systems, which allows a number of sensor locations to be placed along a single linear fibre of extended length. These advantages are particularly attractive for the instrumentation of civil engineering structures, which are exposed to external environmental effects over practical lifetimes of 50 to 100 years. A method of monitoring strains is to use a fibre optic differential interferometric measuring system. Single-mode fibres embedded in a material can be used to detect both strain and temperature fluctuations, although fluctuations in non-laboratory environments could mask the resulting temperature-induced strain. A range of measurement systems based on optical fibres is available.

Fibre optic sensors make ideal sensing systems for composite materials, as they are compatible with them and are extremely small and lightweight, resistant to corrosion and fatigue and immune to electrical interference. With the increasing use of composites in bridge engineering, the development of smart composites will accelerate this trend, as it is difficult to incorporate the same capabilities in other construction materials. Although major advances have been made in recent decades in all the enabling technologies associated with smart structures, the technical and commercial challenges remain formidable. In civil engineering, the problem associated with manufacturing is that the sensors are embedded in the material and must be able to resist damage during manufacture and fabrication. Particular attention must be paid to the sensor choice, fibre coating and movement or damage to the device during manufacture. In general, the choice of smart structure 'system' is extremely critical and work is required to help the designer and fibre optic engineer to select the most appropriate materials for a particular fabrication route and application.

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Appendix. Design codes, codes and specifications for the design of FRP composites in structural engineering with reference to bridge engineering

In recent years, a significant number of design codes and specifications have been published by technical organisations to provide guidance for design with FRP materials for civil engineering. The key publications are listed here.

UK

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Europe

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USA

FRP reinforcing rebars and tendons

- ACI (American Concrete Institute) (1996) 440R-96. State-of-the-art

- report on fiber reinforced plastic (FRP) reinforcement for concrete structures. ACI, Farmington Hills, MI, USA.
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- ACI (2002b) 440.2R-02. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures. ACI, Farmington Hills, MI, USA.
- ACI (2004a) ACI 440.4R-04. Prestressing concrete structures with FRP tendons. ACI, Farmington Hills, MI, USA.
- ACI (2004b) 440.3R-04. Guide test methods for fibre-reinforced polymers (FRP) for reinforcing or strengthening concrete structures. ACI, Farmington Hills, MI, USA.
- ACI (2006) 440.1R-06. Guide for the design and construction of structural concrete reinforced with FRP bars. ACI, Farmington Hills, MI, USA.
- ASTM (1995) D 3039-M95a. Standard test for tensile properties of polymer matrix composite materials. ASTM International, West Conshohocken, PA, USA.
- ASTM (2017) D7957. Standard specification for solid round glass fiber reinforced polymer bars for concrete reinforcement. ASTM International, West Conshohocken, PA, USA.

FRP strengthening systems

- ACI (2002b) 440.2R-02. Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures. ACI, Farmington Hills, MI, USA.
- ICC Evaluation Service (1997) AC125. Acceptance criteria for concrete and reinforced and unreinforced masonry strengthening using fibre reinforced polymer composite systems. ICC Evaluation Service, Whittier, CA, USA.
- ICC Evaluation Service (2001) AC187. Acceptance criteria for inspection and verification of concrete and reinforced and unreinforced masonry strengthening using fibre reinforced polymer composite systems. ICC Evaluation Service, Whittier, CA, USA.

Canada

- CSA (Canadian Standards Association) (2000) CSA-06-00. Canadian highway bridge design code. Canadian Standards Association, Toronto, ON, Canada.
- CSA (2002) S806-02. Design and construction of building components with fiber-reinforced polymers. Canadian Standards Association, Toronto, ON, Canada.
- ISIS Canada (2001) Design Manual No 3. Reinforcing concrete structures with fiber reinforced polymers. Canadian Network of Centers of Excellence on Intelligent Sensing for Innovative Structures, Winnipeg, MB, Canada.

Japan

- BRI (Building Research Institute) (1995) *Guidelines for Structural Design of FRP Reinforced Concrete Building Structures*. Building Research Institute, Tsukuba, Japan.
- Japan Society of Civil Engineers (JSCE) (2001) *Recommendations for Upgrading of Concrete Structures with Use of Continuous Fibre Sheets*. JSCE, Tokyo, Japan, Concrete Engineering Series 41.
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Chapter 19

Temporary works for bridges

J Tod Tony Gee, LLP

This chapter discusses how site location can influence the development and design stages, some of the practical considerations of off-site construction, aspects of temporary works required for sub-structure and superstructure construction, case studies, the effects of successive construction stages on the permanent works and dismantling, decommissioning and demolition. It provides a background to the problems faced by contractors in temporary works, looks at safety in construction and the procurement stages when it can be considered. Finally, it provides a signpost to the many other sources of information that can help the designer on the subject of temporary works.

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Introduction

The intent of the *ICE Manual of Bridge Engineering* is that it will give users sufficient information about all aspects of bridge design for them to have a firm grounding in the engineering principles and allow them to develop a design through informed decision-making. As well as considering the technical complexities of bridge design, it is appropriate to look at the practical challenges of bridge construction in this chapter.

It is now easier to create bridges than ever before. Engineers have an unprecedented advantage over their predecessors: materials are better understood; we have computer-aided design and three-dimensional (3D) modelling software; fabrication facilities, transportation and means of placing components of greater size allow bridges to be built with less disruption on-site; clients have 3D, 4D and 5D model visualisations of their projects; vast amounts of detailed information are shared electronically between all involved with the minimum delay; site safety is improving year on year.

The aim of this chapter is to promote good safety and constructability practices in the execution of bridge construction: the practical complexities of constructing a bridge, safety of the workforce and project cost are affected by the choices made during the design

Constructability and temporary works

Constructability has been defined as ‘the optimum integration of construction knowledge and experience in planning, engineering, procurement and field operations to achieve overall project objectives’ (CII, 1986).

The aim of good constructability is to provide a safe construction method that best meets the requirements of the project, whether this be speed of construction, limiting costs or the use of locally available resources.

Temporary works are a by-product of constructability. Building a structure that cannot support itself requires temporary works to maintain it in position and prevent it from becoming overstressed until the permanent works have gained sufficient strength and structural integrity. After the permanent works are self-supporting, the temporary works can be removed.

Over history, constructability and temporary works design have developed in tandem with bridge design so that structures that only existed as concepts on a drawing board could be brought into being (see Chapter 1).

It might be thought that the safest constructability would be a structure that was self-supporting during all stages of construction – one that required no temporary works. However, as this structure would be designed for the stresses and deflections caused in the temporary state, it is likely that it would have a high degree of redundancy in the permanent condition. The material and work content would make it an inefficient and expensive option.

However, where the permanent structure can be made self-supporting, or where it can be used to provide support to follow-on operations, this should be encouraged. Designing out temporary works will reduce the work content on-site and, by reducing the exposure of the workforce to risks, will make the work safer.

As a topic, the range and complexity of temporary works is so vast that it is beyond the scope of a single chapter in a book.

To give a bridge engineer a basic understanding of temporary works and constructability to help in producing an informed

design, in the limited space available here, four aspects of construction are given consideration:

- the site – aspects specific to the site location – geography, access, availability and cost of materials, workforce and plant, the possibility to fabricate off-site – site establishment – the haul road, temporary bridges, load-bearing platforms and crane foundations needed to carry out the work (as well as hoardings, offices and welfare facilities)
- off-site construction – reasons to consider off-site construction, temporary works and handling requirements
- substructure construction – the temporary works needed to construct the foundations, abutments and piers
- superstructure construction – the temporary works needed to build the bridge deck.

It should be remembered that all aspects of site construction stem from the design of the finished bridge; therefore, there are a great number of opportunities to make good decisions.

The site

Where is the bridge? Geography

The function of the bridge is to provide a crossing of a physical obstruction at a specific location. So the first consideration of temporary works and constructability is geography.



Figure 19.1 Bridge erection in Glasgow, Scotland, latitude 55.858°N; temperatures reached –18°C during the construction of the M74 extension (courtesy of Tony Gee)

The location of the construction site is a fundamental consideration to constructability. It will affect the source of materials and labour, methods of construction and temporary works.

The latitude of the site (Figures 19.1 and 19.2) will affect seasons, hours of daylight and temperature.



Figure 19.2 Bridge erection in Kuwait, latitude 29.321°N; Jamal Abdul Nasser Street Bridge (courtesy of Tony Gee)

The position of the site on the continental land mass will affect weather pattern, temperature patterns and seasonal extremes (Table 19.1).

Wind, rain, monsoons hurricanes, typhoons, flooding, freezing temperatures, cyclical freezing and thawing and extreme heat all affect productivity of the labour force, the construction methodology and the loading requirements on the temporary works.

Short windows between extreme seasonal occurrences will affect construction methodology. These windows can also be related to ecological events, such as bird nesting and fish spawning seasons.

The local geography will dictate physical constraints, such as the range of tides, the extent of the intertidal zone and the presence of mountain ranges, deserts or swamplands, as well as availability of materials, the source of local labour, how far labourers have to travel and the access routes and methods of transport.

The political geography will determine the presence of man-made borders to transportation of plant, labour and materials.

	Example	Typified by	Summer daylight	Summer temperature	Winter daylight	Winter temperature	Snowfall	Rainfall
Extreme north or south	Alaska, Canada, Russia, Scandinavia	Cold, extreme changes in daylight	Almost constant for 4 months	10°C	None for 5 months	–20°C	For 8 months	For 4 months
Tropical	SE Asia, India	Monsoon, hurricane, tempests	Almost equal to winter	22°C	Almost equal to summer	18°C	Very little	Monsoon for 8 months
Desert	Russia, China, Africa, North America, Australia	Temperature, Aridity	Depends on latitude	Depends on latitude	Depends on latitude	Depends on latitude	Very little	250 mm per year

Table 19.1 Differences between physical constraints in some geographical regions

Access to site

Plant, labour and materials must be delivered to the site and unwanted material disposed of to enable the bridge to be constructed. This will require some form of access.

The function of the bridge under construction will be to provide a crossing over an obstruction for a road, railway, canal, pipeline or services. The location of the bridge will be either on a new or existing route.

If the bridge is on a new route, the easiest way to get access to the site may be to build the new road, railway or waterway up to the bridge site and use the new route to get the materials, plant and labour to the site.

If this is not possible, designed temporary roads, earthworks, drainage and bridges may be required, just to gain access to the site from the existing communication network. The new site access will have to be tied into existing access routes, which may require new road junctions, rail junctions, sidings or harbours.

For new bridges on an existing route, it may be possible to use this for delivery of construction plant, labour and materials. The existing route will have restrictions and obstructions, which will limit access. To assess the access route and determine its suitability, the following must be known.

- Components or plant being delivered to site:
 - ☐ weight
 - ☐ dimensions
 - ☐ number of vehicles.
- Restrictions on the existing route:
 - ☐ width or height restrictions under or through bridges or tunnels or between buildings
 - ☐ length restrictions – tight bends, street furniture
 - ☐ gradient of road route
 - ☐ weight restrictions – bridges, roads
 - ☐ whether permits are required to use the route and how long they will take to obtain.
- Is the route affected by tides? This will affect the timing of operations.
- Is the route in the intertidal zone? This significantly restricts the access by both land-based and floating plant.
- How exposed is the route to weather? Will it flood? Become blocked by snow?
- The resilience of the route. Will it deteriorate rapidly under adverse weather? Will it take the traffic?
- Speed: can the route accommodate the number of vehicles in the required duration?

If these limitations are likely to affect construction productivity, temporary works may be required to improve the route.

As a minimum, you need access routes to each of the bridge's support locations. For longer bridges, this can result in major temporary works, such as access jetties or long haul roads.

The largest of bridges over navigable waterways may use dry docks for the preassembly of components, floating plant to transport the components to site and floating cranes to place the components.

Site establishment

Site clearance, site establishment and bridge construction may require the use of construction plant, such as excavators, dumpers, compaction equipment, dewatering plant, piling rigs, mobile, crawler/tower or derrick cranes and self-propelled modular transporters (SPMTs).

On larger projects, the works compound may include aggregate crushing plants, concrete batching plants, reinforcement bending shops, precast yards (Figure 19.3) and site steelwork fabrication yards. All of these will require temporary works, such as fences, cabins, foundations, storage bins and temporary workshops.



Figure 19.3 Site storage for precast bridge segments (courtesy of Tony Gee)



Figure 19.4 Ordsall Chord Bridge installation (Skanska BAM): a substantial piled reinforced concrete slab was required to provide an adequate working platform for the operation of a 1350t crawler crane, with operational track bearing pressures of 600 kPa (courtesy of Tony Gee)

Working platforms and crane foundations

Plant that exerts high bearing pressures, such as piling rigs, SPMTs and cranes (Figure 19.4), will require purpose-designed foundations. Tower crane foundations may be substantial piled structures with a reinforced concrete pile cap.

Use of plant and cranes

The type and size of plant needed to place the components will depend on the size, weight and sequence of the components to be placed. The plant may be so large that it requires other plant to erect it and subsequently dismantle it – this is typical of tower cranes, crawler cranes and larger mobile cranes.

All construction operations are affected by the weather, and plant operation can be restricted under certain conditions, especially moderate to high winds.

The operation of construction plant may be restricted by the users of adjacent sites. In the UK, Network Rail place restrictions on the operation of piling plant and cranes that could collapse onto railways (CPA, 2014, 2018; NR, 2010). Throughout the world, there are restrictions on the height of cranes and construction plant near flightpaths and airports. There may be restrictions on operating near and oversailing specific types of industrial plant. In existing developed areas, there may be restrictions on plant usage, owing to the proximity of overhead cables and electricity distribution power lines.

Under certain circumstances, a large proportion of the bridge superstructure, and even parts of the substructure, may be built offline and then moved into position, by floating, launching, sliding or transporting with SPMTs. In this case, the offline site will need to be prepared for the bridge assembly.

Off-site construction

Reasons to consider off-site construction

Structures have commonly been built using a range of components constructed remotely and delivered to the site (standardised components, such as fabricated steelwork and precast beams, for example). The increase in the availability of large mobile cranes, heavy haulage and mobile transporters has led to an increase in the size and weight of the component that can be placed. The increase in capacity and longer reach of craneage has made more of the construction site accessible.

There are a number of reasons why off-site construction may be preferable to building in situ:

- to enhance safety, for example, to avoid building at height or over water
- a potential reduction in the amount of temporary works required on-site

- the absence of foundations and available working space beneath the structure, for example, in rivers, over valleys, on poor ground or above roads, railways, sewers and tunnels
- potential disruption to adjacent sites, for example, when working near or over railways
- an absence of or unsuitable working space, for example, at a congested city-centre site
- a limited period of access to the site, for example, rail possession, tidal working
- the presence of obstructions, for example, something already on-site, especially if an existing structure is being replaced
- the potential for an improvement in programme delivery, for example, working on several different parts of the project concurrently
- protection from the environment during fabrication
- a potential reduction in production costs.

The elements to be constructed off-site need to be designed for transportation and handling.

Design of elements for fabrication and handling

The construction of an element remotely from the site does not necessarily remove the need for temporary works for its fabrication, instead shifting the works to the factory and changing their nature. Any component that is prefabricated will need to be lifted from its fabrication position onto transport, delivered to the site and lifted off the transport. It may need to be turned and then it must be temporarily supported in position while it is incorporated in the permanent works. During the latter, both the element and the supporting permanent works may be subjected to construction loads from the eccentricity of the element's centre of gravity, hydrostatic concrete pressures and environmental loads, such as wind and waves.

The component should be designed for all these effects. It is suggested that this is done by a suitably experienced and qualified designer under the control of the lead permanent works designer; in the UK, under the control of the principal designer.

The designer should consider the following points.

- What is the size, weight and position of the centre of gravity (CoG)?
 - The point of application of the lift must pass through the CoG, or movement will take place; the load will slew, twist or rotate.
 - Do precast concrete elements have protruding reinforcement?
- Does the load have to be tilted?
- What is the structural and material form of the load to be moved?
 - What is its strength and rigidity?
 - If considering prefabricating or lifting reinforcement cages, remember that reinforcement ties are poor structural connections.

- If the element is made from concrete, what strength should have been achieved before handling?
- How is it supported before moving?
- What is the relative movement between supports?
- Where are the attachment points?
- What is the maximum reaction at each attachment point?
- Will load be transmitted through two, three or four points?
 - Can it rock on just two supports? (For example, when considering four lifting points theoretically equidistant from the centre of gravity, owing to minor variations in setting out, all the load could be taken by only two lifting points.)
- How will the load be supported after placement?
 - Will it be supported immediately, without the need for grout to set?
 - The load must be lined and levelled accurately. Does the detailing allow for this?
 - The temporary supports, and any permanent works they are attached to, must have adequate strength and stability for all ongoing environmental and construction loads, for example, accidental impact, wind and hydrostatic concrete loads, until incorporated in the permanent works.
 - Can it be adequately supported about its CoG? Is there any novelty in the design of the permanent works that makes the temporary support particularly difficult (inclined members, cantilever members).
- How is the member required to connect with the existing structure?
 - Will protruding reinforcement clash with existing reinforcement?

Handling prefabricated elements

The designer of the system to transport, place and temporarily support the element should consider all of the factors listed previously (Figure 19.5). In addition, it is necessary to take account of the following points.

- What size of vehicle is required?
 - dimensions, including protrusions
 - weight.
- What are the direction, distance and height to be moved? Are there changes in the direction of travel and slope or fall?
- What are the site conditions?
 - What are the ground conditions, topography and foundation requirements?
 - Are there constraints from adjacent sites, such as headroom near airports, over railways or near nuclear facilities?
 - What is the route of transportation: road, rail, river or sea? What limitations does the route impose?
 - On-site assembly of subassemblies constructed off-site may be required.



Figure 19.5 Temporary works doubling as (a) lifting beam and (b) temporary support during repairs to the fascia of Westminster Bridge (courtesy of Tony Gee)

- How many items need to be moved; for example, is it a one-off move or are several items to be moved?
- Is speed critical (both programme duration and velocity)?
- Are there environmental constraints, for example, weather, tide and season?
- What provides the fail-safe in case of malfunction?
- Is there any novelty in the design of the permanent works that makes its temporary support particularly difficult?

Substructure construction

(See also Chapter 7.)

The substructure

The substructure must provide the bridge with sufficient bearing and anchorage to prevent it from moving during the temporary stages and once completed. The substructure will comprise abutments and temporary and permanent piers and towers.

The substructure must be founded on a suitable stratum. This will be achieved by excavation until the correct stratum is found, installing piles or a combination of both.

Excavations

Excavations may be formed in open cuts or supported by cofferdams or caissons (both are watertight temporary retaining structures; cofferdams are removed after use, while caissons are left in place). The solution that best suits a particular site will depend on the functional requirements and the site conditions.

The design of the excavation must account for the strength or weakness of the soil, the presence of water, fluctuations in water owing to tidal effects, waves, natural obstructions, voids, contaminated ground or buried structures and loading of the temporary works owing to the presence of adjacent existing structures or land usage (railways or roads, or another bridge). Dewatering and the removal of water from within the excavations may be required. Access will be required into the excavation for plant and materials to construct the substructure.

If the substructure is founded within a river or the sea, a caisson or cofferdam will be required (see Case study 1 in this chapter). The depth and plan area of the caisson or cofferdam may be considerable. A causeway or access bridge may be needed to link the caisson or cofferdam to the shore, with a ramp down into the caisson, or suitable craneage to lift materials into the excavation and remove arisings. There are many options for the form of the caisson or cofferdam.

Once the excavation for the substructure foundation has been created, the substructure can be built using the designed materials of stone, wood, brick, steel or concrete. The method of construction will depend on the materials, the dimensions of the substructure and the number of elements to be used. Some forms of repeated construction will lend themselves to modularisation.

It may be possible to incorporate temporary foundations, for the plant needed to construct the substructure and superstructure, into the bridge foundations and substructure.

Substructures constructed from reinforced soils

Where appropriate, a designer may specify bridge abutments constructed from reinforced earth, comprising a structural anchor buried in backfill and tied to a facing panel. Little in the way of temporary works is required for this type of construction, other than edge protection to safeguard the workforce and temporary support to precast kerb or edge units, which may not be stable until incorporated into the permanent works.

Substructures constructed from concrete

For substructures constructed from reinforced concrete, a number of temporary states must be considered:

- access to carry out the operations
- craneage or plant to lift materials
- the position of the construction joints
- support to the steel reinforcement
- support to the wet concrete.

Access to carry out the operations

Access may be provided by scaffolds (Figure 19.6). Scaffolds are temporary structures, usually comprising vertical members connected by ledgers, transoms and bracing members. These are used to support a platform from which operatives can carry out work. The materials used for the structural members can be



Figure 19.6 Bridge abutment constructed with access from traditional scaffolding system (courtesy of Norwest Holst)

steel, aluminium, wood, bamboo or composite. The platforms can be wood, steel, aluminium or composite. The structural members can be connected by fittings built into the members, couplers or lashings (ropes or twine). While the access scaffold is being constructed, no other work can be carried out in the same space (so programme durations must include the time for erection) and there are costs for erection and hire.

The scaffold structure will require foundations. Most commonly, these are founded at ground level or on the foundation slab. As the level at which the work is being carried out rises, the height of the scaffold is increased to keep pace, so the volume (and hire cost) of the scaffold increases. To maintain stability, the scaffold is usually tied to the permanent structure.

To avoid the need for foundations, it is possible to have climbing scaffold structures that are supported by the permanent works. The scaffold can be supported from cantilever frames, which protrude from the sides of the structure.

Access to concrete piers may be supported from frames, which are in turn supported by fittings into the top of the permanent works structure (Figures 19.7 and 19.8). Jump

forms or slip forms are hung from frames supported from vertically protruding tubes or bars cast into the walls. They can be propelled by hydraulic jacks, so are self-climbing. This has the advantage that, apart from the initial set-up and final dismantling, they do not require craneage to raise them (although materials still have to be fed to the working platform).

For concrete structures of limited height (up to around 10m high), the option preferred by the construction team may be mobile elevated work platforms (MEWPs), like that shown in Figure 19.9. These have the advantage that they do not need the specialist labour or the erection time required for an access scaffold. The disadvantage of using MEWPs is that the scaffold structure – which would normally have been provided for access but may also have provided the reinforcement with structural stability and resistance to external loads – is no longer present. The reinforcement should then be stabilised by other designed means.



Figure 19.9 Independently supported pier formwork (courtesy of Merseylink CJV)



Figure 19.7 Bridge abutment with access provided by mobile elevated working platform (MEWP) (courtesy of BBCL)



Figure 19.8 Independently supported pier formwork: Queensferry Crossing (courtesy of FCBC)

Concrete construction: position of the construction joints

The position of the construction joints will determine the amount of reinforcement that needs to be fixed, the support and the size of formwork that will be required to contain the concrete. This needs to be carefully considered. After forming, construction joints will be visible. These can be disguised by horizontal and vertical features.

Concrete construction: support to the steel reinforcement

When detailing reinforcement, the operatives who have to fix the reinforcement should always be remembered. The sequence in which the reinforcement is fitted, the space needed to fit the bars, ensuring the reinforcement does not clash, and how the weight of the reinforcement is supported should all be

considered by the detailer. The function of most commercially available spacers and chairs is only to provide cover. These are not necessarily adequate to support the weight of the reinforcement, so temporary works may be required.

Modern advances in reinforcement drafting mean that even very complex arrangements can be created without reinforcement clashing (Figure 19.10).

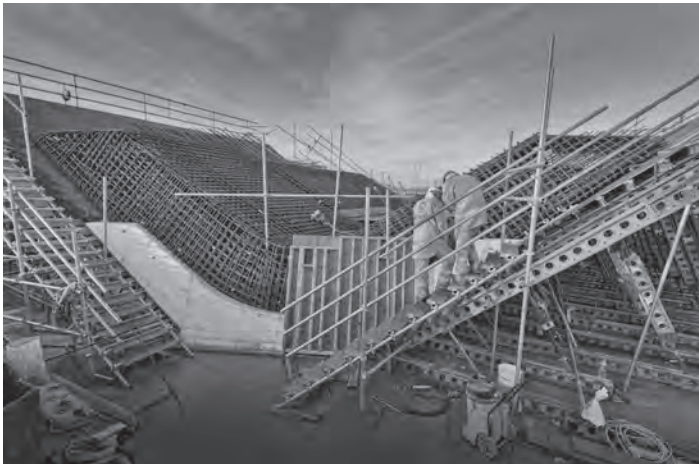


Figure 19.10 Ely bypass inclined pier reinforcement and supporting temporary works (courtesy of Cordek)

Advice is published by the Temporary Works Forum (TWF, 2013) on reinforcement detailing and temporary support. In general, the stability of the reinforcement should be considered as an item requiring temporary works (Figures 19.11 and 19.12).



Figure 19.11 Bridge abutment being constructed adjacent to live railway: temporary vertical posts support formwork, which in turn supports reinforcement and provides a protection screen between the workforce and the railway – access for fixing the reinforcement is from the rear of the wall; the front face must be fixed before the rear face (courtesy of Balfour Beatty and Tony Gee)

Reinforcement with excessive lengths of bars protruding from construction joints, cantilevering reinforcement and



Figure 19.12 Once the reinforcement has been fixed, the rear shutter can be fitted; once temporary propping is installed to the rear shutter, the vertical posts can be removed (courtesy of Balfour Beatty)

sloping reinforcement (Figures 19.13 and 19.14) will all require more temporary works than reinforcement with normal lap lengths, detailed so that bars are not required to cantilever and with vertical faces. The latter will require less temporary works and all be inherently safer. It is not good practice to detail horizontal bars with a greater diameter than the vertical bars or to have larger diameter vertical bars positioned above and lapped to smaller diameter vertical bars.

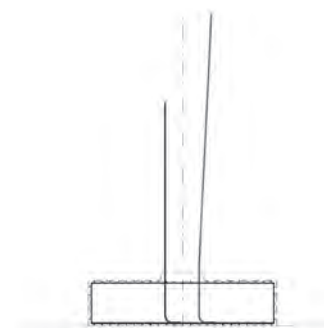


Figure 19.13 Problematic reinforcement detailing (base pour): example of reinforcement that would require significant temporary works; B32 inclined starter bar projecting 5m above kicker; weight per bar, 38 kg

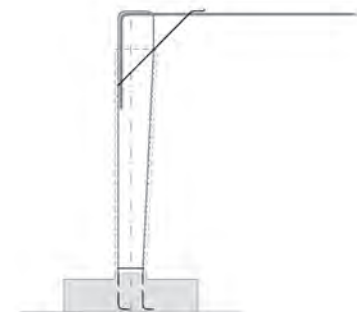


Figure 19.14 Problematic reinforcement detailing (first lift): example of reinforcement that would require significant temporary works; B40 bar, 9.2m long, projecting horizontally 6.9m; weight per bar, 91 kg

Concrete construction: support to the wet concrete

It should always be remembered that wet concrete applies a hydrostatic pressure normal to the surfaces confining it.

Formwork

The part of the temporary works forming the face of the concrete is called formwork (Figure 19.15). This can be a proprietary panel system or a bespoke one, manufactured for the purpose. Bespoke formwork usually comprises a form face (usually plywood or a timber board product), secondaries, which support the form face, and primaries, which support the secondaries. Additional continuous members may be used to align separate formwork panels. (Bespoke timber formwork is also known as shuttering in the UK and the practice uses terms borrowed from the shipbuilding industry, such as *walings*).



Figure 19.15 Kylesku Bridge construction; formwork and falsework to inclined piers (courtesy of Tony Gee)

If the concrete structure has two opposing faces, it is common practice to use removable structural ties, which connect one form to another through the concrete. If formwork ties cannot be used (e.g. if the faces of the wall are too far apart, or there is only one form face that has to be supported, or the form faces are not opposing), an external means of support will be required.

Sloping form faces will produce hydrostatic pressure normal to the form face. Formwork may be required to the upper surface of sloping members to confine the wet concrete.

If the formwork is to be reused many times, it may be more economical to construct it with a steel frame, with steel, timber or composite facing material.

Guidance on the design of formworks can be found in *Formwork: A Guide to Good Practice* (Concrete Society, 2012).

Falsework

The part of the temporary works that provides external support to the formwork, or temporary support to the partially completed permanent structural form, is called falsework. The weight of the concrete or partially completed permanent works will cause vertical loads. Unbalanced hydrostatic pressure from the concrete normal to the face of the formwork may result in horizontal loads. In addition, the falsework may need to resist environmental loads, such as wind, snow, waves or water currents, provide access for the workforce, provide stability to reinforcement and resist the loads arising from the construction (impact loading, plant vibration, horizontal and vertical dynamic loading owing to placing preformed elements, concrete discharging and concrete heaping).

The falsework should provide an adequate load path for all applied loads to reach the vertical and horizontal supports at the falsework foundations. Inclined falsework that supports single-sided formwork will produce vertical and horizontal loads. If the load applied is only horizontal, a load path to resist the induced vertical load at the end of the inclined prop will be required or a mechanism will be created and the formwork will move vertically.

It is recommended that falsework be designed in accordance with the guidance given in BS 5975 (BSI, 2019).

Concrete construction: prefabricated elements

In cases where the time periods to which access to the site can be granted are limited and access by crane is available, prefabricated modular units have been used to construct piers and abutments. The example of Over Bridge 59 (Figure 19.16) shows abutments constructed from modular precast units,



Figure 19.16 Over Bridge 59: abutments constructed using modular precast units (courtesy of Tony Gee)

installed by crane during short rail possessions, which became the substructure for a modular precast bridge.

Superstructure construction

The bridge superstructure can be constructed in a number of ways. These are covered in great detail in *Bridge Deck Erection Equipment – A Best Practice Guide* (Harridge and Mayer, 2018). Guidance is given to a lesser extent in other chapters of this manual (Chapters 9, 11, 14, 15, 16, 17 and 20).

Where some direct access to the site adjacent to and below the bridge is available, the following methods are available.

In situ

If the obstacle being crossed permits a temporary support system to be founded at ground level, the space under the bridge can be filled by falsework (Figure 19.17), the bridge deck formed on top of this (Figure 19.18) and, once it is self-supporting, the falsework struck and removed. This method of construction is typical of masonry arches and in situ concrete.

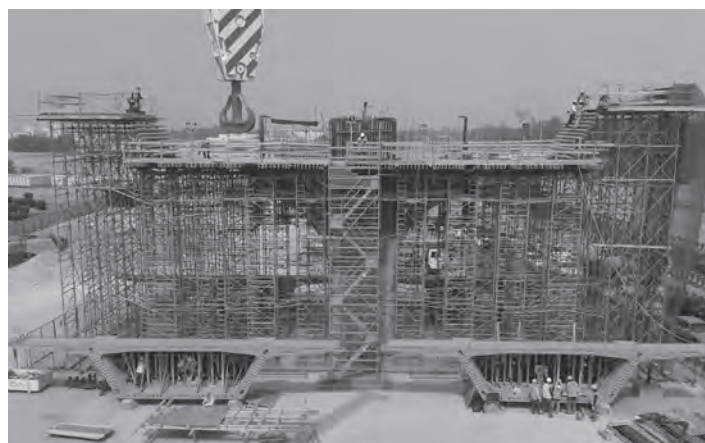


Figure 19.17 System scaffold: Jamal Abdul Nasser Street Bridge (courtesy of Tony Gee)

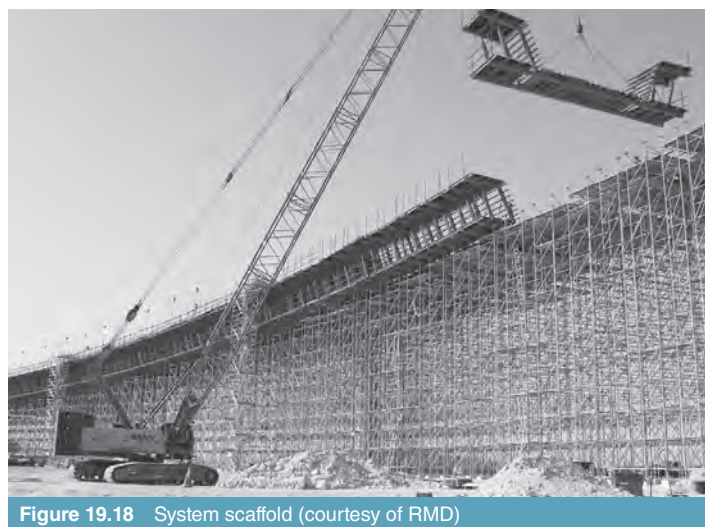


Figure 19.18 System scaffold (courtesy of RMD)

Combined prefabricated elements and in situ

Usually, beams that can support their own weight and the loads from operations enable the remainder of the deck to be constructed in situ (Figure 19.19). If a bridge is to be constructed in this way, care should be taken that the temporary support condition when the element is first landed and before it is incorporated into the permanent works is properly considered.

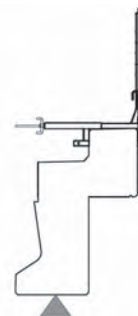


Figure 19.19 Example of Y-E precast reinforced concrete 'boot' beam fitted with precast parapet and temporary edge protection before being placed on elastomeric bearings: as a minimum, wind loads will be applied to the arrangement; the resolution of the vertical and horizontal loads does not pass through the centre of the elastomeric bearing – an elastomeric bearing will not resist rotation; unless adequate temporary support is provided, it is possible for the beam to fall off the bearing (this is known to have happened)

Minimal temporary works will be required if the beams are sufficiently long to span between the piers or abutment and have been designed for the temporary support conditions (Figure 19.20). If not, it may be necessary to construct a completely new temporary works support and foundations, duplicating the function of the permanent pier in the temporary condition.



Figure 19.20 Placing precast concrete beams, Richmond Street Bridge (courtesy of John Murphy and Sons)

It is usual to lift steel beams into position in pairs, with temporary bracing between them (Figure 19.21). Connecting them in pairs aids stability once landed and the bracing provides resistance to lateral torsional buckling during the follow-on

deck construction. Ideally, the bracing will be left in place to eliminate the risks associated with removing it.



Figure 19.21 Placing twinned steel beams with precast kerb, Carters Bridge (courtesy of John Murphy and Sons)

When constructing an in situ concrete deck using formwork to support the wet concrete, leaving it in place permanently will remove the risk associated with stripping it (Figure 19.22). A number of proprietary solutions are available.



Figure 19.22 Formwork for construction of in situ concrete deck between girders: wet concrete is supported by permanent formwork, cantilever edges of deck are supported by removable RMD Paraslim brackets (courtesy of RMD)

Constructed elsewhere and moved into position

This is typified by bridges installed by sliding (Figure 19.23), launching, flotation or SPMT (Figure 19.24). The bridge is constructed as before, substantially complete (enough to be self-supporting), remotely from the finished location. This may be appropriate for single and multispan structures.

Where access to the site adjacent and below the bridge is limited, bridge construction is carried out in these ways.

Launching

Launching a bridge involves constructing it in line with the span, then moving it in the direction of the span so that it cantilevers over its permanent supports until it reaches the next



Figure 19.23 Channel Tunnel Rail Link 342: multispan viaduct with piers and abutments slid into position during a rail possession using slide tracks and hydraulic jacks (courtesy of VINCI)



Figure 19.24 P954 flyover, Bahrain: bridge installation by SPMT (courtesy of Tony Gee)

support. This leads to very high bending and shear loads in the part of the bridge deck directly above the leading support. As the bridge is launched, every part of the span has to cross the leading support, so every part of the deck is subject to these loads. Cantilever deflections, moments and shears can be reduced by using a lighter-weight launching nose or by pulling the nose up using guy lines and towers (in effect, an upwards bow string). If access is possible, the cantilever span can be reduced by introducing intermediate temporary towers.

It is necessary for there always to be sufficient counterweight to balance the portion of the bridge that is cantilevering.

Care should be taken if launching truss structures, as there will be secondary bending effects in the chord member on which the bridge is supported during launching between the nodes.

Case study: Pennair Bridge

An early example of bridge launching is the construction of the Pennair Railway Bridge (Figure 19.25), on the Madras Railway North-West Line, 234 miles from Madras, India (Stoney, 1870).

The bridge is 1674 ft (510 m) long, comprising paired 4 ft (1.2 m) deep wrought iron, rivet plate girders, with 24 spans, each of 64 ft (19.5 m) clear, on abutments and 23 rubble-filled masonry-faced piers, which stand 20 ft (6 m) above the valley flood plain.

The bridge girders were fabricated in England in lengths from 21 to 37 ft (6.4–11.3 m), transported by ship to Madras and then onward using the railway under construction to the bridge site. The first girders arrived on-site on 21 October 1868. The girders were offloaded by hand and spliced on-site, using over 30 000 rivets, being assembled into pairs of beams 140 ft (42.7 m) long, continuous over two spans, weighing 49 tons (44 t) per assembly.

Cast iron rollers were set up on sleepers on the ground and tops of the abutment and piers. Motive power was from a crab winch mounted to the leading end of the bridge, using a 6 in (150 mm) Manila rope from the winch reeved through a pulley anchored to the pier, with the end of the rope attached to the bridge. The winch was operated by 12 men; the average speed of launch was reported as 420 ft (128 m) in 9 h. The girders were then jacked up using screw jacks, the rollers removed and the girders jacked down into position.

The bridge was opened on 1 August 1869, 10 months after the arrival of the girders.

Stoney (1870) recommended the procedure as being safer than conventional timber staging and riveting in situ; it kept the work site out of the river, away from flooding, and allowed construction of girders to be carried out simultaneously. Stoney remarks that this method of bridge rolling system had been in use for a number of years on the Madras Railway and that the method was tried and tested. He also gives examples of contemporary bridges being constructed; the lattice girder Cere Viaduct on the Paris to Orleans railway, with spans of 164 ft (50 m), and the plate girders of the Grande Rivière Bridge, Mauritius.

Span-by-span

Span-by-span construction requires a self-propelled falsework system that temporarily bridges the gap between piers and allows construction of the permanent bridge deck. They are supported from the piers of the permanent bridge. The following are examples.

Movable scaffold systems (MSSs)

These support the formwork required to support the reinforcement and wet concrete until it becomes self-supporting (Figure 19.26). These systems enable the formwork to be struck and moved to the next span.



Figure 19.26 Movable scaffold system courtesy (courtesy of Merseylink CJV)

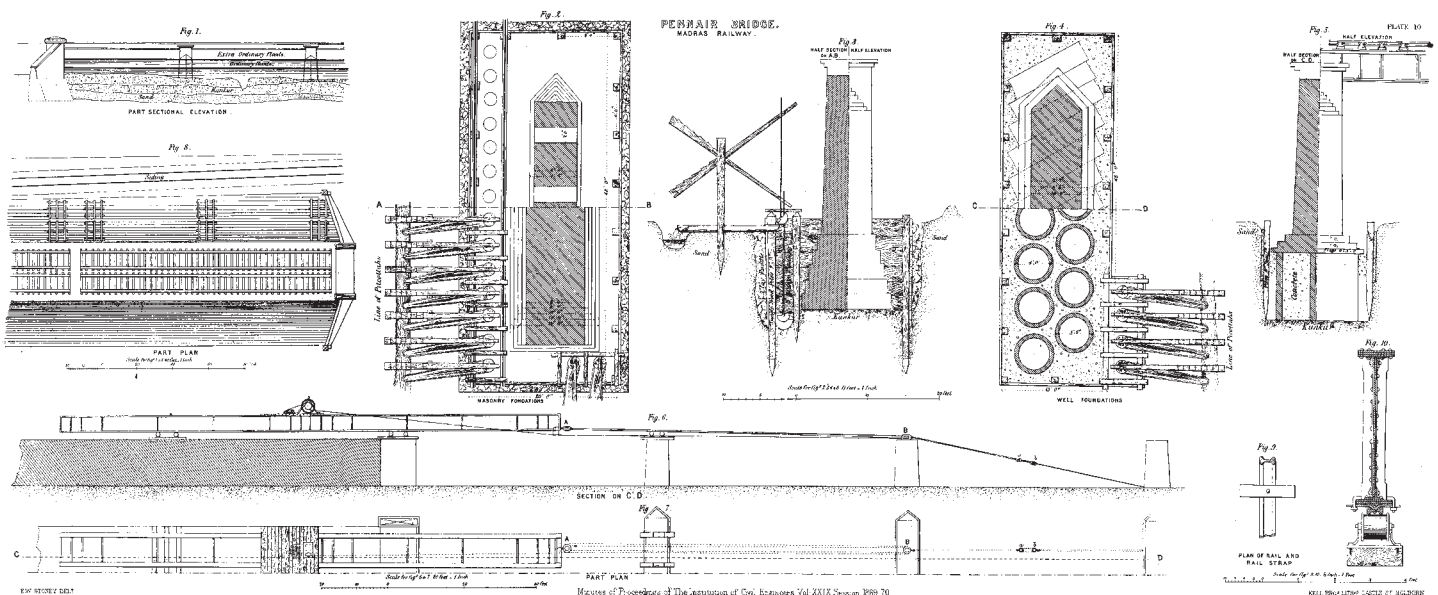


Figure 19.25 Pennair Railway Bridge (Stoney, 1870). The illustration shows: Fig. 2, 3 and 4 (top row; three centre illustrations) cofferdam and dewatering for the foundation construction; Fig. 6 and 7 (bottom row; horizontal images at left) the bridge launching arrangement; Fig. 8 (top row, second from top) the temporary sidings and working area for assembly and launching

Bridge erection gantries

These allow the crane handling and support of prefabricated elements, either longitudinal beams spanning from pier to pier or segments of deck, which are subsequently stitched together (Figure 19.27).



Figure 19.27 Jamal Abdul Nasser Street Bridge: overhead gantry supporting precast elements (courtesy of Tony Gee)

Self-supporting in temporary conditions

Where the distance between the piers is too great to be spanned by a self-propelled falsework system, the deck and permanent works must be designed to be self-supporting in the temporary condition (Figures 19.28 and 19.29). This is typical of balanced cantilever construction, stayed balanced cantilever construction, cable suspension bridges and arch suspension bridges.



Figure 19.28 Queensferry Crossing: deck construction (courtesy of Transport Scotland)

The increments by which the deck is constructed can be prefabricated elsewhere and lifted into position or cast in situ on travelling formwork.



Figure 19.29 Mersey Gateway: deck construction (courtesy of Merseylink CJV)

Further reading

The construction of modern superstructure is dealt with in the seminal reference *ICE Bridge Deck Erection Equipment: A Best Practice Guide* (Harridge and Mayer, 2018). This covers the following aspects of bridge deck construction to a very high level:

- precast balanced cantilever erection by launching gantry
- precast segmental span-by-span erection by launching gantry
- precast girder erection by launching gantry
- precast full span erection by launching gantry
- cantilever erection by lifting frames
- progressive cantilever method using stay cables
- in situ form travellers
- movable scaffolding systems
- incremental launching method
- prefabricated bridge installation methods.

Effects of successive construction stages and temporary conditions during bridge deck construction

During construction, the bridge exists in a number of temporary states, which the permanent works designer should consider.

Temporary fixity

During construction, the stability of the temporary works and the position of the item being constructed must be maintained. This will require the temporary works and the element under

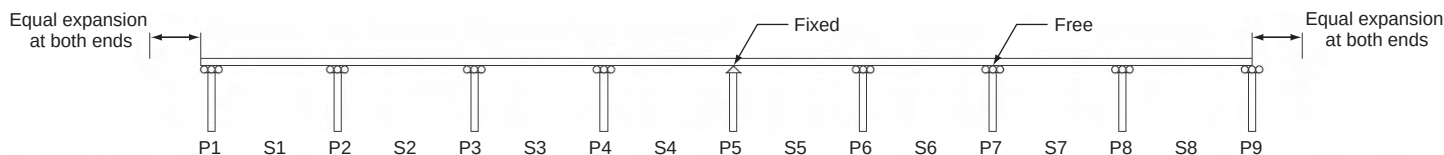


Figure 19.30 Consider the construction of a multispan continuous bridge with longitudinal restraint provided in the permanent condition by a fixed bearings at a single position

construction to be restrained in all three axes – vertically, longitudinally and transversely.

The temporary works and its supports should be sufficiently rigid that the structure does not deflect during construction and the element is built in the correct place with the correct deflected profile. For example, for a single span simply supported at each end, this would comprise rigid vertical supports at both ends, rigid transverse restraints at both ends, a rigid longitudinal restraint at one end and no longitudinal restraint at the other end.

Long-span bridges

Special consideration should be given to finished bridge decks comprising a multispan continuous structure with only one support providing longitudinal fixity (Figure 19.30), if, in the temporary condition, it comprises a series of simple supported spans, each with independent means of longitudinal fixity. During connection of the spans, the sequence of release and transfer of fixity to a single longitudinal bearing, with the consequential increase in range of thermal expansion and contraction, as well as any consequential increases in horizontal load effects on the temporary works or reduction in restraint stability, should be considered. Temporary bearings and jacking and packing to change bearing types may be required.

Consider a bridge construction comprising precast elements spanning longitudinally with in situ concrete stitches between them (Figure 19.31). In the first instance, the precast beams will be landed onto temporary supports. The temporary supports can make use of the permanent piers if the design will accommodate this (temporary support locations off the new piers or abutments). If not, separate temporary structures may be required to provide support.

The economics of designing the permanent works to accommodate the temporary support positions should be considered,

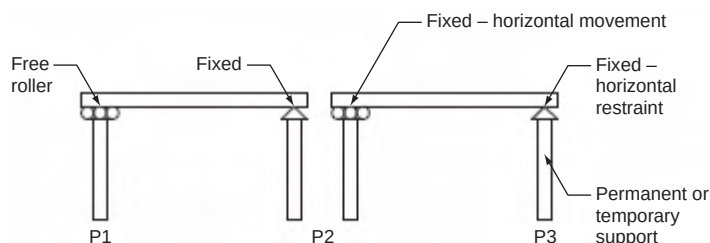


Figure 19.31 Consider a bridge construction comprising precast elements spanning longitudinally with in situ concrete stitches between them

as this will reduce both the temporary works costs and the risks involved in erection and dismantling.

One temporary support to each beam will be nominally fixed longitudinally, the other will be free.

It may be a requirement of the permanent works design that before the in situ stitch can be constructed, the temporary support is nominally fixed longitudinally, either side of the stitch, to prevent movement of the beams while the in situ concrete cures (Figure 19.32).

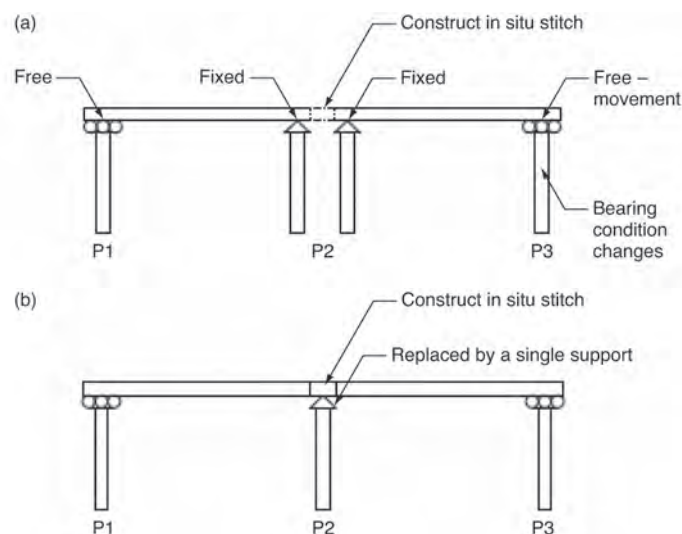


Figure 19.32 The temporary support may need to be nominally fixed longitudinally, either side of the stitch (a), before the in situ stitch can be constructed (b)

As bridge construction progresses, the fixity at each set of temporary supports will have to be adjusted to suit the restraint conditions required at the stitch (Figure 19.33). For a long-span structure, the amount of movement that must be allowed for at the temporary bearings may be considerable. The permanent and temporary works should be designed to accommodate this.

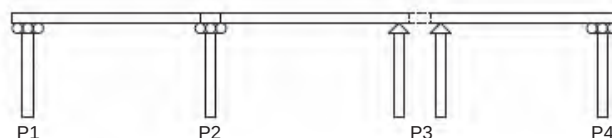


Figure 19.33 As bridge construction progresses, the fixity at each set of temporary supports will have to be adjusted to suit the restraint conditions required at the stitch

Cantilever structures

Horizontal restraint, greater than that required in the permanent case, may be required for balanced cantilever structures to provide transverse stability during construction (Figures 19.34 and 19.35).

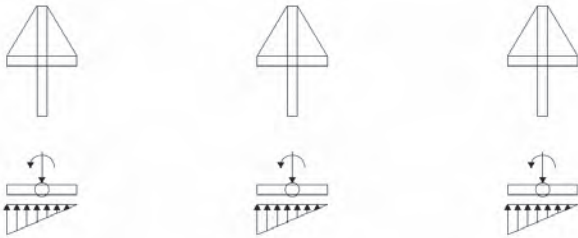


Figure 19.34 During construction, each of the cantilever spans will have different loading, owing to wind; this can only be resisted by moment fixity in plan at the pylon (think of a weather vane)

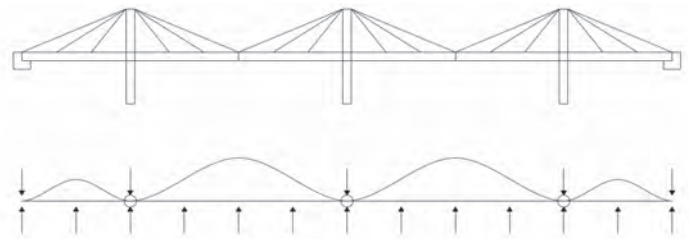


Figure 19.35 Elevation and plan of a balanced cantilever bridge. Once completed and restrained at the abutments, the deck will act as a continuous beam and the in-plan moment restraint at the piers will not be required

from the centre of the span towards the supports. This allows the deck to deflect progressively under its own self-weight. If the direction of striking were reversed from the ends to the centre, the support in the centre would tend to act as a stiff point and attract load, which could lead to the temporary works becoming overloaded.

Changes in continuity, stiffness and built-in stresses

In beam theory, the continuity between two spans over an intermediate support has the effect of reducing the span moments by producing a moment over the support and attracting shear to the central support. However, multispan bridges are often constructed as single simply supported spans, which are then joined over the intermediate supports. In the temporary condition, as a simply supported beam, for the same loads, the moments and deflections will be greater and the shears and reactions at one end increased. This effect should be taken into account and may be prominent if the bridge is being constructed one span at a time from the deck of the previous span.

It is common for beams that form part of the permanent bridge deck to be used as falsework to support the next stages in the construction, for example composite construction bridges where the steel girders are erected first and the concrete deck cast after. As the weight of wet concrete is added to the steel beams, they will become stressed and deflect. Once the concrete has hardened, the stresses formed in the steel beams will become locked in and must be accounted for in the permanent works design. If the deck is cast in sections, the effect will be cumulative.

For aesthetic purposes, the deflected form can be disguised or hidden by precambering the beams beforehand, so that afterwards the curvature of the deck appears smooth.

Transfer of load from falsework to self-supporting

In cases where a deck is supported over its length during construction by an external means, such as falsework supported off the ground or from above, suspended by a bridge erection gantry, it is important that the falsework is struck in sequence

Effects of longitudinal construction joints

When casting adjacent sections of spanning concrete slabs or decks, the effect of the stiffness of the falsework system on the subsequent castings must be taken into account. This is important when a structural joint has to be made between adjacent sections of the structure that have construction joints the full length of the span.

Once the first section of deck has gained strength and the falsework struck (if appropriate), the first section of deck will have a deflected profile. The adjacent section of formwork must be matched to the first section of deck with a tight grout seal and supported by falsework. Once loaded by concrete, the supporting falsework will deflect, but the adjacent (first) structure will remain in place. This can cause a gap to open between the formwork and the structure, losing the tight grout seal.

To avoid this, the falsework adjacent to the existing structure must be sufficiently stiff to prevent deflection or the formwork must be supported from the first deck, or a combination of both.

Bridge launching

When launching composite construction decks, it is usual to cast the concrete deck after the bridge has been launched. Launching the bridge without the concrete deck results in a lighter structure, reducing the required capacity of launching equipment, rollers, pulling mechanism and lowering or load transfer mechanism.

Also, if the concrete deck is cast before the bridge is launched, then during launching the entire length of the concrete bridge deck will be subject to tension as it passes over the supports. The tension may cause cracks to form in the deck, which will affect the stiffness of the structure and may result in the deflections and distribution of load being different from that predicted by structural modelling.

Case studies

Case study 1: Comparison between Mersey Gateway and Queensferry Crossing

Queensferry Crossing

The Queensferry Crossing, is a new four-lane cable-stayed bridge spanning the Firth of Forth.

It has three 216 m high towers, two 223 m side spans and two 650 m main spans, with a navigable clearance of around 45 m at mean high water springs (MHWS). Construction started in 2011 and the bridge was opened in 2017.

The bridge alignment was chosen so that the north abutment would be founded on rock outcrops and could connect to the existing road network and so that one of the piers would be founded on Beamer Rock, a naturally occurring rock outcrop in midchannel.

Mersey Gateway

The Mersey Gateway is a new six-lane cable-stayed bridge spanning the estuary of the River Mersey.

Each span and pier height is different; the bridge has a 110 m high north pier, 80 m high central pier and 125 m high south pier. The river spans are 181 m (side span north), 294 m (main span north), 318 m (main span south) and 205 m (side span south), with a clearance of around 17 m above MHWS. Construction started in 2014 and the bridge was opened in 2017.

The alignment was chosen to suit the restrictions to the positions of the connecting roads to the north and south.

Tower and V-pier foundations: comparison of construction methods and temporary works

Queensferry Crossing

The Queensferry Crossing, the new bridge spanning the Firth of Forth, is aligned so that the north abutment is founded on rock outcrops and connects to the existing road network and so that one of the piers is founded on Beamer Rock, a naturally occurring rock outcrop in midchannel.

The bridge is supported by gravity foundations at the north, central and south piers. For the north and south piers, the gravity foundations were constructed using two-part caissons up to 31 m diameter by 35 m high, weighing in excess of 1200 t (Figures 19.36 and 19.37). The lower part of the caisson was designed to be left in place, with the upper part removable. The lower part comprised a double-walled steel structure and the upper part a removable single-skin steel sheet pile structure. Both lower and upper parts of the caisson were fabricated remotely in a shipyard and floated to the site.

Once positioned on the sea bed, the caisson was anchored and sealed to the seabed and the material removed from within.



Figure 19.36 Installation of top single-skin caisson on permanent double-skin caisson before sinking (courtesy of FCBC)



Figure 19.37 Base of caisson after installation (courtesy of FCBC)

Beamer Rock, which supports the central pier, is exposed during the tidal range. Here, a caisson was formed from interlocking modular precast concrete L-units anchored and sealed to the rock. Wells were drilled to control and relieve uplift owing to the tidal range.

The reinforced concrete bridge pier foundations were then formed within caissons.

Mersey Gateway

The Mersey Gateway is constructed in a tidal estuary, which at low tide exposes a large area of mudflats and the Mersey River channel. At high tide, the mudflats are covered. The alluvium is relatively shallow and covers weathered sandstone beds.

Access to construct the piers was limited. The tidal range of the estuary prevented the large-scale use of floating plant (Figure 19.38). Instead, a 1 km long temporary access bridge was constructed from riverbank to riverbank. Spurs constructed from the access bridge allowed access to three sides of the pier foundation.

The pier foundation cofferdam was formed from a 40 m diameter outer ring and a 21 m diameter inner ring of tied sheet

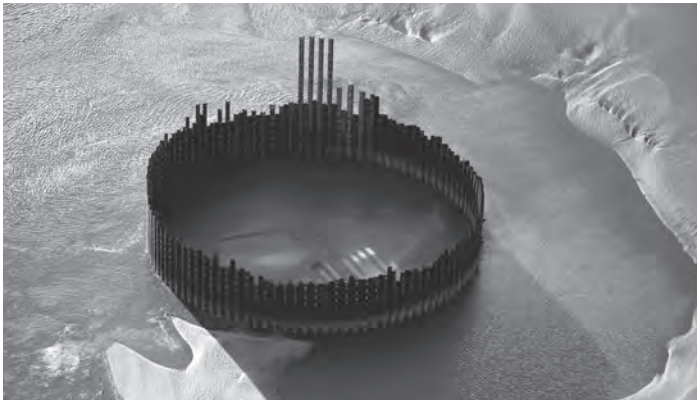


Figure 19.38 Installing sheet piles from jack-up barge – limited access and effectiveness (courtesy of Merseylink CJV)

piles toed into the weathered sandstone bedrock (Figure 19.39). The gap between the sheets was filled with granular material. The design of the cofferdam took into account the tidal range that would be experienced during construction, operation and dismantling and the potential effects of scour.



Figure 19.39 Excavating for pier foundation; access bridge in place (courtesy of Merseylink CJV)

Dewatering wells were installed within the inner ring of the cofferdam and the alluvium within was removed to the bedrock. A mass concrete plug was formed and the reinforced concrete foundation constructed. On completion of the bridge, the inner and outer rings of sheet piles were removed.

Bridge deck: comparison of construction methods and temporary works

Queensferry Crossing

The bridge deck of Queensferry Crossing comprises a composite concrete and steel box girder with steel bottom flange, webs and bracing and a transverse post-tensioned reinforced concrete deck slab. The structural steel part of the deck was prefabricated in China in segments measuring 30m wide

by 16.2m long, weighing 320t, which were delivered to a local dockyard facility and then the 40m wide concrete deck was cast.

The prefabricated composite deck sections were then transferred from the dockside to a barge using SPMTs and pulled by tug to the bridge site, where they were lifted into position using a derrick crane mounted to the end of the previously constructed bridge deck. Once positioned, the deck section was connected by welding and bolting to the erected deck. Cable stays were then connected and stressed (Figures 19.40 and 19.41).



Figure 19.40 Queensferry Crossing: deck construction (courtesy of Transport Scotland)



Figure 19.41 Queensferry Crossing: deck construction (courtesy of Transport Scotland)

Erection was carried out alternately at both ends of the three cantilever decks to maintain a balanced construction.

Mersey Gateway

The bridge deck of the Mersey Gateway comprises post-tensioned and reinforced concrete sections, comprising bottom flange, webs and a 33m wide top deck. The deck was cast in situ in 6m lengths on form travellers and then post-tensioned

after gaining strength (Figures 19.42 and 19.43). Cable stays were then connected and stressed. Erection was carried out at both ends of the deck to maintain a balanced construction.



Figure 19.42 Mersey Gateway: deck construction (courtesy of Merseylink CJV)



Figure 19.43 Mersey Gateway: deck construction (courtesy of Merseylink CJV)

Site constraints

Geography

Queensferry Crossing: The site is in the UK, with a temperate climate, no extremes in weather and some storms and gales. The obstacle crossed is a deep-water estuary with occasional rock outcropping at the surface. The route of the bridge was chosen to make use of natural rock outcrops in midchannel and at the north abutment.

Mersey Gateway: The site is in the UK, with a temperate climate, no extremes in weather and occasional storms and gales. The obstacle crossed is a tidal estuary with fast flowing water in both directions and the potential for scour.

Source of plant labour and materials

Both sites are in a developed country; all resources could be sourced locally.

Man-made constraints

Parts of the site for the Mersey Gateway cross contaminated ground.

Access to the site

Queensferry Crossing: The obstacle crossed is a deep-water estuary with occasional rock outcropping at the surface. Access to both ends of the bridge was possible by roads. The bridge could be accessed from sea by floating plant. Overall, access was good.

Mersey Gateway: The obstacle crossed is a tidal estuary, with fast flowing water in both directions. Access to both ends of the bridge was possible by roads. There was no access to the bridge from the estuary. Overall, access was limited.

Working space

Queensferry Crossing: There was access for floating plant from 360°. The site was mostly not restricted by tides.

Mersey Gateway: Access was limited to that provided by the temporary bridge and spurs. The site was not restricted by tides.

Use of plant

Queensferry Crossing: Land-based cranes could be used at each end of the bridge. Large barges and heavy lift cranes could be used.

Mersey Gateway: Land-based cranes were operated from the temporary bridge.

Reasons to consider remote fabrication of temporary works for pier construction

Safety

Queensferry Crossing: Risks included deep-water construction. In situ foundation was the most practical option. The potential for on-site fabrication and assembly of temporary works was limited by the water depth. Off-site construction of temporary works reduced the amount of work to be carried out in the water.

Mersey Gateway: Risks included the fast flowing tidal estuary. In situ foundation was the most practical option. The potential for off-site fabrication and assembly of temporary works was limited by the water depth. On-site construction of temporary works was the most practical option.

Fabrication environment

For both bridges, off-site fabrication could be carried out in a more controlled environment.

Transport of fabricated components to the site

Queensferry Crossing: This was possible using seaborne transport and floating plant.

Mersey Gateway: Large prefabricated elements could not be transported to the pier site.

Reasons to consider remote fabrication of deck

Safety

For both sites, remote deck fabrication would reduce the requirement to work over water.

Fabrication environment

For both sites, off-site fabrication could be carried out in a more controlled environment.

Transport of fabricated component to site

Queensferry Crossing: This was possible using seaborne transport and floating plant.

Mersey Gateway: Large prefabricated elements could not be transported to the ends of the deck.

Construction joints

Queensferry Crossing: Special care had to be taken using trial erections of the bridge deck to match the adjacent units to ensure that permanent joints would be made properly.

Mersey Gateway: Forming the deck in situ allowed new sections to be match cast to previous sections.

Speed of construction

Queensferry Crossing: The speed of construction was limited by the cycle time of work to lift and joint prefabricated sections.

Mersey Gateway: The speed of construction was limited by the cycle time of work to position formwork, fix reinforcement, pour and cure concrete and strike formwork.

Case study 2: Loughor Bridge

The project involved removing an existing 230m multispan single-track rail bridge at the end of its life and replacing it with a new bridge wide enough for two tracks.

Site constraints

Geography

The site is in the UK, with a temperate climate, no extremes in weather and occasional storms and gales

The obstacle crossed is a tidal estuary, with fast flowing water in both directions and some risk of flooding.

Source of plant labour and materials

The site is in a developed country; all resources could be sourced locally.

Man-made constraints

The site surroundings are in a nature conservation area and site of special scientific interest. There were environmental limits on the time of year that access could be gained to certain areas of the site.

The existing bridge was in place. The period of time between the old bridge closing and the new bridge opening was limited.

The bridge is of historic interest and it was required that some parts of the original structure remained in place.

The area surrounding the site was occupied, so the rail alignment had to be maintained. As the railway could not be moved, the old bridge was in the way of the new bridge.

The existing old substructure and superstructure could not be reused.

Access to the site

The obstacle crossed is a tidal estuary, with fast flowing water in both directions.

Access to both ends of the bridge was possible by roads, with good communications.

Working space

Although the route of the bridge was occupied, land adjacent to the site was available at each end of the bridge.

Some form of access was required for plant to demolish or deconstruct the old bridge.

Use of plant

Land-based cranes could be used at each end of the bridge.

Barges with spud legs (jack-up barges) could be used in the estuary.

Reasons to consider remote fabrication

Safety

Risks included a fast flowing tidal estuary, the proximity of the existing railway, and the bridge construction.

Off-site construction would potentially reduce the amount of temporary works in the water.

Temporary works

The type and amount of temporary works depends on the method of fabrication and installation. It would be desirable to

limit or eliminate the requirement for temporary foundations in the river.

Disruption to adjacent sites

The client wanted to have the old bridge removed and replaced by the new one within a 250h blockade (10.5 days).

Obstruction caused by existing structure

As much of the new bridge as possible had to be built around the envelope of the old bridge without affecting rail operations.

Simultaneous working

Off-site construction would allow parts of the construction work to be programmed in parallel rather than in series, to enable concurrent construction.

Fabrication environment

The bridge could be constructed from components prefabricated off-site. If steel or precast concrete components were used, the bridge could be prefabricated in a workshop environment and then delivered to the site.

For steel components, the joints could be made on-site. For precast concrete, the joints could be made on-site but would need to gain strength through maturity.

Transport of fabricated component to site

The well-developed road network would permit transport of components to the site.

Permanent works design solution

The arrangement of the bridge was a trade-off between the length of span, the number of piers and the loading on the piers. The optimum solution chosen was a seven-span bridge with a maximum span length of 36 m, supported on the abutments and six intermediate piers. The form of the permanent works and the design and detail were chosen specifically with constructability in mind.

Substructure constructability

Abutments were constructed on dry land within cofferdams.

Piers were prefabricated tubular steel piled piers (the embedment of piles provided foundations) with precast concrete pier crossheads supported directly from the piles. The piers were installed using a 350t crane working from a jack-up barge (Figure 19.44).

A temporary crosshead extension was used (Figure 19.45); the piers were extended to one side by a series of structural steel beams supported by a connection to the permanent works crossheads at one end and temporary piles at the other.



Figure 19.44 Floating piling plant (courtesy of Tony Gee)



Figure 19.45 Temporary piers and temporary crosshead extensions (courtesy of Tony Gee)

Superstructure constructability

A temporary assembly area and launching platform was constructed on the land side of the abutment.

The bridge was prefabricated off-site in 24m lengths, delivered to the site, offloaded and assembled on a launching platform. The launching nose and pushing equipment were then added (Figure 19.46).



Figure 19.46 Launching nose, launching equipment rollers, foundations, jacks and anchorages and sliding system (courtesy of Tony Gee)

During the 249h closure of the track, the bridge skeleton was launched over the temporary crossheads, parallel to the existing bridge in three increments. Once launched, the in situ concrete deck was formed. Once the concrete deck had gained sufficient strength, it was used as a working platform for the craneage and plant required to cut, lift off and transport away the old bridge.

The new bridge was then slid sideways into the correct alignment and the end connections, ballast and track installed. The bridge could then be opened. The temporary crosshead extensions and temporary piles were then removed.

Concluding remark

The original crossing was by Brunel. It is interesting to note that some of the constraints would have been considered by the original engineers when it was first built in 1852.

Decommissioning, dismantling and demolition

The removal of a bridge may be just as problematic as its construction. For many bridge structures, the safest method is to remove them in the reverse sequence of their erection.

If it does not matter that spoil from the bridge can drop onto the ground below, the chosen method of demolition may involve using specialist plant, such as impact hammers

and hydraulic crushing jaws mounted on tracked or wheeled hydraulic excavators (Figure 19.47).



Figure 19.47 Bridge demolition M5 Jn 12 (courtesy of Norwest Holst)

If the demolition spoil cannot be dropped onto the ground below, for example, when over a river or existing railway, it must be removed in sections. The location of the cuts, the size of the resulting sections and the temporary works required to support the bridge while it is being deconstructed should be designed by suitably qualified and experienced engineers.

Mobile cranes can be used to remove a bridge in sections, providing there is suitable access for the crane to get to the site, space to erect and operate it, foundations capable of supporting it and somewhere to put the pieces being removed. The availability of larger cranes means that the bridge can be removed in larger pieces from longer distances.

When lifting parts of an existing structure, extreme caution should be used in calculating its weight and the position of



Figure 19.48 Sydney Road Bridge: demolition of reinforced concrete arch by saw cutting and crane lifting (courtesy of John Murphy and Sons)

the centre of gravity. A larger lift capacity can be specified to provide a factor of comfort. However, if the jib of the crane is not positioned directly above the centre of gravity, the load will move horizontally. If the bridge section can be lifted by hydraulic jacks beforehand and the centre of gravity calculated, the risk will be eliminated. If this is not possible, suitably designed temporary guides may be used. If the risk cannot be eliminated, it must be dealt with by operational means. Therefore, the designer should always try to ensure that the item being lifted can be put back down temporarily (from where it has been lifted) so that it can be re-slung if necessary.

Extreme care should be taken when deconstructing an arch in sections (Figure 19.48), to ensure that the resultant out-of-balance horizontal thrusts are resisted.

It may not be possible for a mobile crane to reach the site. In this case, alternative means of removing the bridge will be required, such as temporary gantries (Figure 19.49) or de-launching.



Figure 19.49 Steventon Viaduct: removal of existing bridge without the use of a crane, while supported on a temporary gantry (courtesy of VINCI)

designers to ensure (so far as is reasonably practicable) that any design prepared for the purposes of ‘construction work’ (including construction, maintenance, repair or demolition) takes into account the general principles of prevention and any preconstruction information to eliminate risks, so far as is reasonably practicable (SFARP), and reduces any remaining risks that affect construction workers.

Many readers will be familiar with methods for controlling risks that follow the form:

- eliminate
- reduce
- inform
- control.

Of these, the most important is ‘eliminate’. If a risk is not eliminated, it remains and so, no matter how well controlled, the possibility of it occurring remains. Ultimately, it is then a statistical certainty that, no matter how well controlled it is, it will eventually happen.

The guide from CIRIA C755 (Gilbertson, 2015) contains much good advice on the control of risk in considering general construction operations. It also gives the following advice:

Designers would be wise to stray on the side of caution – especially when considering significant (‘life or death’) risks because after a death legal analysis of reasonableness will be tinged by the reality and emotion of what has actually happened.

There is a feeling that after an accident, in retrospect, it is always obvious what more could have been done to prevent it occurring.

Recent statistics published by the UK Health and Safety Executive (HSE, 2017) for the construction industry list the causes of fatal accidents over the previous 5 years:

- 50% falls from height
- 10% trapped by something collapsing or overturning
- 10% struck by a moving vehicle
- 8% struck by a moving object
- 7% contact with electricity
- 5% contact with moving machinery
- 10% other.

Control of temporary works: the Bragg report and BS 5975

In the UK, a very good process for controlling temporary works is set out in BS 5975 (BSI, 2019). This method of controlling temporary works has existed for almost 40 years. It is based on the recommendations made in the Final Report of the Advisory Committee on Falsework (Bragg *et al.*, 1975), more commonly known as the Bragg report, which had studied a number of falsework collapse throughout the world. Part of the introduction succinctly sets out the problems of constructing temporary works and is as valid today as it was when written (see Box 19.1).

BS 5975: Code of practice for temporary works procedures and the permissible stress design of falsework

While a UK code, this approach to controlling temporary works has not been surpassed anywhere else in the world.

It is recommended that the procedures contained in the code are read and adopted. The aims of the procedures are to

- minimise the chance of errors being made
- maximise the chance of errors being discovered.

In the UK, the code applies to all organisations that would typically be involved in temporary works, including clients, management contractors, construction contractors, suppliers,

designers of permanent and temporary works, utilities operators and facilities maintenance operators.

This requires that every organisation have a procedure for the control of temporary works and a designated individual, a senior individual, who is responsible for establishing, implementing and maintaining the procedures for that organisation.

The code has sections specifically detailing the roles and responsibilities of the client, designers of permanent and temporary works, contractors and suppliers. It introduces the 'temporary works register' and the 'temporary works coordinator'.

While it was originally written with construction phase activities in mind, there is no reason why the methods and approach could not be instigated during the design phase and carried forward into the construction phase.

The problem

The constraints set by the need to temporarily support construction works in exactly the right position are severe. Each project is unique. There are few guides to good practice and there is often considerable doubt about the actual loads which will occur. Much reliance must be placed on the few skilled men who have experience of falsework design. The need to dismantle the equipment afterwards sets fresh problems. The contractor has his own materials and equipment which must be used where possible.

Working conditions on an exposed site are far from ideal. Good planning is needed to ensure that skilled men, materials and equipment are available at the right time. New hazards arise from weather, unexpected site conditions, and the non-availability of critical resources of men or materials, which make modification and improvisation necessary.

Another complication is that so many different specialist organisations are involved. The design of the permanent works may be done by one group of consulting engineers and the design of the falsework by another specialist group. A preliminary design may be needed for estimating purposes and then the final design may have to be done in a great hurry when the contract has been awarded. Equipment may be supplied and erected by specialist subcontractors who have not previously worked together. This diversity introduces difficulties of communication and a confusion of responsibilities which is worsened by complicated or imprecise contractual arrangements.

Finally the work force itself is often transient. Men sign up to work on one site but may not be prepared to stay with the contractor and move to another part of the country for his next job. This exacerbates the problems of providing training and maintaining the discipline on which safety depends.

Over all these activities hangs the need for economy and speed. The contractor is committed to completing the structure for a fixed cost by a certain date. The designer must provide a safety margin, yet he cannot afford to waste material. Safe construction practice must be followed in detail, faults must be corrected, yet the next critical step must not be delayed. Good planning, preparation and inspection are needed to ensure that corners are not cut.

In spite of all these difficulties most falseworks are designed, constructed and dismantled without accident. The aim of the Committee was to find out why accidents occurred and to recommend how they might be avoided.

Box 19.1 Extract from the Bragg report

Efficiency in construction

Many works have been published on the subject of lean construction. In its simplest form, lean means eliminating waste from everything we do. O'Connor and Swain (2013) define the following types of activity.

- **Value adding (VA).** This is the work that changes the shape or nature of the product (or service) in a way that contributes to the final form that the customer is willing to pay for.
- **Essential non-value adding activities (enVAs, or support activities).** These are the tasks that must be completed to enable the value adding activity to be completed, but do not themselves add value. For example, inspection does not add actual value but is necessary up to the point where a process can be improved so that inspection can be eliminated.
- **Waste.** This is any other activity or event associated with carrying out a particular work activity. Waste can be viewed from two perspectives:
 - waste in the work itself (e.g. excessive walking, looking for tools and materials, and poor quality)
 - introduced or 'enforced' waste (e.g. waiting for information, materials not supplied) that has prevented work activity from being carried out.

Applied simply to construction:

- value adding, VA = permanent works
- essential non-value adding, enVA = temporary works
- waste = waste and inefficiency.

Thus, the ideal outcome will be the solution that has the most desirable combination of VA and enVA (Figure 19.50), in whatever terms are most important to the project: cost, duration, environmental impact.

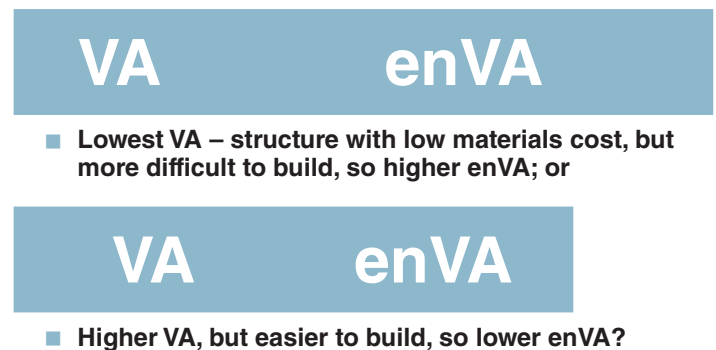


Figure 19.50 VA–enVA comparison

It may be that a design has a low VA content but a very high enVA content; for example, a bridge may have minimal material content or a very complex structural form but require costly and time-consuming temporary works and construction methodology. This may be less desirable to the client than a

bridge that is designed so that it requires less temporary works and is easier to construct (has less enVA) but might require slightly more VA content.

When to look at construction methodology for safety and efficiency

There are many forms of procurement method. Within the UK construction industry, there are the Governance for Railway Investment Projects (NR, 2017), Project Control Framework (HE, 2008), Plan of Work (RIBA, 2013) and Gateway Review Process (OGC, 2011), as well as the Institution of Structural Engineers' Plan of Work (IStructE 2020), which defines the progress of the project through the various stages from initial brief to final completion.

Most follow a similar pattern:

- pre-project phase
- options phase
- development phase
- construction phase.

These gateways (Tables 19.2 and 19.3) or stages provide criteria for the staged delivery of project so that progress and quality can be measured against defined targets and funding released appropriately.

It is suggested that there are four stages at which a formally recorded constructability review should be held:

- design options stage (concept and definition)
- design
- tendering
- project construction.

If possible, a consistent approach and standard should be used. There is evidence from past projects where buildability has been done well during some stages but was absent from others (e.g. well done at options, poorly done at development) Research carried out over a number of years in a number of countries indicates that 40% or so of accidents that occur during construction have their roots and causes in decisions made during the design stage (Williams, 2018).

	Pre-project or initiation		Options phase	Choose options	Development phase			Construction phase	Close	Use	End of use; demolition
HE (2017)	0. Strategy, shaping and prioritisation		1. Option identification	2. Option selection	3. Preliminary design	4. Statutory procedures and powers	5. Construction preparation	6. Construct, commission, handover		7. Closeout	End of use, decommissioning and demolition may have such significant consequences that it is appropriate to use the full project or programme management model for this stage, from pre-demolition project to demolition implementation
NR (2017)	1. Output definition	2. Feasibility	3. Option selection		4. Single option development	5. Detailed design		6. Construct, test, commission	7. Scheme handback	8. Project closeout	
RIBA (2013)	0. Strategic definition	1. Preparation and brief	2. Concept design		3. Developed design	4. Technical design		5. Construction	6. Handover and closeout	7. In use	
CIC (2007)	1. Preparation		2. Concept		3. Design development	4. Production information		5. Manufacture, installation & construction information	6. Handover and closeout	7. In use	
IStructE (2020)	0. Strategic definition	1. Preparation and brief	2. Concept design		3. Spatial coordination	4. Technical design	4.5. Production information	5. Manufacture & construction	6. Handover	7. In use	

Table 19.2 Typical project gateways (courtesy of Tony Gee)

		Options phase	Choose option	Development phase			Construction phase	Close	Use	End of use; demolition
HE (2017)	0. Strategy, shaping and prioritisation	1. Option identification	2. Option selection	3. Preliminary design	4. Statutory procedures and powers	5. Construction preparation	6. Construct, commission, handover		7. Closeout	End of use, decommissioning and demolition may have such significant consequences that it is appropriate to use the full project or programme management model for this stage, from pre-demolition project to demolition implementation
NR (2017)	1. Output definition 2. Feasibility	3. Option selection		4. Single option development	5. Detailed design		6. Construct, test, commission	7. Scheme handback	8. Project closeout	
RIBA (2013)	0. Strategic definition 1. Preparation and brief	2. Concept design		3. Developed design	4. Technical design		5. Construction	6. Handover and closeout	7. In use	
CIC (2007)	1. Preparation	2. Concept		3. Design development	4. Production information		5. Manufacture, installation & construction information	6. Handover and closeout	7. In use	
IStructE (2020)	0. Strategic definition 1. Preparation and brief	2. Concept design		3. Spatial coordination	4. Technical design	4.5. Production information	5. Manufacture & construction	6. Handover	7. In use	
Opportunities for construction review	Feasibility		Options	Preliminary design	Detail design		Construction			

Table 19.3 Opportunities for construction review (courtesy of Tony Gee)

By taking positive steps to improve constructability (reduce work content of permanent and temporary works), a designer will reduce the exposure to the statistical likelihood of accident (Table 19.4).

	Pre-project, initiate	Options phase	Choose option	Development phase			Construction phase	Close	Use	End of use; demolition
HE (2017)	0. Strategy, shaping and prioritisation	1. Option identification	2. Option selection	3. Preliminary design	4. Statutory procedures and powers	5. Construction preparation	6. Construct, commission, handover		7. Closeout	End of use, decommissioning and demolition may have such significant consequences that it is appropriate to use the full project or programme management model for this stage, from pre-demolition project to demolition implementation
NR (2017)	1. Output definition 2. Feasibility	3. Option selection		4. Single option development	5. Detailed design		6. Construct, test, commission	7. Scheme handback	8. Project closeout	
RIBA (2013)	0. Strategic definition 1. Preparation and brief	2. Concept design		3. Developed design	4. Technical design		5. Construction	6. Handover and closeout	7. In use	
CIC (2007)	1. Preparation	2. Concept		3. Design development	4. Production information		5. Manufacture, installation & construction information	6. Handover and closeout	7. In use	
IStructE (2020)	0. Strategic definition 1. Preparation and brief	2. Concept design		3. Spatial coordination	4. Technical design	4.5. Production information	5. Manufacture & construction	6. Handover	7. In use	
Opportunities for construction review	Feasibility		Options	Preliminary design	Detail design		Construction			

Table 19.4 Effect of design decisions on safety (courtesy of Tony Gee)

Table 19.5 shows a methodological approach to reviewing constructability. It identifies the stages when a constructability review may be useful and lists the prerequisites, aims and outputs of the review process. The table and the topics covered can be adapted and changed to suit the requirements of a particular project.

Stage	Option development and selection	Detail design	Preconstruction cost estimate build-up (tender)	Preconstruction stage (site work)
	Design development and options selection stages: NR (GRIP) 1, 2, 3; RIBA 0, 1, 2; HE 0, 1, 2; IStructE 0, 1, 2 Brief or requirements identified; variety of options (forms) developed, e.g. location, number of spans, concrete or steel, practicalities, sequence of construction, aesthetics If temporary works and constructability are not considered at options stage, the structure may be very difficult (unsafe) or impossible (too risky) to build	Development phase: NR (GRIP) 4, 5; RIBA 3, 4; HE 3, 4, 5; IStructE 3, 4 Form chosen; sequence of construction; bearing details, temporary fixity, restraint; thermal movements; residual stresses; temporary support to follow-on construction (temporary support to PC; PC acting as formwork resisting hydrostatic loads; PC acting as edge protection); unsupported reinforcement; correct poor buildability; eliminate unnecessary temporary works	Interface between development and construction phases: NR (GRIP) 5, 6; RIBA 4, 5; HE 5, 6; IStructE 4, 5 Form chosen; detailed design complete; sequence of build (transport, eliminate storage and double handling); plant required; temporary works (and plant) required; improvements to buildability	Construction phases: NR (GRIP) 6; RIBA 5; HE 6; IStructE 5
Actions to be completed before holding constructability review	Functional requirements established Geotechnical desk study carried out Alternative geographical locations identified Stakeholders identified Value criteria established Outline solutions determined Options; preliminary design Site investigation carried out Identify restrictions on timing and boundaries of site possession (rail interface, airport or flight path, tidal working, nearby power transmission)	Form of structure established Geographical locations established Stakeholders engaged Check value criteria are met Outline solutions determined Options, preliminary design advanced Identify interfaces between functions of works Identify interfaces with other contracts Further site investigation carried out Schedule of site possessions drawn up	Design sufficiently mature to avoid major changes to form, material type and sizes General arrangement and reinforcement detailing drawings substantially complete Stakeholders engaged Schedule of possession dates Check value criteria are met Likely timing of work established Key dates identified for constraints and deliverables (possessions, handovers, etc.) Interfaces between functions of works defined Interfaces with other contracts defined Identify work content and work packages Establish methods and routes for delivering materials to site Establish preferred methods for constructing works Source resources for constructing the works Preliminary programme construction of works Contact and engage with specialist contractors Sufficient ground investigation carried out in correct locations for temporary works assessments	Design complete General arrangement and reinforcement detailing drawings complete Necessary site possessions confirmed with stakeholders Budget established Materials for constructing the works sourced Resources for constructing the works engaged Construction team in place Divide site into work packages Develop programme for each work package so that each activity is identified Draft method statements and risk assessments for each work item

Table 19.5 Constructability review guide (courtesy of Tony Gee)

Stage	Option development and selection	Detail design	Preconstruction cost estimate build-up (tender)	Preconstruction stage (site work)
Aim of the constructability review	Identify construction materials and methodology What specialist plant and contractors are required? What geotechnical work is to be carried out? What obstacles have to be overcome? Access to the site (timings: tides, possessions, environmental, site of special scientific interest?) Weather or seasonal conditions (monsoon season, school term times) Identify major items of temporary works (better still, identify how to design them out) Identify potential logistics problems (Can the materials and resources be made available? Where will they be coming from? Does a mine, quarry or borrow pit need to be established? Do people need to be trained?) Identify potential transport problems (Can the materials and resources be delivered to site at the correct time?) Identify sources of uncertainty	Established method for choosing location of construction joints Size of preassembled components to be delivered to site chosen or determined Method of handling (craning) preassembled components. If reinforcement is to be prefabricated, how will the reinforcement cage temporary structure be designed? Method of connecting preassembled components Method of supporting preassembled components before they become incorporated in the works and are self-supporting (includes reinforcement, steelwork and precast concrete) Method of providing strength and stability during early maturity – concrete strength for slip forming Method of providing stability during early stages of construction – additional bracing in composite bridges where the steel beams would rely on the deck to restrain top flange; longitudinal restraint of continuous span bridges that are constructed in incremental spans Identify parts of permanent works that act as support for follow-on construction works (e.g. precast units resisting horizontal hydrostatic loads, permanent formwork) Construction (and access) loads to be carried by permanent works in temporary condition (Are decks strong enough for follow-on work? Access, MEWPs, telehandlers?) Identify access routes for transporting materials and plant to site Identify location of foundations required for any likely temporary works or plant loadings Identify gaps in information that will be required to design temporary works (Is ground investigation sufficient? Correct location? Upper layers sufficiently categorised?) Review boundary of work package and other packages and interfaces for possible constraints and interference (access routes, cranes oversailing, incompatible work carried out during the same possession)	Check that methods for constructing works are practical Check that sourcing of materials is practical Check that sequence or programme for constructing works is practical Check availability of specialist plant and contractors Check that all geotechnical risks are identified Identify solutions to obstacles that have to be overcome Identify solutions to potential logistics problems Identify sources of uncertainty	Review each work package in turn Review boundary of work package and which other packages it interfaces for possible constraints and interference (access routes, cranes oversailing) To each work package site (example given for a bridge construction site): - Define access, egress and any associated temporary works (access road, bellmouth, service crossing, temporary bridges) - Define site boundary fence, welfare, car parking and storage facilities and any associated temporary works (fence, hoarding, cabins, foundations, drainage, outrigger foundations for cranes or Hiabs) - Define hardstanding or laydown area (temporary works) - Define plant movement route (temporary works) Review each step in programme from first to last activity Define extent of temporary excavation (vertical supported or battered, dewatering) Identify affected services (temporary works: diversions, service crossings) Define limits on plant size and materials stockpiles next to temporary excavations Plant for carrying out excavation Define access requirements into and out of excavation, including rescue (temporary works) Define likely craneage requirements and possible crane locations Formwork for foundations Craneage for handling foundation reinforcement and formwork Support for foundation reinforcement Delivery of concrete Access for fixing wall reinforcement Craneage for handling wall reinforcement and formwork Support for wall reinforcement Formwork for wall Concrete delivery

Table 19.5 Continued

Stage	Option development and selection	Detail design	Preconstruction cost estimate build-up (tender)	Preconstruction stage (site work)
Actions to be carried out after constructability review	Best (compromise) solution chosen from options Safety case developed Design risk assessment conducted Risks designed out Risk register developed	Design solution developed from concept to working drawings Safety case developed Design risk assessment conducted Risks designed out Risk register developed Preliminary temporary works register developed; category of risk and check identified Necessary site possessions confirmed with stakeholders	Identify sources of materials for constructing the works Identify source of resources for constructing the works Safety case developed Risk register developed Temporary works register developed; category of risk and check identified Preliminary temporary works design prepared Cost temporary works Schedule movement orders Finalise programme construction of works based on actual timings and dates, taking into account access to site, seasonal effects, tides and possessions Calculate target cost for constructing the works	Obtain certified temporary works designs Finalise method statements and risk assessments for each of the work items Carry out work

Table 19.5 Continued

Concluding remarks

The goal of this chapter has been to highlight some of the practicalities and problems involved in constructing a bridge. It is hoped that if these are borne in mind during the conceptual, planning and design stages, through design reviews and early engagement with construction specialists, the resulting bridge design will be based on informed choices and that structures that fulfil the requirements for form and function will also be safe to construct.

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Appendix. Further reading

Many resources are available for temporary works designers. This is not an exhaustive list of sources, but may be of help.

British Constructional Steelwork Association

The British Constructional Steelwork Association publishes

- 38/05 BCSA. Guide to the erection of steel bridges.

The association also publishes a number of guides on such matters as welding and erection of steelwork.

Building Research Establishment

The Building Research Establishment (BRE), founded in 1921, is a national laboratory that specialises in research connected with the built environment.

The establishment was commissioned by the Federation of Piling Specialists to produce a guide for the design of working platforms, which is used throughout the construction industry in the UK:

- BRE470. Working platforms for tracked plant.

They have also published

- BRE495. Formwork for modern, efficient concrete construction.

CIRIA

The Construction Industry Research and Information Association (CIRIA) has a number of publications that cover important aspects of temporary works and safety (a number of these refer to other aspects of construction but are included as not all bridges are constructed on greenfield sites):

- C517. Temporary propping of deep excavations.
- C558. Permanent formwork in construction.
- C579. Retention of masonry facades. Best practice guide.
- C589. Masonry facade retention best practice.
- C699. Guidance on catastrophic events in construction.
- C703. Crane stability on-site.
- C740. Structural stability of buildings during refurbishment.
- C750. Groundwater control: design and practice.
- C755. CDM2015. Construction work sector guidance for designers.
- C761. Guide to tower crane foundation and tie design.
- P3155. Construction impact – prediction and assessment of damage from ground movements.
- R068. Lateral movement of heavy loads.
- R97. Trenching practice.
- R108. Concrete pressure on formwork.
- R113. Control of groundwater for temporary works.
- R136. Formwork striking times.
- R155. Design of bridges for improved buildability.
- SP95. Sheet pile cofferdams.
- SP121. Access to the workforce.

Concrete Bridge Development Group

- Current practice sheet 13. Incrementally launched bridges.
- Current practice sheet 14. Jacked box underbridges using the Ropkins system.
- Technical paper 9. Fast construction of segmental and launched bridges.

Concrete Society

The Concrete Society publishes a number of guides on temporary works. Of these, *Formwork: A Guide to Good Practice* (3rd edn) is an essential reference for all temporary works engineers designing to support wet concrete.

Construction Fixing Association

The Construction Fixing Association (CFA) was established in 1979 and represents the major manufacturers of fixing systems that are post-fixed, that is, set in drilled holes in all construction materials. They produce a number of guidance documents for product users.

Construction Plant-hire Association

The Construction Plant-hire Association produce a number of publications on temporary works equipment and construction plant, which are of value. These cover the use of

- concrete pumps
- construction hoists
- mobile elevated working platforms
- mobile and crawler cranes
- shoring
- tower cranes
- suction and vacuum excavators.

Particularly relating to cranes, the following may be of interest:

- CPA 1402. Tower cranes alongside railways.
- TCIG 0701. Safe use of top slew tower cranes.
- TCIG 0601. Guidance on the safe use of self-erecting tower cranes.
- TCIG 1101. Climbing of tower cranes.

These are available from <https://www.cpa.uk.net/>.

ESTA

The Europäische Schwertransport Automobilkran (European Association for the Abnormal Road Transport and Mobile Crane Rental Industry) have published some excellent guidance on the use of SPMTs, which is free to download from <http://estaeurope.eu/>.

Federation of Piling Specialists

Founded in 1964, the Federation of Piling Specialists is an industry body of specialist geotechnical and piling contractors. They have provided a number of technical documents, including guidance on the design of working platforms and recommendations for the safe handling of piling and diaphragm wall reinforcement cages. These are available from <https://www.fps.org.uk/>.

Health and Safety Executive

The Health and Safety Executive (HSE) produce a number of guidance booklets on aspects of construction that are applicable to bridge construction.

ICE

ICE occasionally publishes articles on temporary works in its proceedings. In addition, these books will be of value.

Bridge Deck Erection Equipment: A Best Practice Guide

Section I. Definitions, management and safety

1. Glossary of terms and abbreviations
2. Classification of bridge decks
3. Project management
4. Safety

Section II. Bridge deck erection equipment

5. Precast balanced cantilever erection by launching gantry
6. Precast segmental span-by-span erection by launching gantry
7. Precast girder erection by launching gantry
8. Precast full span erection by launching gantry
9. Cantilever erection by lifting frames
10. Progressive cantilever method using stay cables
11. In situ form travellers
12. Movable scaffolding systems
13. Incremental launching method
14. Prefinished bridge installation methods

Section III. Design aspects and precasting

15. Detailing aspects of bridge decks built by BDE

16. Equipment design
17. Mechanical, hydraulic and electrical components
18. Lifting attachments
19. Precasting of bridge decks built by BDE

Section IV. Recommendations for the industry

20. Recommendations for the industry

Temporary Works: Principles of Design and Construction

For general information on all aspects of the temporary works, this book is recommended. The second edition covers:

1. Safety, statutory and contractual obligations
2. Management of temporary works
3. Site compounds and set-up
4. Tower crane bases
5. Site roads and working platforms
6. Control of groundwater
7. Lime and cement stabilisation
8. Jet grouting
9. Artificial ground freezing
10. Slope stability in temporary excavations
11. Sheet piling
12. Trenching
13. Diaphragm walls
14. Contiguous and secant piled walls
15. Caissons and shafts
16. Bearing piles
17. Jetties and plant platforms
18. Floating plant
19. Temporary bridging
20. Heavy moves
21. Access and proprietary scaffolds
22. Falsework
23. Formwork
24. Soffit formwork
25. Climbing and slip forms
26. Temporary facade retention
27. Bridge installation techniques
28. Backpropping
29. Pressure testing of pipelines
30. Basement construction
31. Digital project delivery – visual planning and BIM
32. Rebar stability
33. Needling and forming openings in walls
34. Temporary works in demolition

Institution of Structural Engineers (IStructE)

The Temporary Works Toolkit, a series of 15 articles published in *The Structural Engineer* between October 2016 and September 2017, which covers general temporary works topics, may be of interest.

Temporary Works Forum

The Temporary Works Forum (TWF) was formed in 2009 at an open meeting held at the Institution of Structural Engineers in London. The TWF is an independent, not-for-profit company, which operates on a limited cost base; its primary asset is the influence of its members in industry, and its primary value derives from its meetings on a quarterly basis and its free-to-access publications (available at www.twforum.org.uk). Meetings are open (contact the secretary for an invitation) and address issues across the industry.

The TWF aims to encourage open discussion of any matter related to temporary works. The group is open to any organisation within the industry and sharing this intent.

The TWF's primary objectives with respect to temporary works are to

- give authoritative guidance and, when required, professional leadership to the industry
- consider aspects of permanent works and interfaces between permanent works and temporary works, as are relevant
- consider both current practice and likely development
- be aware of trends and innovations in design, construction and use.

The TWF publishes a number of guides on design and good practice, available from their website, including:

- TW11.031. Training template for permanent works designers
- TW12.06. Design management of temporary works
- TW13.01. Stability of reinforcement prior to concreting (October 2013)
- TW13.032. Differences between permanent and temporary works (August 13)
- TW14.01. The use of European Standards for temporary works design
- TW14.02. Clients' guide to temporary works (January 2015)
- TW15.120. Draft design of granular working platforms for construction plant (October 2015)
- *The Role and Responsibilities of Permanent Works Designers with Regard to Temporary Works*
- TW16.106. Principal designer: guidance on temporary works (January 2017)
- TW17.037. Principles for the management of temporary loads, temporary conditions and temporary works during the construction process

- TW19.01. Precast concrete: good practice and common issues in temporary works
- TW19.02. Working platforms – design of granular working platforms for construction plant: a guide to good practice
- TW19.03. Temporary works procedure: sample
- TW20.01. Effective management of scaffolding to BS 5975:2019
- TW20.02. Constructability: a guide to reducing temporary works
- TW20.03. Rebar guide. Part 1: management (reinforcement stability)
- TW20.112. Constructability guide checklists
- TW21.045. An introduction to BIM.

The TWF works closely with the HSE and Structural-Safety Ltd (<https://www.cross-safety.org/uk>). The HSE attend all TWF meetings and some members of TWF are also members of SCOSS (the Standing Committee on Structural Safety, which is a committee of Structural-Safety Ltd.). The TWF also promotes the use of CROSS (confidential reporting on structural safety), in a scheme established by Structural-Safety Ltd.

National Association of Scaffolding Contractors

The National Association of Scaffolding Contractors (NASC) were established in 1945. They produce a number of guidance documents on the supply and use of scaffolding. TG20 is their guide on the design of scaffolds constructed from tube and fitting. Publications are available from <https://www.nasc.org.uk/>.

Proprietary suppliers

There are a large number of suppliers of proprietary equipment, who will provide guidance and product literature to assist with temporary works for all aspects of the construction of bridge substructures and superstructures.

US Department of Transportation Federal Highway Administration

There are a number of excellent publications from America relating to accelerated bridge construction. They have benefited from much of the work that has gone on in Europe. Publications on bridge launches, bridge slides and SPMTs are available from <https://www.fhwa.dot.gov>.

Chapter 20

Bearings

Kennedy Reid Atkins
Rob Wheatley Atkins

Design considerations relating to bearings are described with diagrams for articulation, loading, fixity, restraint, movement and rotation. Types of bearings are also described and comments made relating to the choice of bearings. The installation and bedding of bearings is detailed and illustrated for different types of deck, while mention is made of inspection and maintenance for different bearing types. Bearing replacement is treated in some depth, dealing with jacking strong points, replacement sequence, jacking load, articulation, jacking operation, bearing removal and installation, grouting and de jacking. Twenty-three diagrams and nine photographs follow, illustrating both problematic and good practice.

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Design

Introduction

Bridge bearings provide a means of transferring loads between the superstructure and substructure of a bridge, while accommodating or controlling the articulation.

Articulation

Bridge decks are subject to translational and rotational movements and to forces from gravity, traffic, wind, seismic action and friction. In supporting bridge decks, bearings must cater for these forces and movements. Translational movements principally arise from temperature changes, creep, shrinkage and prestress. Rotational movements principally arise from dead and superimposed loading and from traffic loading. However, as the deck bends under loading, the bottom chord extends, resulting in additional translational movements on the bearing. Conversely, differential temperature changes through the depth of the deck and parasitic effects of prestress cause additional rotational movements at the bearings. Seismic loading can induce cyclical movements in all six degrees of freedom.

Loading

Traffic, dead and superimposed loads are distributed to the deck supports, resulting in different vertical loads on the bearings, while braking, traction, skidding and parapet impact apply principally horizontal forces. Wind can apply horizontal and vertical forces; at times the bridge might be subject to seismic, snow, flood and impact loading. The bearings themselves provide resistance to movement, resulting in frictional forces. Unless the sliding surfaces of the bearings are set horizontally, the dead weight of sloping decks can result in horizontal forces or sliding. Any significant movement at the abutment of a

sloping deck with horizontal sliding surfaces can result in a step at the expansion joint.

Fixity

Single-span decks traditionally have fixed bearings at one support and sliding bearings at the other, enabling horizontal forces to be transmitted through the fixed bearings and the expansion movement to be accommodated at the sliding bearings. Alternatively, the deck may be allowed to float, being supported at each end on elastomeric bearings, which have sufficient shear stiffness to transmit horizontal forces and, at the same time, adequate flexibility to accommodate temperature movements. For shorter spans, the deck can be fixed at both ends, expansion being accommodated by compression of the deck or flexure of the abutment and compression of the fill behind the support. The advent of integral bridges enables longer decks to be accommodated in a similar manner. Semi-integral bridges tend to have sliding bearings at each end support and a fixed bearing at the central pier; if they are single-span bridges, the deck may float, as noted previously. The build-up of lateral earth pressure on integral abutments owing to cyclic thermal movement will result in flexure of the central pier in order to reach overall equilibrium. This equilibrating force must pass through the fixed bearing. Fixing one end of a single-span semi-integral bridge will concentrate the lateral earth pressure from one side onto the fixed bearings.

Restraint

Under repetitive transverse movements, decks can creep or 'walk' sideways and restraints are required. However, if bearings are set some distance apart, such that their restraints oppose each other, changes in temperature can lead to the generation of very high forces. In a single-span deck that is neither floating nor integral, it is common to fix the bearings along

one support in the longitudinal direction, with only the central bearing or bearings fixed transversely, while the bearings on the other support are free to move in both directions, with only the central bearing or bearings guided longitudinally (i.e. restrained from transverse movement). This restraint must be designed to prevent the deck rotating in plan about a vertical axis under eccentric braking, skidding or parapet impact forces (see Figure 20.1).

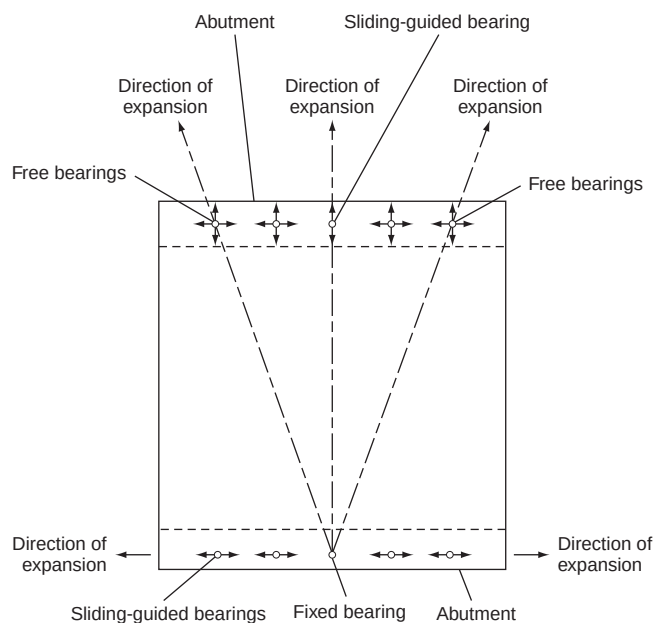


Figure 20.1 Plan on deck (reproduced with permission © Atkins)

All these bearings would be provided with rotational capacity about an axis parallel to the support, although on skew bridges rotational capacity about both axes is often required. On large plate girder bridges, where adjacent deep girders might be subject to very different loads and therefore different rotations, the bearing restraint forces preventing the bottom flanges of the adjacent girders from moving longitudinally in opposite directions can be sufficient to cause bursting failure of the abutment face (see Figure 20.2). Similar effects can occur under uniform loading on skew decks.

Where a shallow deck has a line of sliding guided bearings along one support, each providing restraint to longitudinal movement, the tolerance gap between the bearings and their guides can mean that the longitudinal loads will not be spread uniformly between the various bearings; this needs to be taken into account in determining the capacity of these restraints.

Multispan structures

The bearings are usually fixed on a strong support capable of carrying longitudinal loads near the middle of the structure, with the remaining bearings free to move longitudinally to accommodate temperature movements in either direction away from the fixed pier towards the ends of the viaduct. This may not be suitable with river valley crossings, as the highest piers may be in the middle and, owing to their flexibility, inappropriate for carrying the longitudinal forces. Alternatively, the flexibility of tall and slender piers may be exploited by the design to enable the longitudinal loading to be distributed between a number of supports. Fixed bearings installed on adjacent slender piers will share the longitudinal loads and accommodate thermal movements by flexure. The piers must

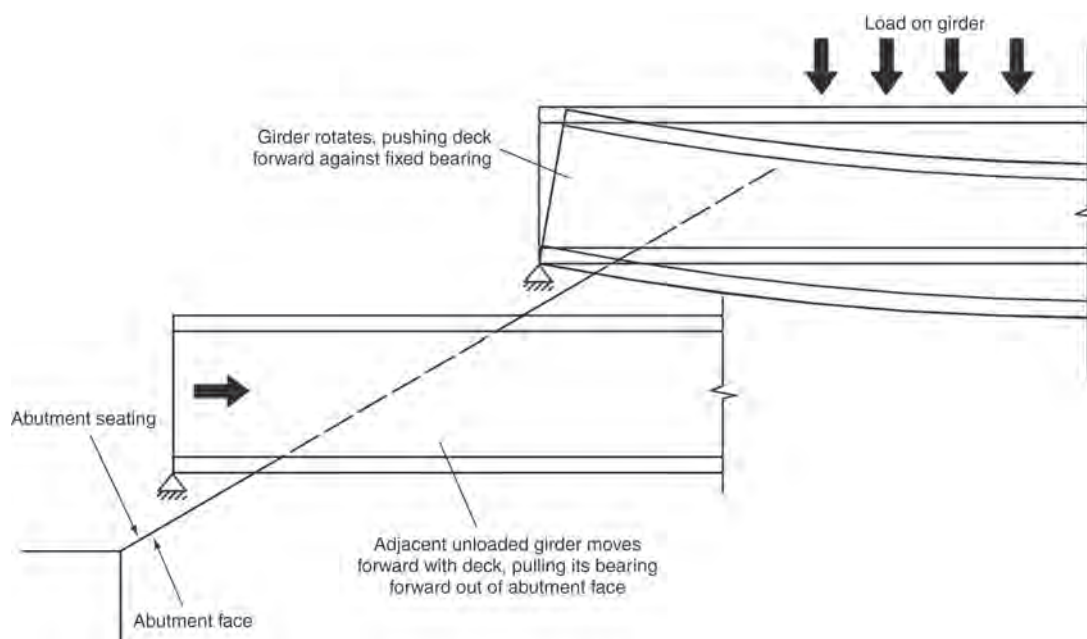


Figure 20.2 Prying action between deep deck beams (reproduced with permission © Atkins)

also accommodate the resulting eccentricity of vertical loading. If any deck is inclined longitudinally, it is usual to install the fixed bearing at the lower end.

Temperature movements

A bridge deck expands in all directions from the fixed bearing point. The resulting 'star' of movement indicates the direction of the guides on the sliding-guided bearings and demonstrates the need for free-sliding bearings elsewhere (see Figure 20.1).

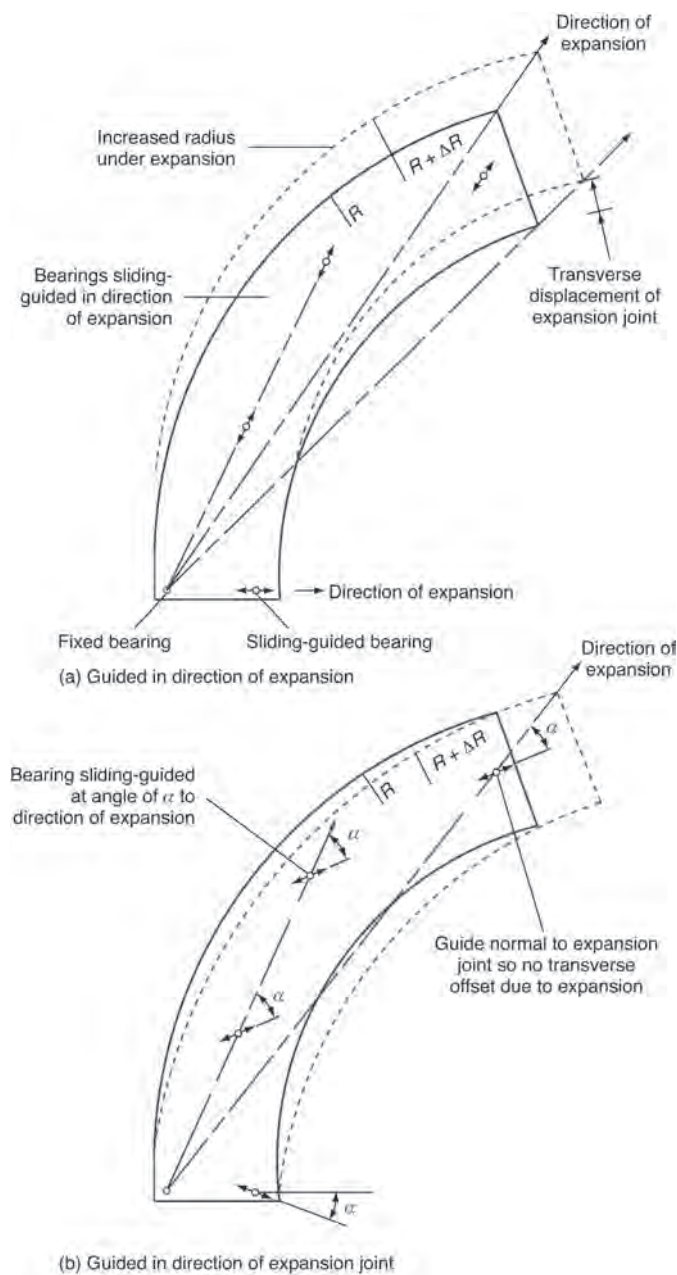


Figure 20.3 Short, wide, curved deck (reproduced with permission © Atkins)

In expanding in this way, curved decks increase their radius. Short curved decks need to have free-sliding bearings or sliding-guided bearings following the movement star on succeeding piers (see Figure 20.3). Long, narrow, curved decks, however, can be 'trained' to follow their initial radius in their expanded form by using tangentially guided bearings on each pier (see Figure 20.4). In this case, the guides must be designed to cater for the transverse forces required to bend the deck in a horizontal plane to maintain the same radius during temperature changes.

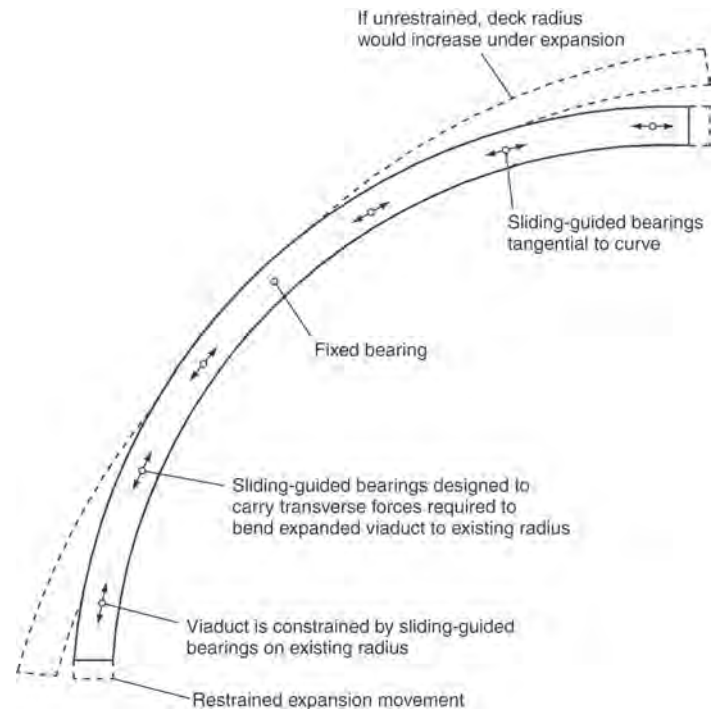


Figure 20.4 Long, narrow, curved deck (reproduced with permission © Atkins)

Rotation

To avoid the deck sliding longitudinally or transversely, the sliding surfaces should be set horizontally. However, a rotational axis is required along each support; with longitudinal and transverse falls, horizontally set bearings will provide a stepped axis at each support. For solid slab decks, this can be accommodated by spherical or elastomeric rotational elements in each bearing, but it could cause problems with longer rocker bearings.

Types of bearings

Bearings can be elastomeric, pot, spherical, cylindrical, roller, rocker, knuckle, leaf, guided, pendulum or restrained. Many bearings can be used in conjunction with a sliding element (see Figure 20.5).

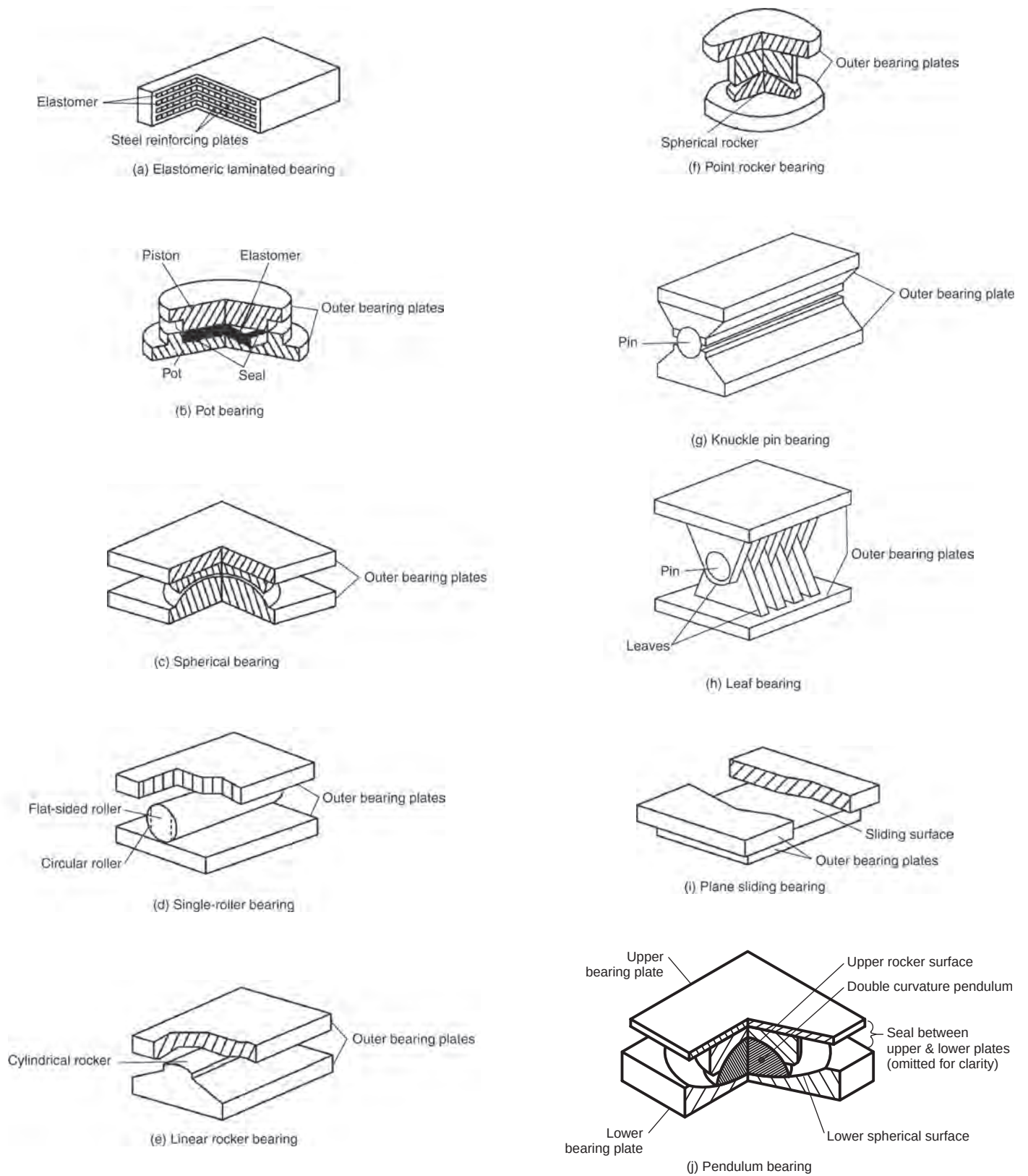


Figure 20.5 Types of bearing (reproduced with permission © the Highways Agency and Atkins)

Elastomeric bearings

Elastomeric bearings can be vulcanised rubber or neoprene. The material is essentially incompressible volumetrically, but an elastomeric pad bulges sideways under vertical pressure and so provides elastic support and rotational capacity. In addition, elastomeric pads have shearing movement capacity and so can accommodate horizontal movement. By inserting horizontal steel plates spaced within the height of the pad, a laminated bearing is formed. This restricts the bulging on the sides and provides a stiffer bearing under vertical load, while accommodating similar shearing movement capacity. The rotational capacity of an elastomeric bearing is limited to the compression applied by the minimum vertical load to avoid uplift at the edge and is also limited by the maximum compression. The shearing resistance must be taken into account in the design of the supports.

Pot bearings

A pot bearing consists of a metal cylinder containing an elastomeric pad compressed by a piston engaging with the cylinder. The engagement of the piston can transmit transverse loading. Alternatively, the pot bearing can be used in conjunction with a sliding element. The confined elastomer acts as an incompressible fluid and permits significant rotation for a given minimum vertical load, combined with a significant capacity for vertical loading. To enable the piston to rotate in the cylinder, the elastomer is retained in the cylinder by seals. These determine the rotational fatigue life of the bearing. Particularly where pot bearings are being used as replacements on an existing structure, it is important to consider that they do compress to some extent under initial vertical load and slowly recover when unloaded.

Spherical bearings

Spherical bearings consist of metal convex and concave surfaces sliding on each other, one face usually being covered with polytetrafluoroethylene (PTFE) and the other being hard chromium plating, austenitic stainless steel or aluminium alloy. The PTFE face is usually dimpled to assist the retention of silicon grease used as lubricant. Spherical bearings provide for rotation about any horizontal axis without significant compressibility. Some restraint to transverse force is provided by the inclination of the spherical surfaces, but positive or significant horizontal restraint must be provided by guides or restraints. Spherical bearings can be used in conjunction with a plane sliding element to permit translation.

Cylindrical bearings

These are similar to spherical bearings, with rotation about a single axis.

Roller bearings

These consist of a curved metal unit rocking on a metal plate, providing only rotational movement. They can provide for rotation about either or both horizontal axes and are frequently used in conjunction with a sliding element. The rocker interface is dowelled to prevent slippage and to transmit horizontal forces.

Knuckle bearings

These are hinged bearings that rock on a pin joint, but they are no longer commonly used.

Leaf bearings

These are interleaved pin bearings, which can take tension as well as compression. Rotation is only permitted about a single horizontal axis.

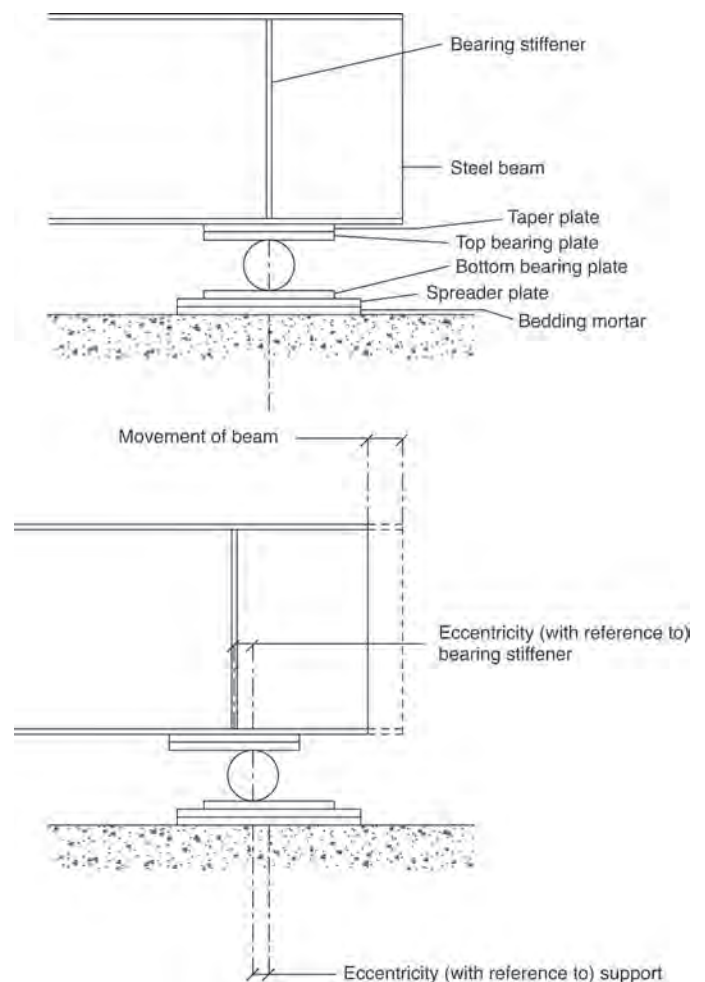


Figure 20.6 Elevation on roller bearing, showing eccentricity on both beam and support as the roller moves (reproduced with permission © Atkins)

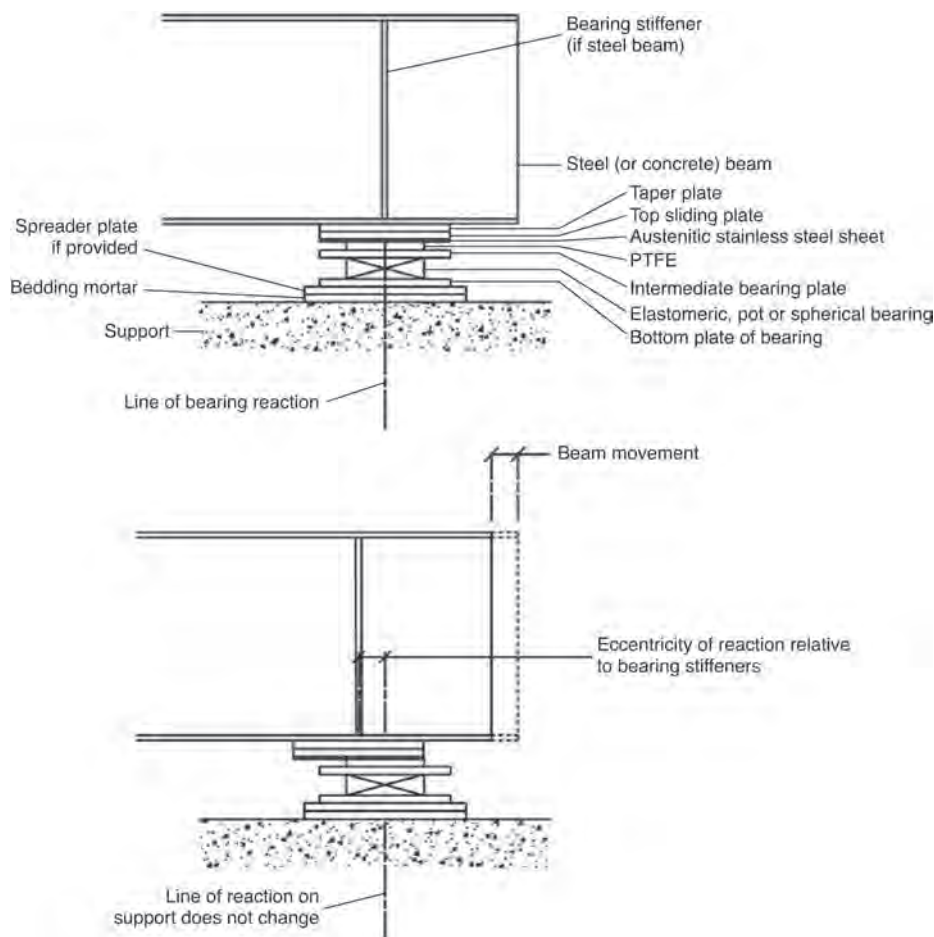


Figure 20.7 Elevation on sliding bearing, showing eccentricity on the beam as the beam moves (reproduced with permission © Atkins)

Guided bearings

These are bearings fitted with sliding guides, which direct movement in a particular direction. If they do not transmit vertical load, they are referred to as *guide* bearings (as distinct from *guided*). They can, if required, accommodate rotations, and the guides are capable of transmitting horizontal loads. The sliding material, in accordance with EN 1337 (BSI, 1998a, 1998b, 2000, 2003, 2004a, 2004b, 2006a, 2006b, 2006c, 2006d, 2009) may be CM1, which is a composite material consisting of three layers – a bronze backing strip and a sintered interlocking porous matrix, impregnated and overlaid with a PTFE–lead mixture – or CM2, which consists of a flexible metal mesh that is sintered into a PTFE compound, with the bearing or sliding surface having the thicker PTFE coat. They generally slide against an austenitic stainless steel sheet and give better wear resistance but have a higher friction than the PTFE used for plane sliding surfaces.

Sliding elements

These are plane sliding surfaces, similar to those used for spherical and cylindrical bearings but with lubricated, dimpled PTFE sliding on austenitic steel sheet.

Choice of bearings

As a general guide for shorter-span bridges, and particularly those with concrete decks, where rotations are small, elastomeric bearings are economical. For medium spans, and more particularly for steel–concrete composite bridges, pot bearings are appropriate and economical. For longer spans, for greater vertical stiffness or to accommodate greater rotations in relation to the vertical load, spherical bearings are used. For longer spans where very low friction is required, roller bearings might be used, but see the caution under the section on roller bearings. For rotation about a single axis, where high vertical loads must

Friction pendulum bearings

By combining the principles of both spherical and point rocker bearings, a special variation can be created, called the friction pendulum bearing. This is designed as an isolation bearing for use in seismic protection of the supported structure. The bearing comprises two to four concave sliding surfaces, depending on the need to absorb earthquakes of differing magnitude, with the more complex bearings referred to as triple pendulum bearings (i.e. having three increasing levels of pendulum response). The bearing is designed so that any lateral displacement will mobilise the vertical weight to continuously return the bearing back to its lowest energy position, at the bottom of the pendulum swing. This effectively recentres the bearing. The bearings have a much greater capacity than elastomeric bearings, which can also recentre a structure following lateral displacement.

Restrained bearings

These provide fixity against horizontal movements and can accommodate rotations. If they do not transmit vertical load, they are referred to as restrain bearings. Fixings for guide or restrain bearings are required to resist axial tension and shear and may need to be significantly larger than those for bearings with vertical loading.

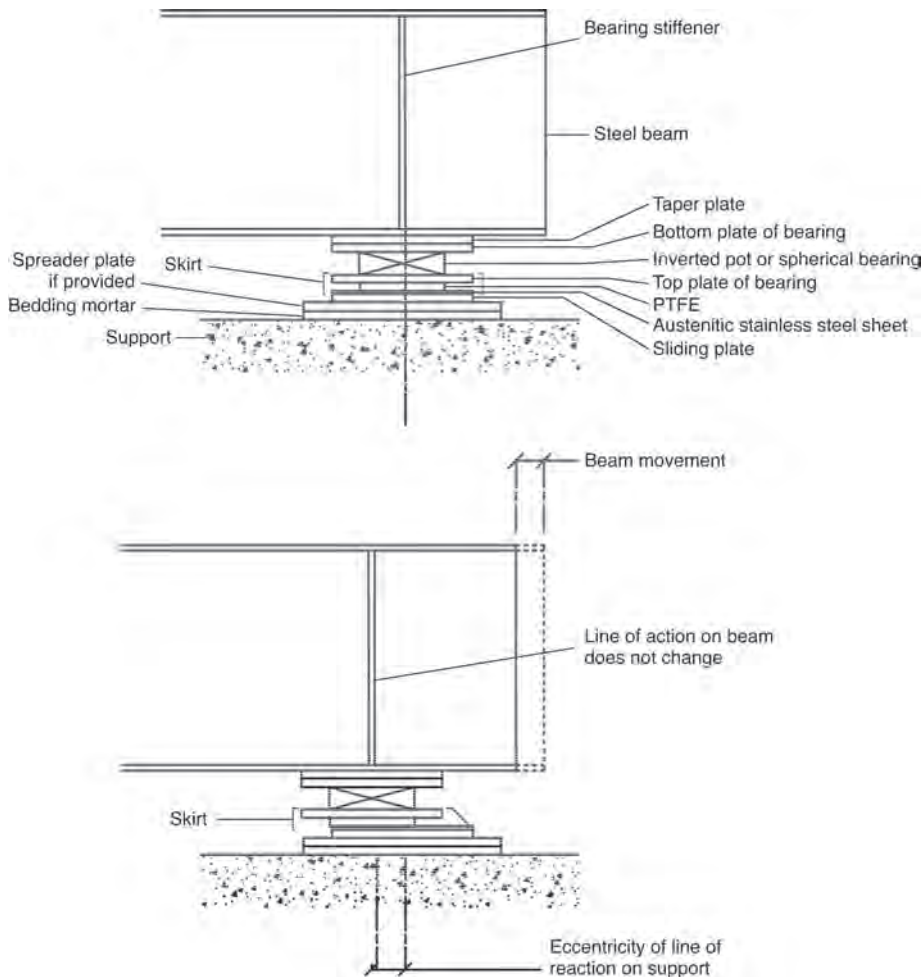


Figure 20.8 Elevation on inverted sliding bearing, showing eccentricity on the support as the beam moves (reproduced with permission © Atkins)

be accommodated, rockers might be used. Where restraints in the form of downstands would not provide adequate capacity, separate guide or restrain bearings may be used. Leaf bearings may be used to transmit uplift forces.

Eccentricity

The line of action of vertical load moves as the bearing slides or rotates, resulting in an eccentric reaction on either the deck or the support, or, in the case of rollers, both (see Figures 20.6 and 20.7). In the case of a steel beam, a reaction eccentric to the bearing stiffeners can cause problems; for this reason, pot or rocker bearings may be installed inverted, with the pot or rocker fixed to the beam and sliding on a plate attached to the support. The support must then be designed to accommodate the eccentric reaction at the limits of travel, and a skirt is required to keep the sliding surface free of detritus (see Figure 20.8). However, as skirts can easily become damaged or might

not be replaced after inspection, it is preferable to design the bearing stiffeners for the eccentricity and not invert the bearing. With roller bearings, an eccentricity of half the movement is applied to both the deck and the support.

Replacement

Bearings should be installed so that they can be replaced with another suitable bearing if and when required. The method of installation needs to be carefully planned during the original design of the structure. A decision needs to be made as to what live loading the structure will be carrying during jacking and bearing replacement. Provision for jacking to enable replacement requires

- adequate clearance between the support and the deck for jacks of adequate capacity with locking ring facility to be inserted
- sufficient space on the support adjacent to the bearing for jack placement
- the support and deck to be strong enough to carry the jacking loads
- sufficient room for the replacement bearing to be installed by sliding the old bearing out and the new one in, with minimal lifting of the deck
- anchor points provided to which temporary restraints can be attached to replace fixity lost during bearing replacement.

It may also be necessary to provide temporary bearings in conjunction with the jacks to permit translational or rotational movement during replacement.

Bearing schedule

The bearing schedule completed by the designer enables a suitable choice of bearing to be made. The schedule must detail; maximum and minimum, ultimate and service, vertical and horizontal loads, as well as the maximum longitudinal, transverse and rotational movements to which the bearing will be subjected. Coexistent, permanent and variable loads and movements must also be defined. For example, the design of an elastomeric bearing needs to ensure a minimum vertical load at maximum rotation to avoid lifting off along one edge, and the information provided in the bearing schedule must ensure that a bearing will be adopted such that this will not happen. Bearing schedule pro formas are provided in the standards.

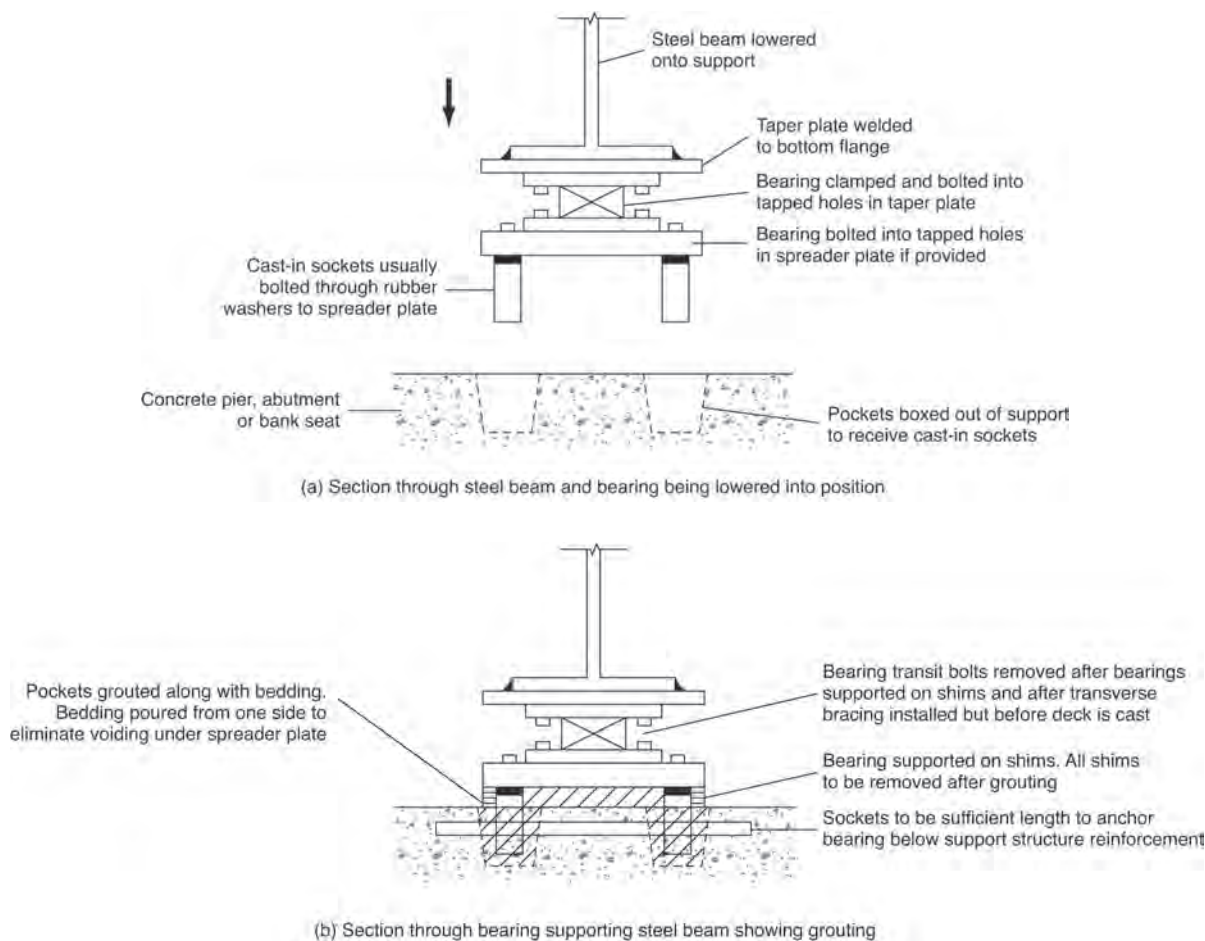


Figure 20.9 Section through bearing supporting steel beam (reproduced with permission © Atkins)

Electrolytic action

Different metals must be insulated from each other to prevent bimetallic corrosion. Consideration should also be given to detailing of the component parts to prevent moisture creating an anodic connection between dissimilar metals.

Installation

Introduction

The methods of installation suggested here are for general guidance only. The engineer needs to be satisfied that the details are appropriate for the particular situation.

Transport

Care must be taken in the handling and transport of bearings so that they are not damaged. Transit brackets (which should be painted in a contrasting colour) prevent the bearings separating, sliding or rotating and retain compression in preloaded PTFE.

Storage

Bearings need to be carefully stored in a clean dry environment at a moderate temperature and inspected for damage prior to installation. Under no circumstances should bearings be dismantled, otherwise sliding and other surfaces can be permanently damaged by dirt.

Steel beam deck

It is common to attach the bearing to the steel beam before lifting in the beam (see Figure 20.9). Elastomeric bearings can be attached with adhesive, and other forms of bearings by bolts. To accommodate longitudinal fall, taper plates may be welded or bolted to the deck beam and the bearing bolted to the taper plate. Spreader or spacer plates may be included in a stack of bolted plates, but all plates need to be machined and installed to tight tolerances to ensure that the load application to the bearing is even and that the sliding surfaces can be set horizontally. The taper plates should also accommodate the beam precamber and the beam, slab and superimposed dead weight rotations, allowing for creep (by long-term Young modulus, E)

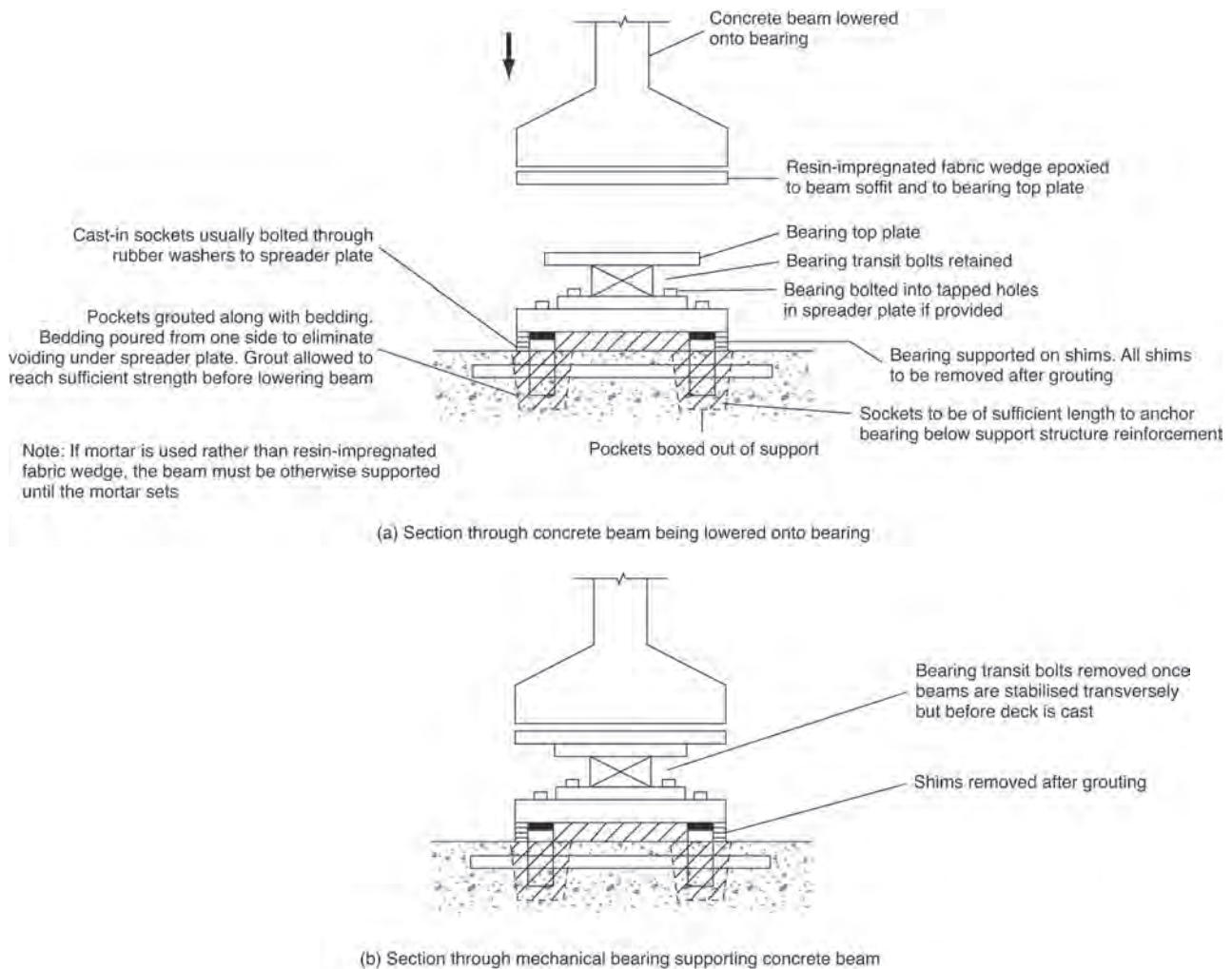


Figure 20.10 Section through mechanical bearing supporting concrete beam (reproduced with permission © Atkins)

and shrinkage of the slab, so that, in the long term, the bearing sliding surfaces are horizontal. The designer needs to determine the extent to which these calculations are required, depending on the span and scale of the bridge.

Precast concrete beam deck

It is common to install the bearing on the support on shims and grout underneath the bearing. Once the grout has reached sufficient strength, the beam is lifted onto the bearing, either on a wedge of wet mortar or on a thin layer of epoxy on a resin-impregnated fabric wedge, itself epoxied to the bearing top plate (see Figure 20.10). Where mortar is used, the beam needs to be supported until the mortar hardens, to prevent extrusion of the mortar, attraction of the load to the touching end of the bearing and consequent eccentric load on the bearing. As with the steel beam, the resin-impregnated wedge should accommodate longitudinal fall (and, where applicable, cross-fall), beam precamber, and beam, slab and superimposed dead weight rotations, allowing for the creep and shrinkage of the

beam and slab so that, in the long term, the sliding surfaces are horizontal. Again, the designer needs to determine the extent to which these calculations are required. A mortar wedge will only accommodate longitudinal fall, beam precamber and beam self-weight rotation. For elastomeric bearings, see Figure 20.11. Elastomeric bearings can transmit horizontal forces through friction but more often require a positive means of location, in accordance with EN 1337. Resin mortars are often used to provide a positive means of location.

In situ concrete deck

The slab is cast over the bearing, complete with cast-in sockets bolted against the top plate, and carefully sealed around the edges to prevent grout leakage reaching the bearing below. The bearing is grouted onto the support before casting the slab and the sliding surfaces are set inclined so that after striking the deck falsework, and once creep and shrinkage have taken place, the sliding surfaces are horizontal. Downstands are

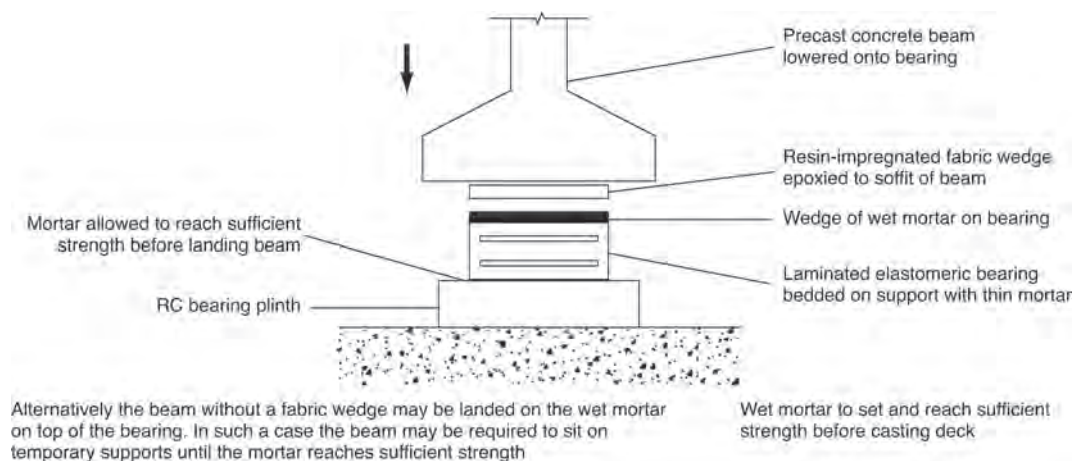


Figure 20.11 Precast beam on elastomeric bearing (reproduced with permission © Atkins)

required beneath the concrete deck to eliminate longitudinal and crossfalls so that the bearings can be replaced (see Figure 20.12). For elastomeric bearings see Figure 20.13.

Transit straps

These are for transport and storage. They are not designed to restrain the bearing under the dead weight of the deck beams or deck slab and should be removed before the bearing has to undergo rotation or translation. Other means of support, such as wedges, are required to restrain the bearing during deck placement. There are, unfortunately, cases of bearings in service with the transit straps still fastened.

Bedding

Pockets are normally cast in concrete supports, which provide positional tolerance for cast-in sockets, usually bolted through

rubber washers to the underside of the bottom plate to avoid bearing on the sockets. Bolts should be greased before insertion into sockets to aid possible later removal. The bolts and sockets need to be able to carry the horizontal loading applied to the bearing, with appropriate factors of safety, although some allowance can be made for friction on the base using minimum vertical load. Except for elastomeric bearings, a positive (bolted) connection should always be provided. With the introduction of increased cover to reinforcement, the sockets now need to be longer to engage fully with the reinforcement mat in the bearing plinth, abutment, bank seat or pier top. The design of the sockets and fixing bolts must allow removal of the bearing for replacement without damage to the supporting structure. Rusted bolts can require considerable torque to free up and it is good practice to prevent sockets from slipping at the interface with the concrete when trying to undo bolts. Sockets can be specified as hexagonal or have a tang added to engage with the surrounding concrete.

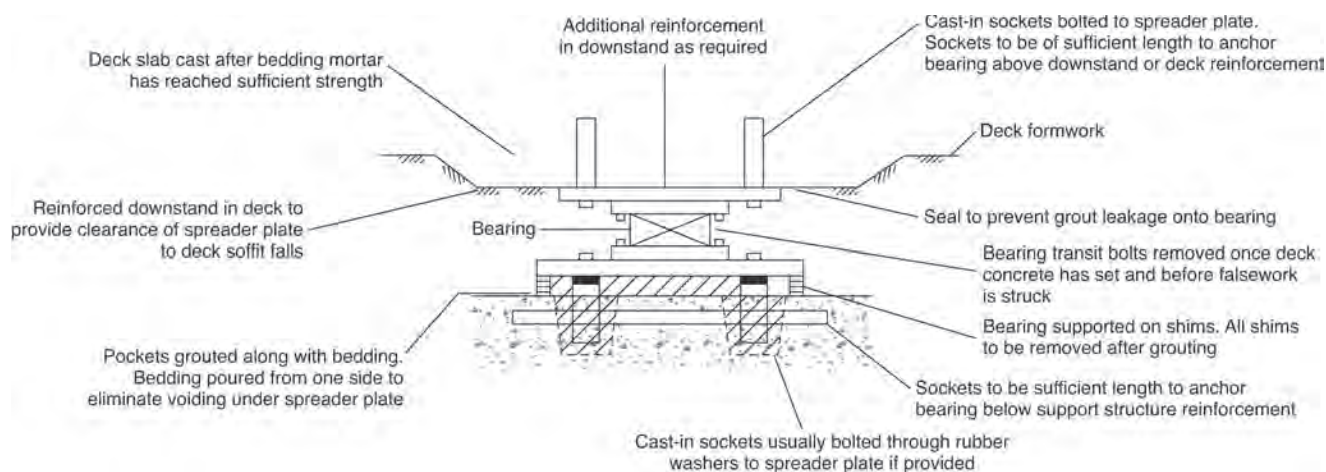


Figure 20.12 Section through mechanical bearing showing grouting preceding casting of concrete deck slab (reproduced with permission © Atkins)

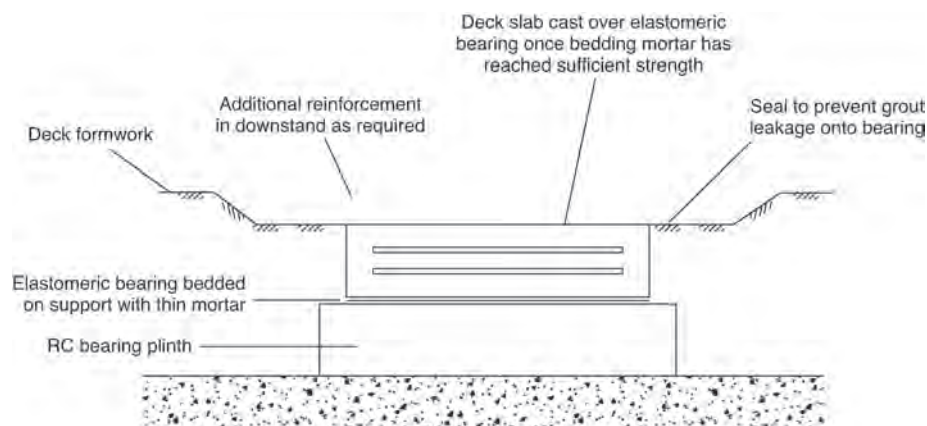


Figure 20.13 Section through elastomeric bearing below concrete deck slab (reproduced with permission © Atkins)

With mechanical bearings, the bottom plate is supported on steel shims to carry the weight of the deck beam. Once the deck beam has been landed, the bearings, together with the pockets, are grouted up. After the grout has hardened, the shims are removed to avoid hard spots under vertical loading, the holes made good and the holding-down bolts tightened and locked as required. The composite deck slab or slab deck can then be cast. Elastomeric bearings attached to deck beams can be lowered onto a bed of mortar. Alternatively, the elastomeric bearing can be installed on the support and the beam lowered onto wet mortar on top of the bearing. In either case, the beam must be supported until the mortar hardens. With slab decks, elastomeric bearings are first bedded on mortar, which is allowed to harden before casting the deck.

Specification, workmanship and installation of the grout or mortar are extremely important to control shrinkage or expansion and compressibility, to ensure that voids do not remain between the bearing and the structure, and to ensure sufficient strength. The grout needs to be poured from one side, retaining a head, to eliminate air. It is preferable to test first against a glass plate mock-up to ensure this. The grout plinth needs, initially, to extend at least 50 mm beyond the bearing plate for stability and should be deep enough to enable the grout to flow freely. The grout should be trimmed back vertically to the face of the base plate to prevent spalling. The Highways Agency's (1992a) *Notes for Guidance on the Specification for Highway Works 2600 Series* limit the height of the grout, for stability under load and to limit settlement.

Protection

A final coating is normally required following installation, in accordance with manufacturer's instructions. This should include all exposed surfaces of the stack of bearing plates. Before applying site coats of paint, sliding surfaces must be masked or protected.

Inspection and maintenance

Inspection

Inspection of a bearing can indicate not only whether it is in good condition but also whether it is accommodating rotational and translational movements of the structure, and sometimes whether settlement or distress, such as impact damage, has occurred to the structure. Reference should be made to previous inspections to highlight areas of concern and gauge rate of deterioration. The type of bearing should be identified, together with any design limitations or assumptions, to assist in taking specialist tools and in identifying problems in situ.

General items

General items of inspection are

- the presence of dirt or debris and water ponding on or around the bearing
- any cracking or distress to the adjacent parts of the structure
- whether the holding-down bolts are loose or corroded
- whether the transit bolts have been removed
- whether the protective treatment is cracked or peeling or leading to corrosion
- whether the bedding is sound or cracked or decaying
- whether there is any indication of plate bending, indicative of voiding in the bedding or of uneven pressure distribution on the deck beams
- the condition of the skirt
- whether there is provision for future replacement, such as space for jacking, strong points for jacking or a special design of bearing that might be difficult to remove.

Checks should be made that bearings are not supported on hard points, such as shims not removed after installation of bedding. An endoscope with light attached can be used to inspect inaccessible areas. During detailed inspection, the bearing movements and rotations should be recorded, together with clearance gaps.

Elastomeric bearings

These should be inspected for excessive shearing or bulging (limited bulging under a vertical load is normal), or cracking or splitting, particularly at the intermediate plates.

Sliding bearings

These should be inspected for the condition of the sliding surface – dirt, debris, corrosion, lack of lubrication, bowing of the austenitic stainless steel plate – and whether there is any indication of seizure or insufficient sliding or of sliding to the limit of travel. The swept area of the sliding surface will be cleaner and shinier than the unswept area. The condition of any skirt or protective cladding should be recorded, and the protrusion of the PTFE measured using a feeler gauge around the perimeter at corners and sides. Where necessary, skirts will need to be removed during inspection, but they must be replaced.

Guides

Guides need to be inspected for wear, excessive clearance and satisfactory sliding operation.

Restraints

Restraints need to be checked for fixity and lack of movement.

Spherical bearings

On spherical bearings, the protrusion of the PTFE should also be noted, and any indication of seizure or of the rotation capacity of the bearing being taken to its limit recorded.

Roller bearings

These should be inspected for dirt, debris, corrosion, cracking, misalignment, lack of uniform bearing, seizure or excessive travel, or excessive rotation of the top plate relative to the bottom plate.

Rocker bearings

Rocker bearings and their plates should be inspected for dirt or debris under the rocker, corrosion, cracking or slippage. The end of the rocker can indicate the amount of rocking taking place.

Pot bearings

The seals of pot bearings should be inspected for extrusion of the elastomer and for proper engagement and rotation of the piston in the pot.

Bearing maintenance

Maintenance can involve keeping the bearing free of dirt and debris; keeping sliding surfaces clean, polished and lubricated; rubbing down protection, removing corrosion and repainting; repairing or replacing skirts and cladding; ensuring that holding-down bolts are kept corrosion free, locked tight, and greased; touching up bedding mortar and, where necessary, rebedding. (For details, see the section on replacement.) Only localised touching up of mortar under the edge of a bearing plate is recommended; otherwise the lack of even support is unacceptable. Care must be taken to ensure that grit or other debris does not penetrate beneath or through bearing covers during repainting, as this could damage the sliding surfaces or restrict intended movement.

Need for replacement

Where cracks in elastomeric bearings have penetrated to the plates, corrosion could begin to damage the integrity of the bearings, necessitating their replacement. Spherical or sliding bearings need to be replaced or refurbished if the protrusion of the PTFE becomes minimal, while pot bearings need to be replaced or refurbished if the seal is broken. The reason for the PTFE wear or broken seal needs to be determined and corrected, otherwise the process might be repeated with the replacement bearing. Bearings with cracked metal parts must be replaced.

Stick-slip

If the sliding surfaces have become worn, bearings can exhibit a stick-slip mechanism, whereby excessive force is required to make the bearing slide and it moves some distance, relieving much of that force, before stopping and sticking again. The forces generated can significantly exceed those for which the structure has been designed, with potentially serious consequences for the structure.

Hysteresis loop

Even without the extreme of a stick-slip condition, normal limiting static and kinetic friction means that forces have to build up under temperature changes before the bearing slides; subsequently, reverse forces are generated before it slides back again when the temperature reverts. In terms of a graph of force against movement, the hysteresis loop thus formed is key to an understanding of the satisfactory performance, or otherwise, of existing bearings in service (see Figure 20.14).

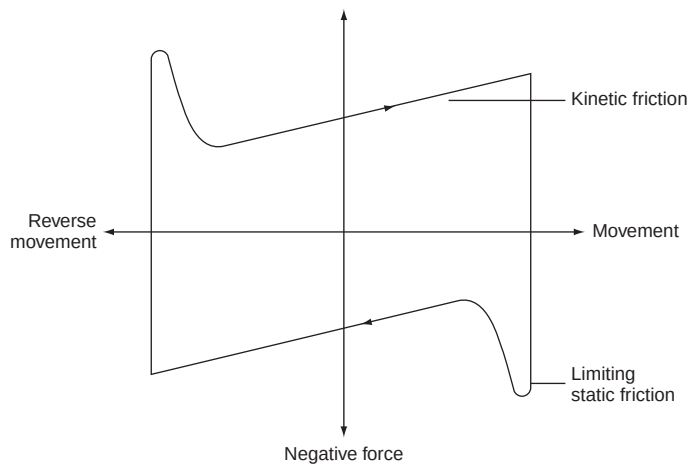


Figure 20.14 Hysteresis loop (reproduced with permission © Atkins)

Replacement

Jacking strong points

The key to the replacement of bearings is a successful jacking operation. If strong points are already provided, they need to be checked for capacity. If not, the structure capacity needs to be reviewed and provision made for the jacking. The supports need to be checked for bursting forces under the proposed jacking positions and, if inadequate, spreader plates must be provided, clamps or straps fastened or anti-bursting reinforcement cast in. If jacking against a slab deck, the deck should be similarly checked. If jacking against a concrete box girder, the diaphragms may require strengthening, if only because the jacks are supporting it in a different location from the bearings. Similarly, a steel box girder diaphragm might have to be strengthened or braced. Steel beams may be lifted using an A-frame supporting bracket engaging either side of the top flange–web welds, although these welds might need to be enlarged and the beam web above the bottom flange will need to be temporarily propped for transverse stability. Brackets might need to be bolted or welded to larger girders, in which latter case, ultrasonic testing might be required to check for laminations to avoid lamellar tearing, precautions taken to mitigate weld distortion, and load restrictions applied during welding to avoid overstressing the heat-affected zone. Concrete beams and slabs may have to be lifted on frames in front of the pier or abutment supported on the spread footings or pile caps.

Replacement sequence

Manufacture of the bearings may take months, so programming is essential. Depending on the structure type and details, replacement of bearings can take several days, or even weeks. Once access has been provided, together with any temporary works or permanent modification, the process consists of

- jacking to release the load on the permanent bearing
- removal of the original bearing and bedding (which might require additional jacking)
- removal of existing sockets (sometimes)
- preparing surfaces for new bearings
- setting the deck to level
- installing the new bearing
- grouting to rebed the bearing
- transferring load to the new bearing
- checking the installed bearing for compliance or tolerance.

Jacking load

Since the thickness of a concrete slab can vary from the design dimensions, it is not easy to ascertain how much jacking load is required to lift the deck or deck beam off the bearing. Regardless of whether a single beam or the full width of the deck is being lifted, the transverse stiffness of the deck slab, bracing or end diaphragm may result in load sharing between the bearing locations under jacking, masking variations between the dead load carried at each location. When the bearing lifts off, the slope of the graph of jacking load against vertical deflection changes. Unfortunately, this change plots as a curve rather than as a kink in the graph. The gradients on the graph below and above the curve can, however, be back-plotted to provide the kink location and thus determine the dead load at lift-off. With significant transverse stiffness, however, difficulties can still arise, and these are exacerbated by the flexibility of the structure between the jacking and bearing locations and between the position at which the vertical deflection is measured and the bearing itself.

The vertical deflections can be recorded using electronic gauges, for example, linear variable differential transformers (LVDTs), and displayed on a central console and the jacks can be controlled and their pressures measured and recorded centrally, so that the whole operation can be fully coordinated. In any event, the jacking should be carried out in stages, recording the loads and deflections and inspecting the jacks, equipment, bearings and structure at each stage. Up to lift-off, the jacking should be controlled by jack load; after lift-off, the jacking should be controlled by deflection. Loads recorded during jacking should be compared with design loads to assess the validity of design assumptions. In this way, overstressing of the deck, overloading of the jacks and excessive deflection of the structural elements can be avoided. Extensive and detailed calculations may be required in advance of the jacking to determine tolerable variations in deflections and loads and avoid structural overstress. Decisions must be made on the amount, if any, of the live load permitted, either during jacking (which is not recommended) or once the structure is supported on locked-off jacks. Only small vertical deflections may be tolerated by the expansion joint, typically 3 mm.

Articulation

Careful consideration is required of the articulation of the structure while it is on jacks. The different location of the jacks compared with the bearings must be taken into account, particularly if the jacks are not on the same rotational axis as the bearings. The jacks might have a greater vertical stiffness than the existing bearings. Jacks with swivel heads will be required to accommodate rotation, and lubricated PTFE or austenitic stainless steel sliding plates must be provided to permit longitudinal and transverse movement. Consequently, it might be necessary to design and install temporary restraints prior to jacking, to prevent the deck walking out of position and to carry braking, traction, skidding and parapet impact loading. These are needed not only during replacement of fixed and sliding guided bearings but also to compensate for the possibility of the lifted deck partially or fully disengaging the guides or restraints of the bearings. Consideration may need to be given to supporting the deck on temporary bearings, as the swivel heads of jacks are designed for seating rather than for live load rotations.

Jacking operation

Reference should be made to EN 1494 (BSI, 2001), which provides information on jacks. Jacks, pumps, pressure relief valves, pressure gauges and deflection gauges need to be tested and calibrated before use. The jacks, pumps, gauges and hoses must be capable of lifting the sum of

- the dead and superimposed load of the deck or deck beam
- an allowance for variation in the slab and surfacing thickness
- an allowance for unexpected transverse distribution of the load on jacking
- an allowance for lateral earth pressure on the end screen of semi-integral structures
- a factor of safety
- an allowance to free the locking ring on jacking down.

For complex operations, a jacking trial can be useful to ensure that the operation will go more smoothly when the bearings are actually removed and replaced. Only jacks with locking rings should be used; these should be loosely screwed up to follow the lifting of the jack, and then tightened at the end of the operation. Assuming that traffic is to run once the jacking lift is complete, the capacity of the jack on the locking ring must be adequate to carry the live load with a suitable factor of safety. On jacking down, a significant load may be required to free the locking ring. The jacks need to have sufficient travel to accommodate

- installation clearances on the jacks
- bearing elastic deformation on unloading
- local deformations of loaded areas above and below the existing bearing and the jacks

- bearing removal clearance
- displacement owing to the relative positions of the jack and the bearing.

Bearing removal

Once the deck load is supported on jacks, with sufficient clearance for bearing removal, the bedding mortar is normally removed by water jetting. Strict safety procedures are required, owing to the lethal danger of a high-pressure water jet and flying debris. Precautions are also required to ensure that the surrounding structure and jacking equipment is protected from damage from the water jet. If the holding-down bolts are fastened in cast-in sockets, it should be possible to slide out the bearing once it has been unbolted. If the holding-down bolts are rag bolts cast directly in the concrete, either the deck must be lifted sufficiently to disengage them from the bearing plate, or else they must be sawn or flame cut.

Bearing installation

If the cast-in sockets are intact and uncorroded, it may be possible to reuse them. If time permits, the existing bearing plates can then be used as templates to drill holes in the new plates; alternatively, the existing plates, if in good condition, can be reused. Otherwise, rag bolts may have to be dug out of the concrete support and the holes enlarged for grouting in sockets, usually fastened through rubber washers to the spreader plate. The sockets must go below the reinforcement mat. Similarly, the top plate of the existing bearing can be used as a template to drill holes in the new top plate, so that it can be offered up and bolted to the existing taper plate. With precast beams, the new bearing can be mortared and clamped to the beam; with concrete slab decks, the cast-in sockets can be reused.

Once the new bearing is lifted in and fastened to the deck, it can be supported on shims for stability. If the deck is carrying live load while supported on the jacks, the newly installed bearing will have to rotate and the transit bolts need to be removed. Alternatively, the bearing may need to slide with temperature changes prior to de-jacking, also requiring removal of the transit bolts. For bearings permitting translation, it is unlikely that the new bearing will be installed at the same temperature as the original. Hence, it may be necessary to offset the upper and lower parts; this will require release of the transit brackets. This operation must be carried out with great care under close supervision by the bearing manufacturer to avoid damage or contamination to the PTFE and stainless steel sliding surfaces.

Grouting

Since the bearing must carry live load as soon as the deck is de-jacked, it is essential that no voids remain under the spreader plate on grouting. Care must be taken with the grouting specification and trials carried out under a glass plate to demonstrate that the grouting process will go smoothly. Normally, grouting is carried out from one side of the bearing. The cast-in sockets

will be grouted in as part of the process. Once the grout has set, the shims must be removed to avoid hard spots and the holes filled with high-strength shrinkage-compensated grout.

Dejacking

Dejacking can only begin once the bedding mortar has reached adequate strength and must be carried out as carefully as jacking. Detailed calculations are again required, to ensure that the intended dead and superimposed dead load are carried by the replacement bearings. Transverse distribution of load, compressibility of the stack of plates of the new bearing and the reduced elastic modulus of the young bedding mortar are all factors to be considered. Although additional jacking load will be required to enable the locking rings to be slackened, only the minimum load required to enable this should be applied. Wherever possible, this operation should be completed when the live load on the structure is at its lowest level. Again, coordination between jacks is required.

The operation should be carried out in stages, unscrewing the locking rings at each stage just enough to enable the lowering in the next stage to take place, and taking vertical deflection readings in conjunction with jack pressures and loads. Lowering should be controlled by deflection until touch-down, following which control should be by jacking load until the jack loads have been released. Settlement of the bearing base plate must be carefully measured using deflection gauges to ensure that compression of the bedding mortar is restricted to approximately 0.3 mm or a calculated acceptable amount. This provides confirmation that the bedding is substantially void-free and that the mortar has properly hardened. For sliding bearings, checks should be made that sufficient PTFE protrusion exists and that sliding plates are flat. The bearings should be inspected for satisfactory operation under live loading before the jacking equipment is removed.

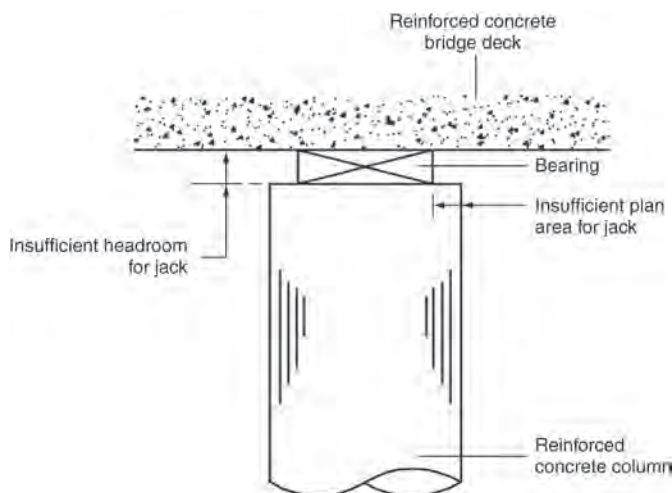


Figure 20.15 Pier with insufficient room for jacking (reproduced with permission © Atkins)

Illustrations of practice

This section illustrates examples of good and bad practice. Figures 20.15 to 20.24 all indicate problem areas.



Figure 20.16 Lack of provision for jacking (reproduced with permission © the Highways Agency)



Figure 20.17 Lack of provision for jacking (reproduced with permission © the Highways Agency)



Figure 20.18 Lack of provision for jacking: detail (reproduced with permission © the Highways Agency)

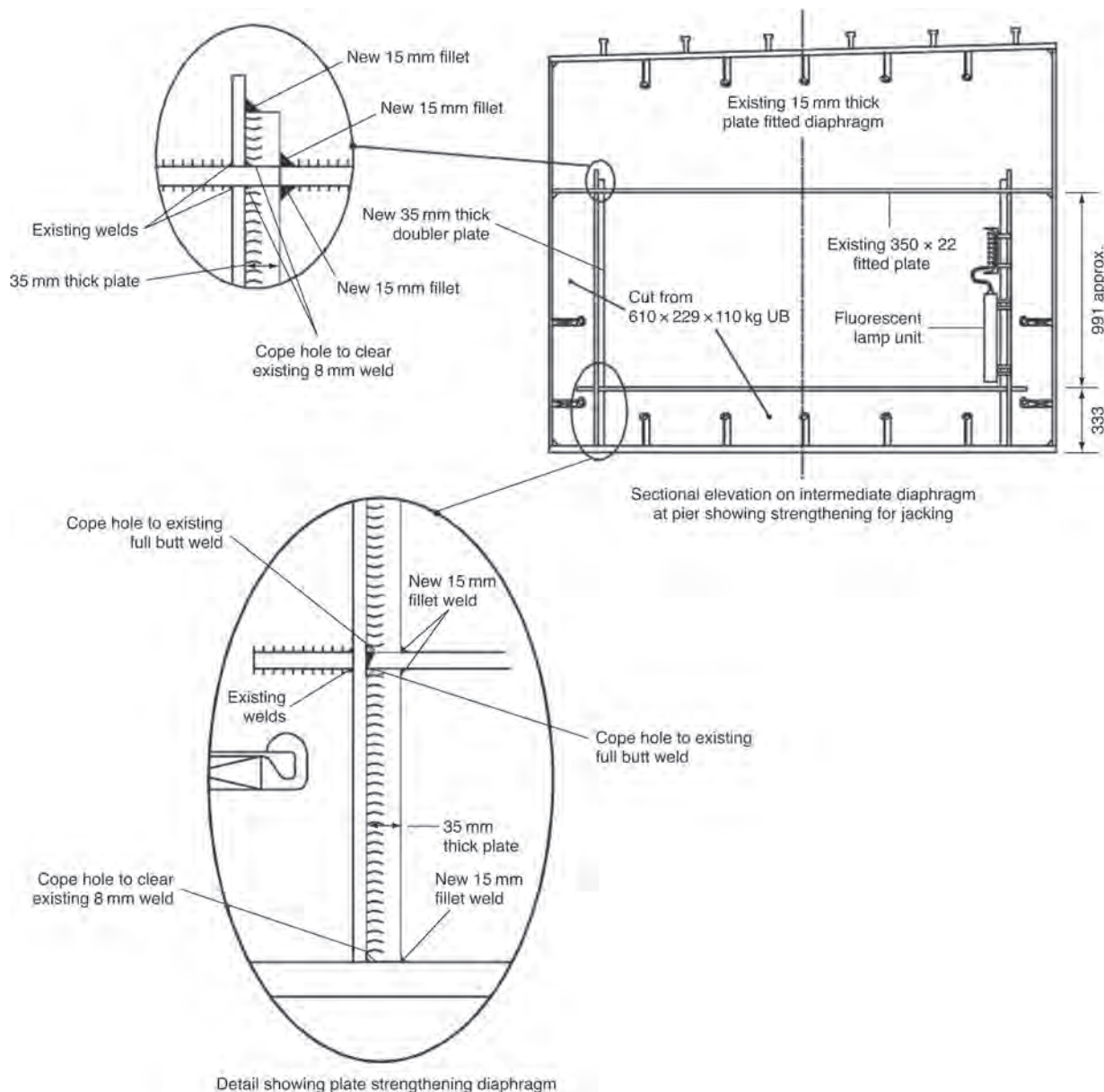


Figure 20.19 Steel box strengthening for jacking strong points (reproduced with permission © Atkins)

Problem areas

Figures 20.15 to 20.18 are all examples of situations in which bridge piers have been built without either vertical or plan space for the insertion of jacks for bearing replacement. Figures 20.16, 20.17 and 20.18 show three real-life examples of this practice.

Figure 20.19 shows how steel box strengthening can be used in instances where a bridge deck has not been provided with jacking strong points.

Figure 20.20 demonstrates the problems that can arise with elastomeric bearings when the deck is anchored to the abutment by a bonded dowel bar passing through the bearing.

Bearings that have reached the end of their translation travel

Figure 20.21 is a plan of a viaduct. If the viaduct is constructed from end A to end B, temporary longitudinal fixity is required at end A until the fixed bearing at C is installed, when the bearing at A can be released. As the viaduct construction continues through D and E to B, longitudinal movements owing to temperature and shrinkage occur, and, if prestressed, owing to prestress, which in turn is modified by prestress losses and creep. Depending on how far these have occurred, and on the effective bridge temperature at the time, bearings D, E and B must be set in turn to suit, so that future changes in these effects can

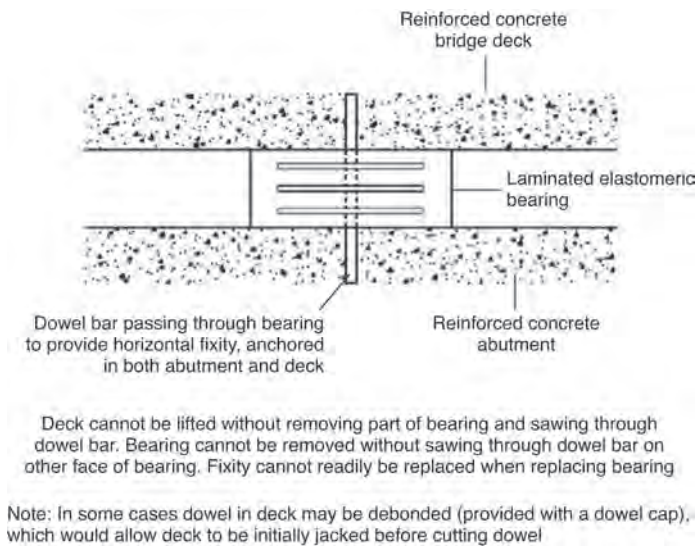


Figure 20.20 Dowelled elastomeric bearing (reproduced with permission © Atkins)

be accommodated within the limits of travel of the bearings. If bearings are not chosen with sufficient sliding capacity, or if the setting calculations are incorrect, the bearings may run out of travel at some future date, resulting in a serious condition for the integrity of the viaduct, by either failure or seizure of the bearings. Bearings might also run out of travel, owing to excessive movements of the substructure or through construction tolerances not being corrected for prior to installation.

Bearings subjected to live load fretting

Figure 20.22 illustrates an example of live load fretting. Wear of sliding bearings was originally thought to result from temperature changes and reversals. However, traffic loading on the deck, in bending it and extending the bottom chord, results in a very large number of small reciprocal sliding movements, which do cause significant wear. This is termed live load fretting, and has been known to result in the shortening of bearing life. For example the DU surface of sliding bearings has been known to have worn through, exposing the bearing to material below, of significantly higher friction coefficient.

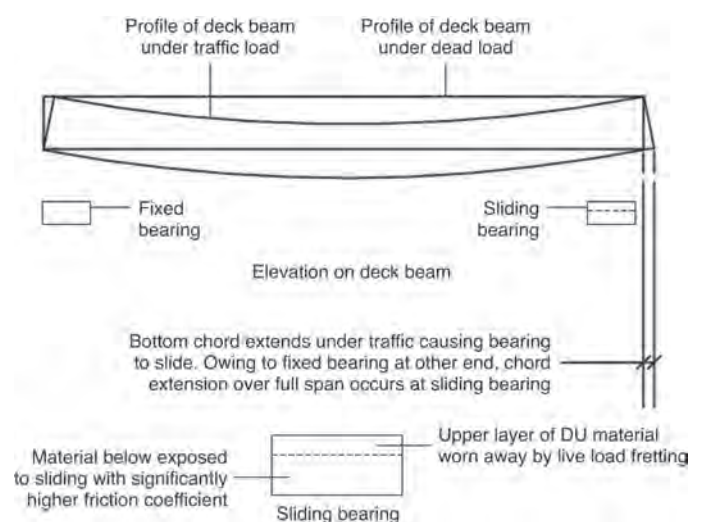


Figure 20.22 Live load fretting (reproduced with permission © Atkins)

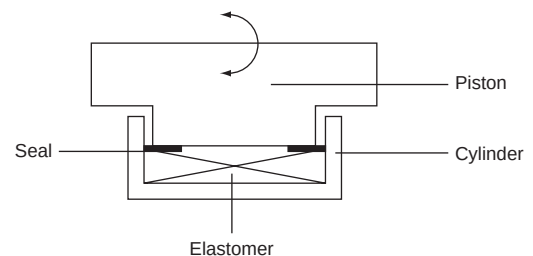


Figure 20.23 Pot bearing (reproduced with permission © Atkins)

Pot bearing in lively deck subjected to seal break down

Pot bearing seals, as illustrated in Figure 20.23, are subjected to vertical and rotational movements of the piston in the cylinder and are required to resist the pressure in the elastomer, which is, by nature, hydraulic. They are tested under these conditions to ensure an adequate fatigue life. However, if the deck is particularly lively, the seal may fail, the elastomer extrude and the bearing become no longer serviceable. This has been known to occur at the pier of a cable-stayed bridge, where the bearing was subjected to unusually lively movements.

known to occur at the pier of a cable-stayed bridge, where the bearing was subjected to unusually lively movements.

Roller bearing cracking owing to excessive metal hardness

To provide very low friction, stainless steel rollers of very high hardness have been used, but high hardness might compromise corrosion or fatigue resistance. Hardness limits have been introduced to avoid the problem.

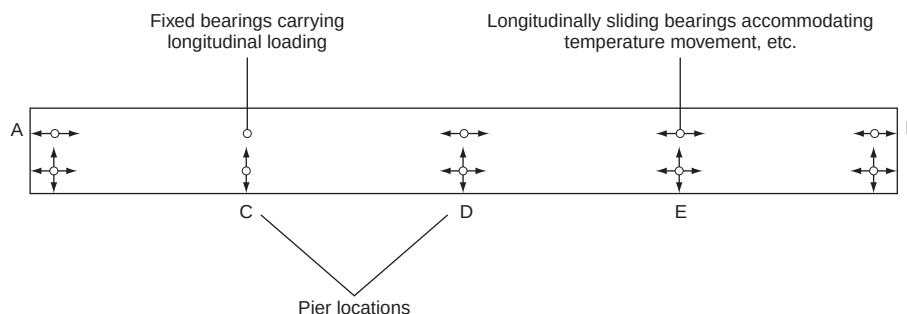


Figure 20.21 Plan on viaduct (reproduced with permission © Atkins)

Mechanical bearing top plate deflection due to high-build paint thickness

Prevention of corrosion of bearing plates has been sought by requiring the paint coating to be returned a minimum of 50mm beyond the edge of the meeting surfaces of bearing plates. However, with the advent of high-build paint coatings, this might result in non-uniform bedding of the plates on each other, to the extent that the plates might bend, with injurious effects on the bearing, as illustrated in Figure 20.24. To avoid this, it is now recommended that the paint not be returned beyond the edge of the plate, but that a final paint coating be applied to the plate edges and to any exposed plate returns after installation of the bearing. The cumulative effect of the individual plate tolerances in a stack of plates might also result in non-uniform bedding of the bearing.

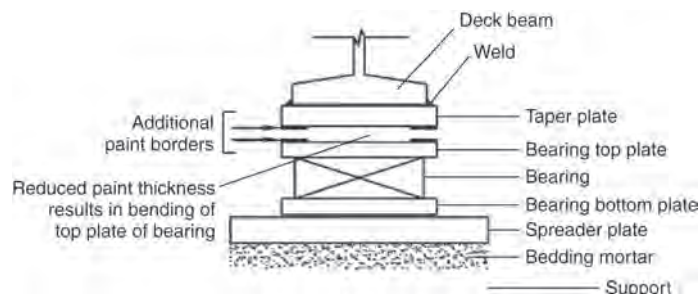


Figure 20.24 Section through deck beam at bearing (reproduced with permission © Atkins)

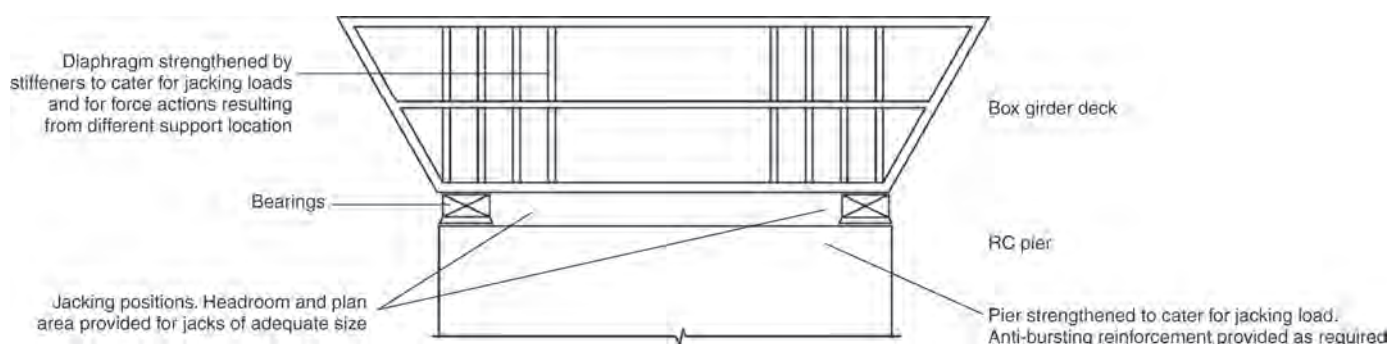


Figure 20.25 Temporary supports either side of bearing for landing deck beam (reproduced with permission © the Highways Agency)

Good practice

Figures 20.25 to 20.32 all illustrate examples of good practice. Figure 20.26 illustrates an instance where a bridge pier has been constructed with adequate provision for jacking. Figures 20.26 to 20.30 show actual examples of bridge piers with adequate space and strong points for jacking.

Figure 20.31 exemplifies an elastomeric bearing that has been appropriately restrained.

Figure 20.32 shows a bearing with taper, spacer and spreader plates all appropriately protected.



Figure 20.26 Elastomeric bearing with facility for replacement (reproduced with permission © Atkins)



Figure 20.27 Jacking down off jacking trestle (reproduced with permission © the Highways Agency)



Figure 20.28 Jacking steel diaphragm beam, showing transverse restraints (reproduced with permission © the Highways Agency)



Figure 20.30 Jacking concrete deck (reproduced with permission © the Highways Agency)

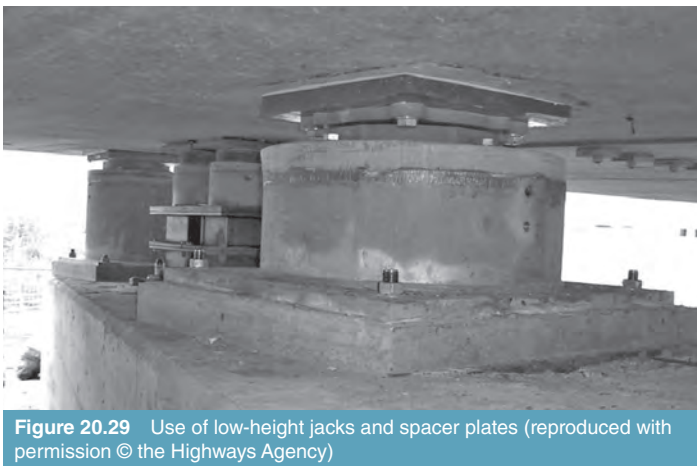


Figure 20.29 Use of low-height jacks and spacer plates (reproduced with permission © the Highways Agency)

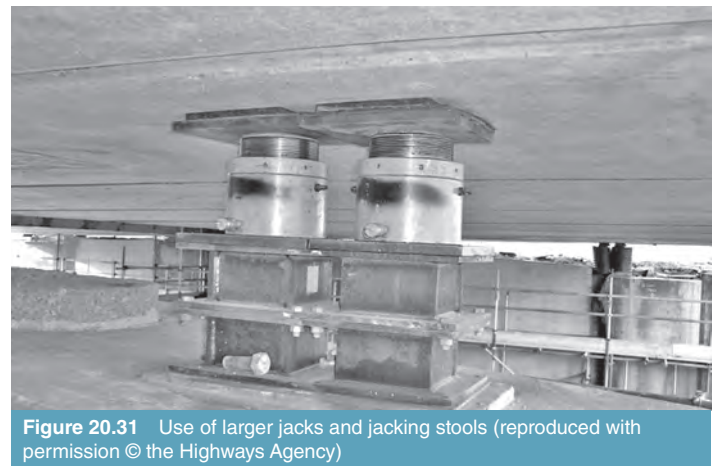


Figure 20.31 Use of larger jacks and jacking stools (reproduced with permission © the Highways Agency)

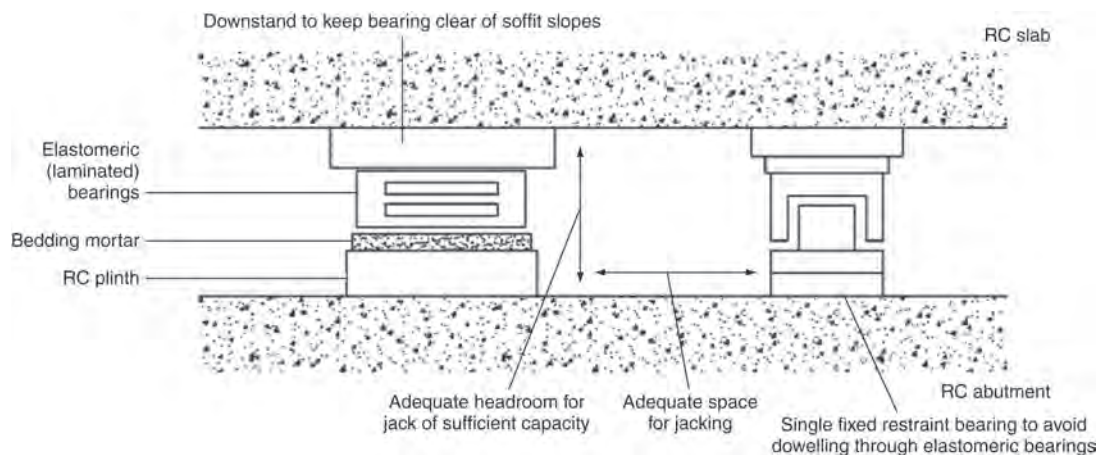


Figure 20.32 Section through deck beam, with bearing plates appropriately protected (reproduced with permission © Atkins)

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Chapter 21

Bridge accessories

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This chapter considers parapets, expansion joints, waterproofing and drainage. Parapet containment levels, working width classes and impact severity levels are tabulated, and a risk assessment approach to containment is outlined. Parapet materials are described, as well as design considerations. The common types of expansion joint are described, together with the design approach, drainage measures, detailing and best practices. A brief description of varied types of bridge deck waterproofing is presented, with descriptions of detailing, installation and maintenance.

Introduction

This chapter looks at the various additional elements that are so important in bridge design and construction. Each of the following is examined in turn:

- road restraint systems
- expansion joints
- waterproofing
- drainage.

Road restraint systems

Definitions and test requirements

Road restraint systems (RRSs) on a bridge are intended to contain errant vehicles or to protect pedestrians, bicyclists and vehicular traffic and guide pedestrians. They also serve to provide protection to the areas beneath and adjacent to structures, for example, other roads, railways or buildings.

Depending on the function they are designed to serve, RRSs are broadly classified as vehicle restraint systems (VRSs) and pedestrian restraint systems (PRSs). Each system comprises

subelements that are essential for the overall performance of the defined system. A general classification of the RRS along with the system components is shown in Figure 21.1.

The variation in types of vehicle using highways is considerable and constantly changing. Vehicles vary in size, both in length and height, overall weight, axle spacing and weight distribution. A vehicle's response to an impact can also vary considerably from model to model, owing to such features as engine position, provision of crumple zones or suspension type.

Dynamic tests, which are conducted on a representative panel of parapet, are designed to check the parapet's response to the standard vehicle impact for the appropriate containment class. Standard production vehicles in roadworthy condition should be used. Owing to the rate of change in vehicle design, it is desirable that vehicles used in the testing should be examples of vehicles in current production and less than 5 years old.

To satisfy the vast array of requirements for parapets in a single design would be almost impossible; it is therefore normal to specify for a specific containment requirement. In the UK, RRSs are defined based on the risk; thus, the key parameters for the vehicle parapets, including transitions over the bridge, should be through the standards of the Road Restraints Risk Assessment Process (RRRAP) and the UK Design Manual for Road and Bridges CD 377 Requirements of RRS (HE, 2020). These standards describe the procedures to be followed by the

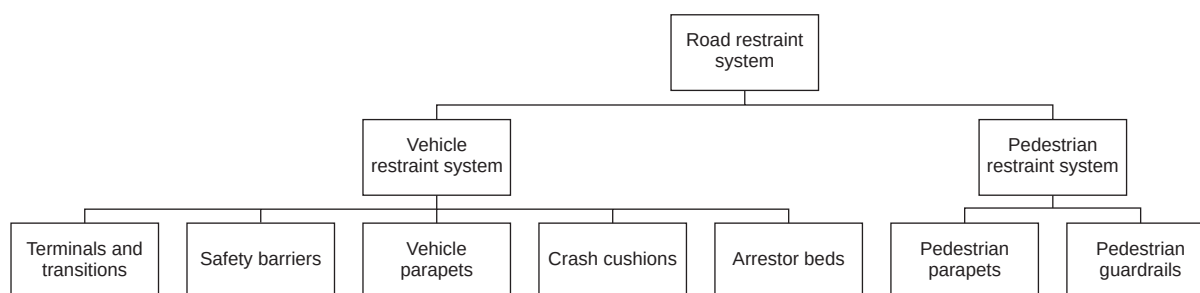


Figure 21.1 Road restraint system

various parties involved in the design and provision of various types of RRS on the road network to ensure compliance with European Commission requirements for RRS set out in BS EN 1317-1 to BS EN 1317-6. To satisfy the requirements of the relevant containment class, the vehicle must be retained and redirected in the prescribed manner.

Based on the level of risk within the RRRAP, RRSs should be proposed for the bridge and need to define the following minimum parameters for the safety barriers, vehicle parapet and transitions:

- containment level (e.g. N1, N2, H1, H2 or H4a)
- impact severity level (ISL) (e.g. ISL Class B)
- working width class (W1 to W8)
- the maximum height that allows the required visibility
- length of safety barrier and parapets needed

Standard BS EN 1317 contains testing requirements for ten containment levels, as given in Table 21.1.

Damage to the vehicle during testing must be of an acceptable nature and forces imparted to the vehicle's occupants must be kept to a minimum. Although parapet designs are completed to satisfy the specific requirements of a particular level of containment, consideration should be given to their response to other vehicles on the highway. For this reason, BS EN 1317 requires parapets of most containment levels to be additionally tested for a small car (1300kg or 900kg) in addition to the prescribed containment requirements. Impact severity is defined in BS EN 1317 in terms of the accident severity index (ASI), theoretical head impact velocity (THIV) and post-impact head deceleration (PHD), all of which are measured during full-scale dynamic testing. Standard BS EN 1317 defines two impact severity levels, as given in Table 21.2.

Road restraint systems are further defined by working width classes, which are determined by the deflection characteristics of the barrier and the behaviour of the vehicle in full-scale dynamic tests. The working width level of a road restraint system determines the space required to accommodate the barrier. Standard BS EN 1317 defines eight working width classes, as given in Table 21.3.

Containment level	Type of vehicle	Total vehicle mass: kg	Impact speed: km/h	Impact angle: degrees
Low-angle containment				
T1	Car	1300	80	8
T2	Car	1300	80	15
T3	Rigid HGV	10000	70	8
Normal containment				
N1	Car	1500	80	20
N2	Car	1500	110	20
Higher containment				
H1	Rigid HGV	10000	70	15
H2	Bus	13000	70	20
H3	Rigid HGV	16000	80	20
Very high containment				
H4a	Rigid HGV	30000	65	20
H4b	Articulated HGV	38000	65	20
Containment level T3 must be additionally tested for the following:				
	Car	1300	80	8
Containment levels N2, H1, H2, H3, H4a and H4b must be additionally tested for the following:				
	Car	900	100	20

Table 21.1 Testing requirements for containment levels, as given in BS EN 1317

Impact severity level	Accident severity index	Theoretical head impact velocity: km/h	Post-impact head deceleration
A	≤1.0	≤33	≤20g
B	≤1.4	≤33	≤20g

Table 21.2 Impact severity level, as defined by BS EN 1317

Working width level	Working width: m
W1	$W \leq 0.6$
W2	$W \leq 0.8$
W3	$W \leq 1.0$
W4	$W \leq 1.3$
W5	$W \leq 1.7$
W6	$W \leq 2.1$
W7	$W \leq 2.5$
W8	$W \leq 3.5$

Table 21.3 Eight working width classes, as defined by BS EN 1317

Requirements for vehicle parapets

Standard CD 377 (HE, 2020) specifies that bridge parapets must have a minimum containment level N1 for a speed of less than 50 mph and N2 for a speed greater than 50 mph. However, bridges over railways or other sensitive pieces of infrastructure should be provided with a very high containment level (H4a).

Supports to lightweight structures, such as steel gantries and footbridges, will often require at least a H1 containment barrier to avoid the need for designing for high collision loads. The RRRAP may require that a higher containment is provided than the minimum stipulated.

Standard CD 377 specifies that the impact severity level (ISL) for vehicle parapets must be either level A or B, as stipulated in BS EN 1317-2. Impact severity level A affords a greater level of comfort for vehicle occupants than level B.

Standard CD 377 specifies that the maximum working width class allowed by the local hazard should be used for normal containment. For higher and very high containment level vehicle parapets, the vehicle intrusion class should also be considered in order to determine the maximum working width class.

The overall length of the VRS for one or more hazards includes a minimum length of full-height full-containment VRS in advance of, alongside and after the hazard or hazards to ensure protection. The overall length should comprise a continuous length, which may or may not be from one type of

product (e.g. when both metal and concrete safety barriers are provided, both would constitute a continuous length).

The parapet should be provided for the full length of the bridge deck and along the full length of the wing walls where they broadly follow the road alignment. Suppliers should be consulted for the minimum length of the vehicle parapet before it connects with the safety barrier. For H2 and H4a containment levels, the parapet should also extend 25 m in advance of the nearer point of no recovery (PNR) for the hazard to a position of 10 m beyond the further point of no recovery for the hazard. However, on two-way single carriageways, the 10 m length requirement should be increased to 25 m. See Figure 21.2.

In general, the minimum lengths for the safety barriers are provided in advance and beyond the hazard, as given in Table 21.4.

Containment level	Minimum 'full-height' length of safety barrier: m	
	In advance of hazard	Beyond hazard
Normal (N1 or N2)	30.0	7.5
Higher (H1 or H2)	30.0	10.5
Very high (H4a)	40.0	18.0

Table 21.4 Length of vehicle restraint system before and after hazard

The total length of the safety barrier must be the length of need plus the additional length declared by the manufacture in advance of, and after, the length of need, to ensure that the safety barrier attains full containment at the points required.

For example, a bridge has a 30 m parapet length over the deck and wing walls. If the length of need (LoN) of an N2 safety barrier is 67.5 m (30 m + a hazard length of 30 m + 7.5 m), but full containment is only to be achieved 20 m from the safety barrier end of the terminal, then the minimum required length of the safety barrier between terminals will be 107.5 m (i.e. 20 m + 30 m + hazard length of 30 m + 7.5 m + 20 m). However, if the same bridge is on a two-way single carriageway, the 7.5 m length on departure will be increased to 30 m, as vehicles can approach from both sides.

Also, CD 377 specifies that the minimum heights for vehicle parapets must be measured above the adjoining paved surface; these are given in Table 21.5.

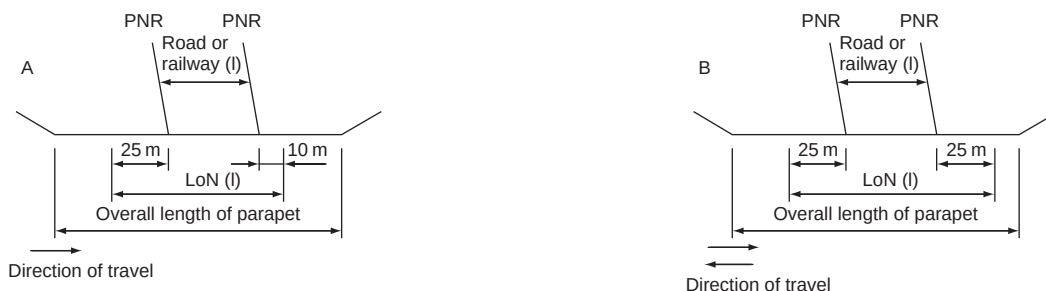


Figure 21.2 Parapet extension from point of no recovery (PNR): LoN, length of need

Type of parapet	Height: mm
Bridge over another road	
For vehicle parapets except as below	1000
For cycleways immediately adjacent to the vehicle parapet	1400 ^a
For very high containment applications or accommodation bridges	1500
For bridleways immediately adjacent to the vehicle parapet	1800
Bridge carrying road over railway	
For all bridges and structures over railway	1800
^a Current version of code specifies 1500mm, but this will be changed to 1400mm in the next revision.	

Table 21.5 Height of bridge parapets

Design requirements of parapets and supporting structures

The main structural members of bridges should not be designed to act as vehicle parapets and separate VRS systems should be provided to the required containment level and working width. This is to ensure redundancy of the system and prevent structural damage on critical load-carrying elements resulting from vehicle collision.

Global effects

To determine global effects resulting from a vehicle impact, Table NA.6 of the national annex to BS EN 1991-2:2003 (BSI, 2020) defines the loads based on the containment level of vehicle parapet, as given in Table 21.6.

The forces defined in Table 21.6 should be applied uniformly over a length of 3 m at the top of the traffic face of the vehicle restraint system and in a position along the line of the vehicle restraint system that produces the maximum effects on the part of the structure under consideration.

Local effects

As given in Clause 4.7.3.3 (2) of BS EN 1991-2:2003, the structure supporting the vehicle parapet should be designed to sustain locally an accidental load effect corresponding to at least 1.25 times the characteristic local resistance of the vehicle parapet (e.g. resistance of the connection of the parapet to the structure) that need not be combined with any other variable

load. These loads are supplied by the parapet manufacturer. Since the parapet system to be used may not be known at the time of preparing the design, the designer will be required to make assumptions relating to the loads applied to the supporting structure. Any limitations required to the design of the parapet, which may be imposed by these assumptions, must be made clear on the drawings.

Parapets can be constructed from a variety of materials, but most modern parapets are of metal (generally steel or aluminium), reinforced concrete, a combination of reinforced concrete and metal or, to a lesser

extent, masonry. Standard CD 377 does not allow the use of masonry parapets for new and replacement parapets, except where agree with the overseeing organisation.

Metal parapets

Metal parapets (Figure 21.3) are generally of post and rail construction, with post centres spaced at approximately 3.0–4.0 m. The space between the horizontal rails is normally limited to a maximum of 300 mm, to prevent penetration of the front of an impacting vehicle. Metal parapets have been designed with vertical in-fill bars between upper and lower horizontal rails but these have generally been found only to be suitable for low levels of containment, since they tend to arrest an impacting vehicle rather than redirecting it.



Figure 21.3 Metal parapet with and without fencing

Class	Transverse force: kN	Longitudinal force: kN	Vertical force: kN	Examples of application
A	100	–	–	Normal containment flexible parapets (e.g. metal post and rail parapets)
B	200	–	–	Normal containment rigid parapets (e.g. reinforced concrete parapets)
C	400	100	175	Very high containment flexible parapets (e.g. metal post and rail parapets)
D	600	100	175	Very high containment rigid parapets (e.g. reinforced concrete parapets)

Table 21.6 Loads from Table NA.6 of the national annex to BS EN 1991-2:2003 (BSI, 2020)



Figure 21.4 Concrete parapet

Concrete parapets

There is no current standard for the design of concrete parapets (Figure 21.4). In the absence of any guidance, these must be designed in accordance with the now superseded BS 6779-2, as amended by Paragraph 4.78 of CD 377 (HE, 2020). When designing to BS 6779-2, the normal level of containment of BS 6779-2 is considered to be equivalent to normal containment level N2 of BS EN 1317-2, and the high containment level of BS 6779-2 is considered to be equivalent to very high containment level H4a of BS EN 1317-2.

Reinforced concrete vehicle parapet panel walls must have a minimum thickness of 180 mm for normal containment level N2 and 325 mm at the critical design section for very high containment level H4a.

Reinforced concrete vehicle parapet panel walls must have a minimum length of 2.0 m and a maximum length of 3.5 m. For the reinforcement in the in situ vehicle parapet wall, γ_m must be 1.0, not 0.8 as given in Table 4 of BS 6779-2.

Vehicle parapets for normal containment level N2 must be designed for an equivalent static nominal load for a nominal

bending moment of 100 kN over 1.0 m, and not 50 kN over 1.0 m, as given in Table 2 of BS 6779-2.

Vehicle parapets for normal containment level N2 must be designed with shear transfer provision between adjacent panels. An equivalent static nominal load of 50 kN must be transferred between adjacent panels within the top 0.5 m of the sections.

Combined metal and concrete parapets

Combined reinforced concrete and metal parapets (Figure 21.5) normally consist of a reinforced concrete plinth of between 400 and 900 mm in height surmounted by one or two horizontal metal rails.

There is currently no design guidance for metal and concrete parapets. Where such a parapet is required, it would need to be designed from first principles and tested in accordance with BS EN 1317. It should be noted that testing approvals for new parapets take a significant amount of time and must therefore be planned well in advance of their installation.

Masonry parapets

Plain masonry parapets (Figure 21.6) are inclined to fracture into small blocks or even individual bricks when impacted. The detached sections present a serious risk of causing a secondary accident and such parapets should generally only be considered suitable for situations requiring low levels of containment and in areas where detached masonry would not be hazardous. If a masonry finish is required for aesthetic reasons, this can be achieved by the use of cladding on a reinforced concrete parapet. In such situations, the cladding, which should not generally be placed on the traffic face of the parapet, must be securely anchored to the concrete core of the parapet. The cladding should not be considered structural and should be fixed in a manner to avoid increasing the ultimate capacity of the concrete parapet at its designed failure position.

Masonry vehicle parapets must not be used on road bridges and structures over or adjacent to roads, except where agreed by the overseeing organisation. These should be designed from first principles. As these parapets will fall outside the performance class, a departure from standards is usually required.

Many authorities provide metal safety barriers in front of masonry parapets to reduce the risk of injury from vehicle impacts with masonry parapets.



Figure 21.5 Combined metal and concrete parapet



Figure 21.6 Masonry parapets

Expansion joints

Introduction

Wherever two elements of a structure are moving relative to each other, it is often necessary to provide an expansion joint that seals the gap between the two elements while accommodating the relative movements. For bridges, the gap is usually that between the deck end and the abutment ballast wall. However, on long viaducts, additional joints will often be needed between sections of deck in order to limit the movement at any one point. Expansion joints are, by virtue of their function, a point of weakness within a bridge and history has highlighted many examples of joints leaking. As water, laden with de-icing salts, has leaked onto bearing shelves or pier supports, corrosion of the reinforcement has frequently resulted. The repairs that are required cost significantly more than the capital cost of the joint alone, especially when the cost of traffic delays is taken into account. It is therefore important to give full consideration to the design, detailing and installation of bridge expansion joints in order to minimise the risk of the bridge owner being faced with high repair bills in the future.

The vulnerability of expansion joints is one of the driving forces behind the increased use of integral bridge construction, in which the need for expansion joints is eliminated by connecting the deck directly to the abutments. Many transportation agencies have empirically established maximum lengths for jointless bridges. In the UK, the use of integral bridges was mandated through the application of BD 57 (HE, 2001) for bridges with overall lengths less than 60m long and skew angles of up to 30°. For non-integral structures, if the total design horizontal movement is less than 5 mm and vertical movement is less than 0.5 mm, expansion joints can be ignored, and the gap needs to be filled with seal or sealant covered by waterproofing, with due care to the detailing.

As it is not practical to eliminate movement gaps for bridges with longer lengths and larger skews, the use of expansion joint systems is required to accommodate cyclic movements without

imposing significant secondary stresses on the superstructure. Sealed expansion joint systems provide barriers, preventing run-off water and de-icing chemicals from passing through the joint onto bearings and substructure elements below the bridge deck. Expansion joint systems are thus expected to perform such that the following basic requirements are met.

- They are able to accommodate the full movement range of the adjacent decks (structures) ending at the joints. The movement could be vertical, horizontal, transverse and longitudinal or rotational.
- They provide a smooth ride and skid resistance for traffic passing over the joint.
- They sustain all loads subjected to it from passing traffic without damage to the surfacing or supporting structure during their working lives.
- They will be fully watertight at all points, from parapet to parapet, throughout the entire movement range.

Whether expansion joints are designed to be watertight or not, it must be assumed that joints may leak in the future, special attention therefore needs to be placed by the bridge design engineers on the following:

- protection of all concrete surfaces below the joint, with the appropriate protective coating system
- provision of proper drainage (large-diameter drainage pipes and adequate drainage slopes) at the level of the bearing seats to ensure that staining of the abutment exposed faces and pier sides does not occur
- provision of drip grooves at specific locations, such as under corbels supporting the joint in an abutment gallery and along deck soffit edges, to ensure that water does not cause the contamination of reinforced concrete elements of the bridge and of bearings
- provision of bearing plinths to ensure that bearings are out of reach of any possible flooding of the abutment and pier tops
- provision of stainless steel reinforcement close to all concrete surfaces below a joint that are inaccessible for future maintenance

Joint type	Total acceptable longitudinal movement		Maximum acceptable vertical movement between two sides of joint: mm
	Minimum: mm	Maximum: mm	
Buried joint under continuous surfacing	5	20	1.3
Asphaltic plug joint	5	40	3
Nosing joint with poured sealant	5	12	3
Nosing with preformed compression seal	5	40	3
Reinforced elastomeric	5	a	3
Elastomeric in metal runners	5	a	3
Cantilever comb or tooth joint	25	a	3

The minimum of the range is given to indicate when the type of joint might not be economical.

^aMaximum value varies according to manufacturer or type.

Table 21.7 Expansion joint types (HE, 1994)

ETAG 032 joint family	Total longitudinal movement: mm		Maximum vertical movement between sides of joint: mm
	Minimum ^a	Maximum	
Buried expansion joint	5	20	1.3
Flexible plug joint	5	40	3
Nosing expansion joint, with poured sealant	5	12	3
Nosing expansion joint, with preformed compression seal	5	40	3
Mat expansion joint	5	Varies ^b	3
Cantilever comb or tooth joint	25	Varies ^b	3
Supported expansion joint	150	Varies ^b	3
Modular expansion joint	25	Varies ^b	3

^aMinimum of the range is given to indicate when the type of joint might not be economical.

^bMaximum value varies according to manufacturer or type.

Table 21.8 Extract from CD 357 – Expansion joint types (HE, 2019a)

- where pedestrians have access, the closure of all expansion joints with an open joint gap at surface level in the footway using either a cover plate or a load-bearing seal below the top surface of the joint.

Replacing joints can prove to be very expensive once all the associated traffic management and delay costs are considered. These can easily outweigh the initial capital expenditure in whole-life costing terms. Unfortunately, specific expansion joint systems are often selected based on their initial cost, with minimal consideration for long-term performance, durability and maintainability.

Design of expansion joints

The nominal movement capacity of an expansion joint is the allowance for displacement of the parts of the main structure under varied loaded conditions. The movement capacity should be assessed for three directions: longitudinal, transverse and vertical. Design loads and movements for expansion joints are laid down in Annexure G of ETAG no 032 Part 1 (EOTA, 2013). For Highways England projects, this is further augmented by CD 357 (HE, 2019a), which has recently been updated to provide consistency with ETAG 032. It should be noted that this has led to minor changes in terminology for referencing bridge joints, as can be seen by the document extracts given in Table 21.7, from BD 33/94 (HE, 1994), and Table 21.8, from CD 357. The terminology used in BD 33/94 for the expansion joint type ‘elastomeric in metal runners’ is now combined with that for the modular expansion joint used in CD 357.

The expansion joint should be designed and built in such a way that the loads and imposed deformations or displacements that are liable to act on it during construction and use of the works will not lead to any of:

- collapse of the whole or a part of the works
- major deformations to an inadmissible degree

- damage by an event to an extent disproportionate to the original cause.

As a general criterion, the expansion joints must accommodate movements produced by concrete shrinkage and creep, post-tensioning shortening, thermal variations, dead and live load, wind and seismic loads and structure settlement. Expansion joints should be designed to accommodate all shrinkage occurring after their installation. For unrestrained concrete, ultimate shrinkage strain after installation, β , may be estimated as 0.0002 (Washington State DoT, 1997). More detailed estimates can be used, which include the effect of ambient relative humidity and volume-to-surface ratios (AASHTO, 1994). Shrinkage shortening of the bridge deck, $\Delta_{\text{shrinkage}}$ (in mm), is calculated as

$$\Delta_{\text{shrinkage}} = (\beta) \cdot (\mu) \cdot (L_{\text{trib}}) \cdot (1000 \text{ mm/m}) \quad (21.1)$$

where L_{trib} is the tributary length of the structure subjected to shrinkage (in m), β is the ultimate shrinkage strain after expansion joint installation (estimated as 0.0002 in lieu of more refined calculations) and μ is a factor accounting for restraining effect imposed by structural elements installed before the slab is cast (HE, 2019b), which is 0.0 for steel girders, 0.5 for pre-cast prestressed concrete girders, 0.8 for concrete box girders and T-beams and 1.0 for flat slabs.

Thermal displacements are calculated using the maximum and minimum anticipated bridge deck temperatures. These extreme values are functions of the geographic location of the structure and the bridge type. Thermal movement (in mm) is calculated as

$$\Delta_{\text{temp}} = (\alpha) \cdot (\delta T) \cdot (L_{\text{trib}}) \cdot (1000 \text{ mm/m}) \quad (21.2)$$

where α is the coefficient of thermal expansion, which is 0.000011/°C for concrete and 0.000012/°C for steel, L_{trib} is the tributary length of the structure subjected to thermal variation (in m) and δT is the temperature variation (in °C).

Any other predictable movements following joint installation, such as concrete post-tensioning shortening and creep, should be included.

Where large skews feature in the design, it should also be noted that this will adversely impact the joints through twisting. To avoid future joint failures, this twisting effect will need to be considered in the selection of an appropriate joint.

Types of joint

Expansion joints can be broadly classified into three categories, depending on the total movement range accommodated; these delineated ranges are somewhat arbitrary (Figure 21.7). Small movement range joints encompass all systems capable of accommodating a total movement range of up to about 45 mm. Medium movement range joints include all systems capable of accommodating a movement range of 45–130 mm. Large movement range joints accommodate total movement ranges in excess of about 130 mm. The delineated ranges are arbitrary and overlapping movement ranges can be found between these broad classifications. The maximum vertical movement capacity for most expansion joints is of the order of 3 mm, except for the buried expansion joint, for which it is about 1.3 mm.

Different types of expansion joint are briefly described in the following sections. However, the exact details will vary between manufacturers; for exact details, an appropriate supplier should be contacted.

Buried expansion joints

These joints are formed in situ using components such as the waterproofing membrane, acting together with an elastomeric pad or proprietary flashing, to distribute the deformations to a greater width and to support the surfacing, which is continuous over the deck joint gap (Figure 21.8). A lubricant or thin sheet of flexible material may also be installed as a debonding layer for the waterproofing membrane to prevent adhesion between selected components below the waterproofing. Crack-induced cuts (8 mm wide by 25 mm deep) filled with a bitumen-based sealant or reinforcement grid are introduced in the asphalt surfacing to control cracking above the joint. The elastomeric

pad is preferably recessed into the concrete so as not to reduce the thickness of the surfacing across the joint.

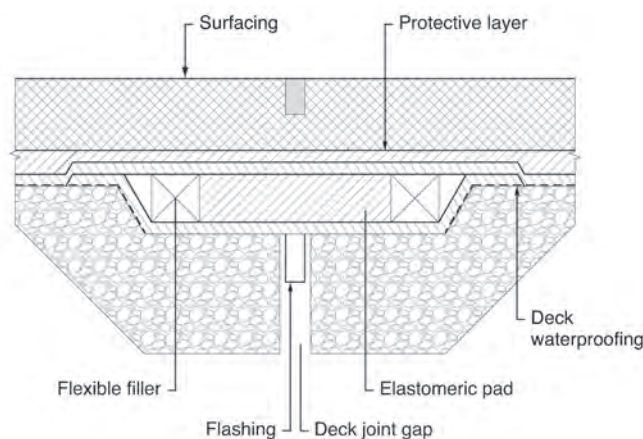


Figure 21.8 Example of buried expansion joint

The designed longitudinal movement range for these joints could vary from very small movements of a couple of millimetres to a maximum of 20 mm; the maximum vertical movement between two sides of the joint should not exceed 1.3 mm. These joints are typically installed where the asphalt wearing surface is placed directly on top of the tunnel roof slab or underpass bridge deck, at longitudinal joints in the roof slab of tunnels or underpasses, at transverse joints between the approach slab and abutment back wall, or at expansion, contraction and construction joints in a tunnel base slab.

Flexible plug expansion joints (asphaltic plug joints)

This is a proprietary in situ joint system comprising a binder and aggregate system composed of specially blended polymer-modified asphalt and specific aggregate, placed in layers in a prepared expansion joint blockout (Figure 21.9). This joint system will provide a flexible waterproof bridge joint and smooth running surface, which will allow for a total joint movement of 40 mm (± 20 mm).

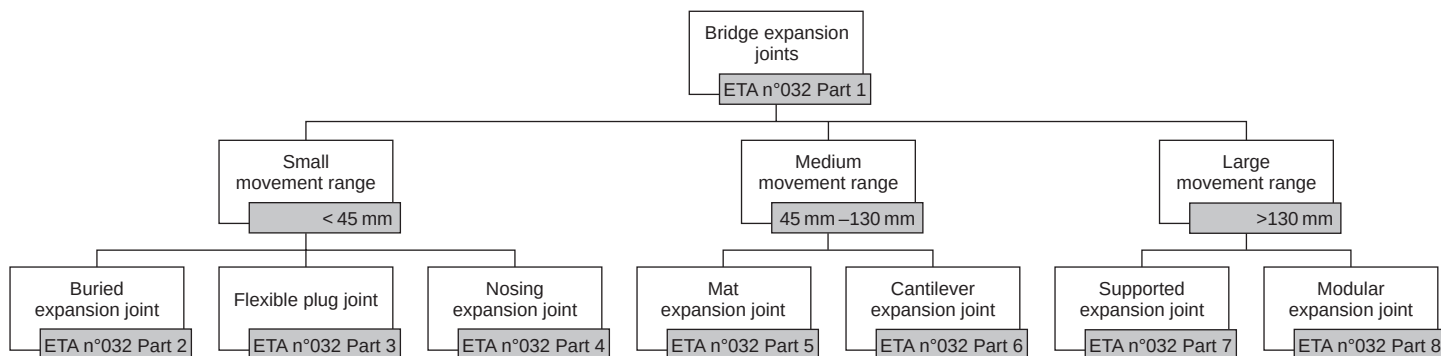


Figure 21.7 Bridge expansion joints

These joints should not be used to accommodate differential vertical displacements or roadways subjected to significant traffic acceleration and braking, owing to the creep susceptibility. These joints have also performed poorly in highly skewed bridges and joints subjected to large rotation. These joints are normally installed at a nominal 500 mm width, with adjacent surfacing not less than 100 mm thick.

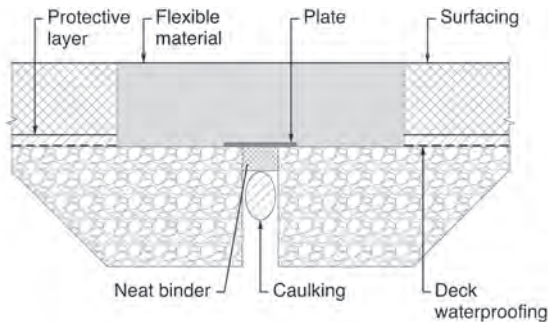


Figure 21.9 Example of flexible plug joint

Nosing expansion joints (compressible seal joints)

Compression seals are continuous elastomeric sections, typically with extruded internal web systems, installed within an expansion joint gap to seal the joint effectively against water and debris infiltration (Figure 21.10). The nosing protects the adjacent edges of the surfacing at the expansion joint at road level, provides a straight edge for the placing of joint sealants or provides a support structure for the metal runners forming part of propriety expansion joint systems. Because compression seals are held in place by friction, their performance is depends very much on the close correlation of the construction joint width and the design joint width. If the joint opening is constructed too wide, friction force will be insufficient to prevent the compression seal from slipping out of the joint at wider expansion gap widths.

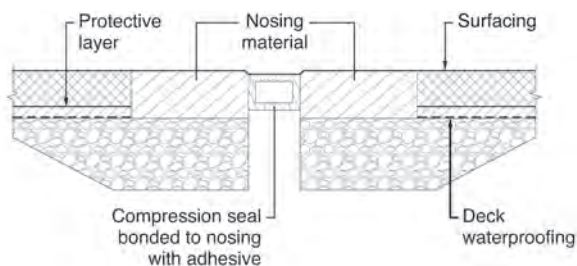


Figure 21.10 Example of compressible seal joint

In design calculations, the maximum and minimum compressed widths of the seal are generally set at 85% and 40% of the uncompressed width (HE, 2019b). These widths are measured perpendicular to the axis of the joint. It is also generally assumed that the width of the seal at about 20°C is 60% of its uncompressed width. For skewed joints, bridge deck movement

must be separated into components perpendicular to and parallel to the joint axis. Shear displacement of the compression seal should be limited to a specified percentage of its uncompressed width, usually set at about 22% (HE, 2019b).

Proprietary specialist cementitious materials should be prepared and placed strictly in accordance with the manufacturer's specifications and recommendations and be subject to the approval of the engineer. Because the joint is subjected to wear from vehicular traffic and therefore cannot be protected, all reinforcement within the nosing unit should be of non-corrodible steel, preferably stainless steel.

Mat expansion joints (bolted-down panel joints or reinforced elastomeric joints)

This proprietary bolted-down movement joint, which spans the joint gap, consists of prefabricated reinforced elastomeric units of variable lengths joined together by means of a tongue-and-groove (male–female) connection welded with special glue (Figure 21.11). The units are bolted down into blockouts formed in the concrete bridge deck on each side of the expansion joint gap. Expansion and contraction are accompanied by uniform stress and strain across the width of the panel joint between anchor bolt rows.

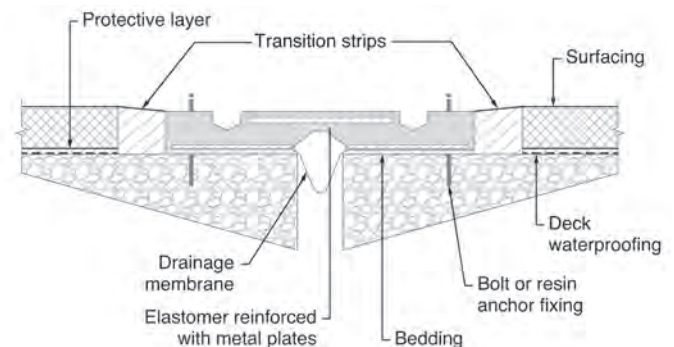


Figure 21.11 Example of reinforced elastomeric joint

For durability reasons, it is necessary that all expansion joints installed on highway structures are watertight. Because of the number of units that make up the joint and the consequential numerous connection points that exist, this type of joint is susceptible to leakages so therefore may not be considered watertight. A secondary collection of the seepage water taken to positive drainage is therefore necessary. The details of the rubber troughs and drainage pipes that will collect the seepage water will need careful consideration.

Unfortunately, the bolts and nuts connecting bolt-down panels to bridge decks are prone to loosening and breaking under high-speed traffic. The resulting loose panels and hardware in the roadway present hazards to vehicular traffic, particularly motorcycles. Care will need to be taken in the detailing of the anchor bolts to ensure that they remain tight and do not come undone in service. Where possible, tensioned cast-in bolts should be used to anchor these joints, or if site-

drilled installations are used, the holes should be under-reamed prior to fixing the bolts. Performance of the bolts needs to be confirmed by testing.

Cantilever expansion joints (comb, tooth or finger joints)

A cantilever joint is made up of 'male' and 'female' parts, individually bolted to each side of the deck joint gap. The female part consists of a steel plate with fingers fixed at one side of the expansion gap (Figure 21.12). The male part is an opposing plate with protruding fingers, which is fixed at the other side of the gap. As the bridge deck expands and contracts, the fingers of the male part slide longitudinally between the fingers of the female part, always maintaining contact with (and receiving support from) the base plate of the female part below.

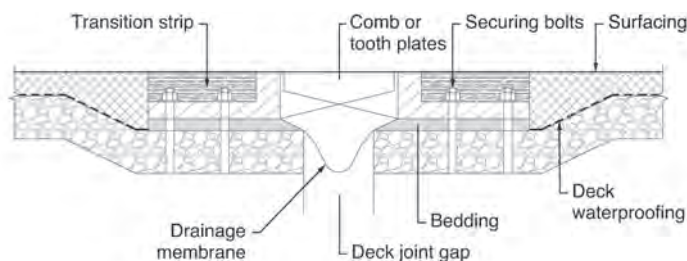


Figure 21.12 Example of finger joint

These joints are proprietary expansion joint systems, custom-designed and fabricated to individual specifications, and best suited when the primary movement is along the axis of the fingers themselves. They do not accommodate transverse movements very well.

Generally, steel finger joints do not provide a seal against water intrusion to substructure elements. Elastomeric or metallic troughs should be installed beneath the steel finger joint assembly to catch and redirect water and debris run-off. However, unless regularly maintained, these troughs clog and become ineffective.

Supported expansion joints

Supported expansion joints can be classified in the following subfamilies:

- bridging plate expansion joints without fingers
- bridging plate expansion joints with fingers
- roller shutter expansion joints.

In general, a supported expansion joint consists of one subcomponent flush with the running surface, which is fixed by hinges on one side and sliding supports on the other side (by a second element), and which spans the deck joint gap. The expected structure movement is allowed through sliding on the non-fixed side of the hinged subcomponent, that is, on

the supporting element that is anchored to the substructure.

Bridging plate expansion joints without fingers consist of a plate supported by a rotational bearing at one side and a sliding element at the other side, supported by a plate (Figure 21.13).

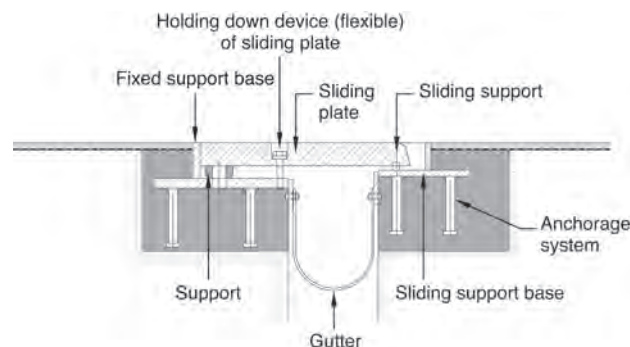


Figure 21.13 Plate expansion joint without fingers

Bridging plate expansion joints with fingers are composed of finger elements forming a sliding plate, one end of which is anchored and the other end of which is supported with the ability to slide on the opposite side of the gap (Figure 21.14). A second plate with fingers is attached on this side of gap and fills the spaces between the fingers of the sliding plate.

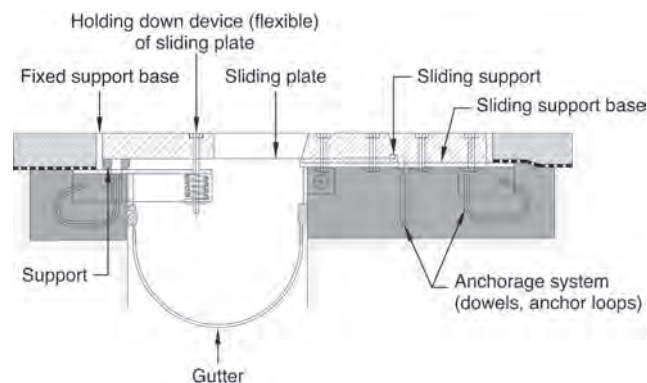


Figure 21.14 Plate expansion joint with fingers

A roller shutter joint comprises steel sliding plates hinged to each other and to the bridge plate, which slide on a slide frame to compensate for expansion movements (Figure 21.15). The fixed plate, lying tangentially, provides for a smooth and quiet riding surface. The faces of the hinged cams, sliding plates and slide frames are armoured by a special welding process to make them wear resistant. The bushes in the cams are also designed for a long life. The road surface areas are equipped with either grooves or a special resin compound incorporating quartz grit to prevent skidding. Neoprene strips are fitted in the gaps between the sliding plates to resist water and dirt. The roller shutters are provided with a steel channel to collect any water passing through the supporting surfaces of the fixed and the sliding plates to the substructure, and to direct it towards the drainage system of the structure.

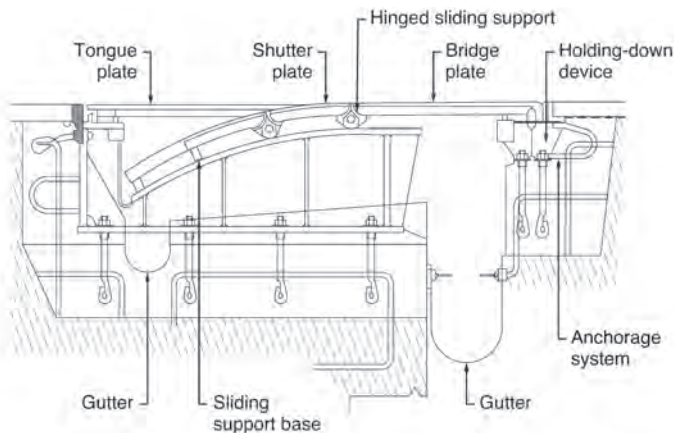


Figure 21.15 Example of roller shutter expansion joint

Modular bridge expansion joints

Modular bridge expansion joints are complex structural expansion joint systems, which comprise a series of centre beams supported atop support beams (Figure 21.16). The centre beams are oriented parallel to the joint axis, while the support bars span parallel to the primary direction of the movement. The support bars are, in turn, supported on sliding bearings mounted within support boxes. Polytetrafluoroethylene (PTFE)–stainless steel interfaces between elastomeric support bearings and support bars facilitate unimpeded translation of the support bars as the expansion gap varies. The support boxes are embedded in bridge deck concrete on each side of the expansion joint.

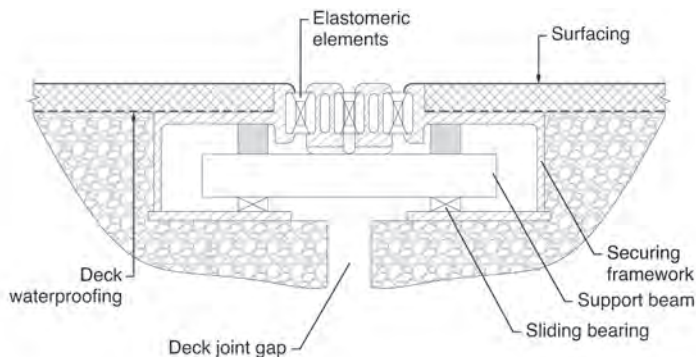


Figure 21.16 Example of modular expansion joint

Elastomeric strip seals or elastomeric box-type seals attach to adjacent centre beams, providing resistance to water and debris intrusion. Strip seals nominally allow the movement of 75 mm and thus the total movement distances of modular bridge expansion joints are designated in multiples of 75 mm. To minimise impact and wear on bearing elements, the maximum gap between adjacent centre beams is limited, typically to about 90 mm.

Installation and maintenance of expansion joints

Expansion joint components and connections experience severe loading under harsh environmental conditions. Long-term performance and durability of expansion joint systems require the synergistic application of high-quality fabrication, competent construction practices, assiduous inspection and routine maintenance.

Two of the observed causes of expansion joint failure are faulty installation and poor materials. Care should be taken in installing expansion joints, and manufacturers' recommendations shall be followed. Trained operatives should be employed, and particular attention paid to known points of weakness, such as the interface with the bridge deck waterproofing.

Expansion joints should be installed as late as possible in a bridge's construction to allow for shrinkage, creep and settlement movements to have taken place, at least in part, before the expansion joint gap is fixed. For joints fixed to the deck with cast-in anchors, the joint system is installed before the surfacing is laid. The anchors are cast into boxed-out recesses left in the deck or abutment, having been tied into the main reinforcement with suitable additional bars. Other joint types, which are either bonded or bolted to the concrete, are generally installed after the deck surfacing is in place, with the deck joint gap and waterproofing previously covered with a thin layer of plywood of the same width as the joint. The surfacing is then saw cut and removed, enabling the joint to be installed.

The proper functioning of the expansion joint must not be affected by accumulation of debris. Cleanability can either be achieved by a good design or by good accessibility to allow removal of debris. Good accessibility to the drainage system for flushing and maintenance must be provided.

To collect water that does leak through the expansion joint, it is common practice to provide a drainage system beneath the deck joint gap. This system should have adequate access for inspection and maintenance and allow discharge from the structure into a suitable road drainage system or soakaway. Water passing through the surfacing and running along the waterproofing should ideally be collected and discharged through a subsurface drainage system before it reaches the expansion joint. The interface between the expansion joint and the deck waterproofing should be watertight.

Waterproofing

Introduction

Waterproofing protects structures against water infiltration, which can cause expensive and irreversible damage. It is mostly invisible after construction is completed.

The use of de-icing salts for winter maintenance on highway bridges is causing considerable damage to the concrete

Element description	Protection system
Top surface of bridge deck	Spray applied waterproofing
Vertical faces at deck ends and abutment curtain walls	Protective coating
Top faces of piers and abutment bearing shelves	Protective coating
Inaccessible areas that may be subject to leakage, for example beam ends	Protective coating

Table 21.9 Protective system for exposed surfaces

bridge assets. The chloride salts penetrate the concrete and cause corrosion of the reinforcement and prestressing steel wherever run-off gets access, including splash zones. It is thus recommended to protect the non-buried concrete surfaces given in Table 21.9 with appropriate waterproofing system as a minimum.

Buried concrete surfaces below ground are subjected to attack by salt and other chemicals in the soil and by fluctuations in moisture level and temperature, all contributing to their deterioration. It is normal practice to apply waterproofing to all buried concrete surfaces more than 300 mm below finished ground level, such as the rear faces of abutments and wing walls. Waterproofing in these situations prevents water seeping through cracks and other defects to the exposed faces.

Different types of waterproofing system are briefly described in the following sections (Figure 21.17). However, the exact details will vary between manufacturers; the appropriate suppliers should be contacted to discuss particular details of their systems.

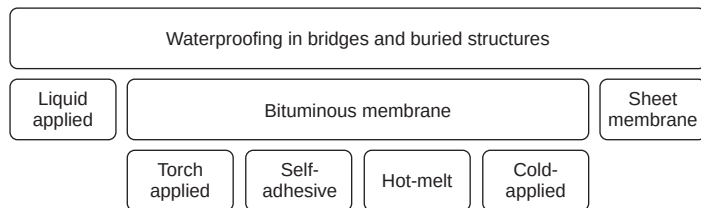


Figure 21.17 Waterproofing systems

Liquid-applied waterproofing

Liquid-applied bridge deck waterproofing

Liquid systems are mainly acrylic, epoxy, polyester, polyurethane or water-dispersible polymer-based (Figure 21.18). They are one- or two-part moisture or chemically curing solutions and can be applied by brush, squeegee or spray to form a seamless coating on the concrete surface (Figure 21.19). In particular, sprayed applied membranes can be rapidly applied and have good adhesion to the concrete.



Figure 21.19 Application of bridge deck waterproofing

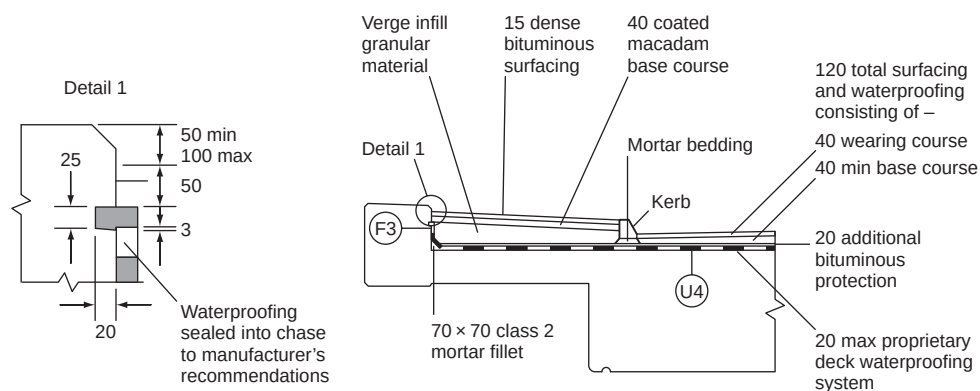


Figure 21.18 Typical bridge deck waterproofing detail (dimensions in mm)

Liquid-applied bridge deck waterproofing is not intended to receive direct vehicular traffic in service and in this case will always be used beneath overlays of asphalt or concrete, which may have a protective character or additional waterproofing function. Bridge deck waterproofing may remain uncovered when subject only to pedestrian or cyclist traffic or when used in non-trafficked areas.

Liquid-applied protective coating

Except where waterproofing or cladding is specified, concrete surfaces that are not in contact with the soil and are exposed to chemical attack should receive a concrete protective coating. These include concrete members that may be subjected to carbonation (XC class), chloride from seawater (XS class) or other sources (XD class) and de-icing salts, including splash zones (XF class).

Depending on the site conditions, the concrete classified may be subjected to more than one of the actions described in Table 1 of BS EN 206-1 and the environmental conditions to which it is subjected may thus need to be expressed as a combination of exposure classes. The exposure classes to be selected depend on the provisions valid in the place of use of the concrete.

The exposure classification does not exclude consideration of special conditions existing in the place of use of the concrete or the application of protective measures, such as the use of stainless steel or other corrosion-resistant metal and the use of protective coatings for the concrete or the reinforcement.

Based on the exposure and the place of application, the protective coating suggested needs to be fit for purpose and thus may be required to have one or more characteristics (Table 21.10).

Characteristic	Check whether applicable
Carbon dioxide diffusion resistance	It is not necessary that all these characteristics are applicable and need to be carefully captured before selecting the type of protective coating.
Chloride diffusion resistance	
Sulphate ion diffusion resistance	
Water vapour transmission – breathable	
Resistance to water absorption	
Waterproof	BS EN 1062 with various parts may be applicable as appropriate.
Active crack-bridging	
Abrasion-resistant	

Table 21.10 Typical characteristics of protective coating

Bituminous membrane

This system does not provide full tanking and needs to be used in addition to the drainage layer behind the walls. The drainage system collects water behind the walls into perforated pipes and discharges it to the nearest soak pit.

The bituminous membrane system in the walls and top faces of horizontal membranes is installed after the application of

primer materials, which are brush or roller applied.

Bituminous waterproofing membrane work in vertical members is normally followed with the placement of protection boards, within 5 days of the application.

Torch-applied bituminous membrane

Torch-applied bituminous membrane is a thin layer of water-tight material fully bonded to the surface. It is a flexible system, able to maintain its waterproof capacity without causing cracks. This is a fully bonded system and requires torching for the application. It is suitable for the retaining walls, abutments and sides of the buried box structure. However, it cannot be used for the bottom of the foundations.



Figure 21.20 Application of torch-applied bituminous membrane

Self-adhesive bituminous membrane

Self-adhesive bituminous membranes are self-adhering sheet membranes consisting of a rubberised asphalt compound or polymer-modified bitumen, with integral polyester fabric reinforcement, coated to one side of a polyethylene film (Figure 21.21). The membrane normally has a minimum overall thickness of 2.0mm, with a seldge to provide a bitumen-to-



Figure 21.21a Self-adhesive bituminous membrane

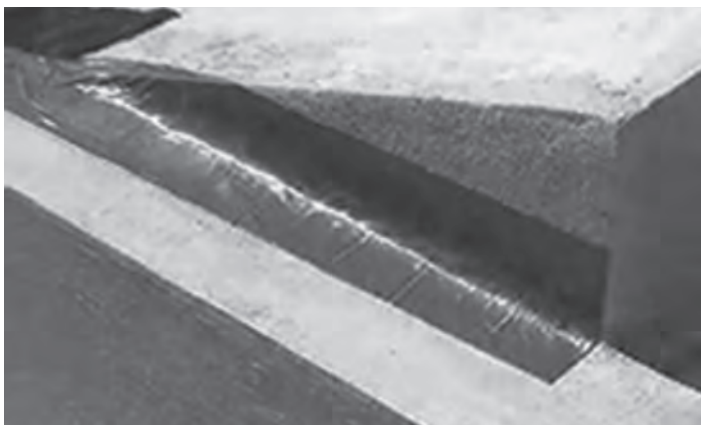


Figure 21.21b Self-adhesive bituminous tape

bitumen seal. The properties of the waterproofing membrane should not significantly diminish with age.

This is a fully bonded system; it does not require torching for the application and thus could be used for the bottom of the foundation in addition to walls of retaining structures. Since it does not require torching, it is safer and faster to install.

Hot-melt bitumen

Hot-melt bitumen is a polymer-modified bitumen, either elastomeric or plastomeric, packed in a solid block state. For application, the block is heated and applied to the substrate, forming a fully bonded waterproofing layer (Figure 21.22). Local in situ application offers an excellent alternative for areas with complex details.



Figure 21.22 Application of hot-melt bitumen

Cold-applied bitumen

Bitumen liquid-applied membranes are the perfect solution for seamless, fully adhered and safe systems (Figure 21.23). They are polymer-modified bitumen emulsions, which require no

torch or flame to be applied. They are available as water-based or solvent-based coatings. These are quite widely used for retaining walls, abutments and sides of the buried box structure since they do not require torching and form a fully bonded system with crack-bridging capability. This is quite safe and fast for application.



Figure 21.23 Application of cold-applied bitumen

Protection board

Ideally, bituminous membrane systems are protected with boards; these are multi-ply, semi-rigid boards composed of a mineral-filled asphaltic core between a layer of asphalt-saturated felt or kraft liner and a glass fibre mat liner that is weather-coated and has a polyethylene film facing. Protection boards are approximately 120cm wide, 240cm long and 3.5 mm thick, with at least 75% asphalt content by weight.

The puncture resistance of the board is a measure such that no effect is noted on the underside of the protection board following a drop test (68° tip angle of chisel, weight 0.619 kg, drop height 0.46 m, average puncture depth 1.1 mm).

Sheet membranes

Bridge underpasses, cut and cover tunnels and their approach structures, which are designed to exclude groundwater, must be externally protected by a waterproofing membrane. The waterproofing system should ensure watertightness and provide long-term protection to the concrete from the ingress of waterborne chlorides or other deleterious substances.

The waterproofing system forms an essential part of the watertight structures and therefore needs careful consideration in terms of the tanking system. Waterproofing systems for the base slab, walls and roof slab need to be compatible with each other so that junctions do not lead to weak spots. Waterproofing for such structures could be provided through the application of either liquid-applied waterproofing or a sheet membrane system. A combination of the two systems is quite

common, depending on the type of structure. However, care must be taken in terms of compatibility between the two systems to ensure that weak spots are not developed at the junction between the systems.

Sheet waterproofing membrane is normally a single layer of compartmented loosely laid or mechanical bonded, 2 mm thick, synthetic PVC sheet. These membranes extend under trenches, pits and depressions to suit the structural profile; the end laps are ideally staggered with lap joints double-seam welded and pressure tested to ensure watertightness. Care should be taken over the details of corners, membrane termination, pipe penetration, pile head treatment, and the like, for which the manufacturer should provide specific shop drawings, with details of reinforcing strips, collars, metal clamps and sealants.

Application of loosely laid membranes

Horizontal application

Normally, sheet membranes on horizontal surfaces are laid over a 370 g/m², nonwoven, needle-punched, polypropylene geotextile, which is placed over blinding concrete. Membranes are laid loosely over the geotextile and compartmentalised by welding a PVC water stop at predetermined locations (Figure 21.24).



Figure 21.24 Application of loosely laid sheet membrane

Vertical application

As a protection layer, 370 g/m² nonwoven, needle-punched polypropylene geotextile is laid on all vertical and sloping surfaces to protect the waterproofing membrane from damages from placing and vibrating concrete and to reduce drag-down on the membrane by the placing action of the concrete.

Application of mechanically bonded membranes

Mechanically bonded membranes are locked into the structure by ribs when concrete is poured directly against them. These membranes are laid directly over the blinding concrete, which

needs to be free from protrusions and thus should have wood float finish. Concrete cover blocks shall be used over the membrane for the placement of steel reinforcement to hold the bars securely in place.

For vertical members, the membrane should be mechanically fastened using appropriate fixings, such as G-clamps or wooden battens. Welding for vertical and horizontal surfaces is normally done with an overlap of 50 mm at both sides, with ribs facing upwards. Welding is ideally carried out using hot air welding equipment. Joints are tested for integrity.

Drainage

In the initial concept stage of the design of a bridge structure, consideration should be given to each element of the structure to ensure that the effects of water, particularly when contaminated with road salts, do not seriously affect the life of the structure.

To minimise surface and subsurface erosion and to reduce lateral hydrostatic pressure behind and between the bridge abutment, wing walls and retaining walls, a proper back-of-wall drainage system is essential for these elements of the bridge.

Water trapped within the road surfacing on the high side of a deck joint can, through hydraulic pressure from wheel loading, cause failure of the bond or seal between the joint and the waterproofing systems.

Back-of-wall drainage for abutment and retaining walls

An effective drainage system within the approach fill behind the abutment and walls consists of a medium located immediately adjacent to these elements that will allow migration of water to a bottom drain that will move the water away from the area (Figure 21.25). This can consist of 150 mm diameter porous drainpipe located on top of the wall base, which can be enclosed in no-fines concrete. Above this, for the full height of the wall, a drainage medium is provided. This can take the form of hollow concrete blocks or a free-draining granular material. In some cases, the drainage system could be composed of

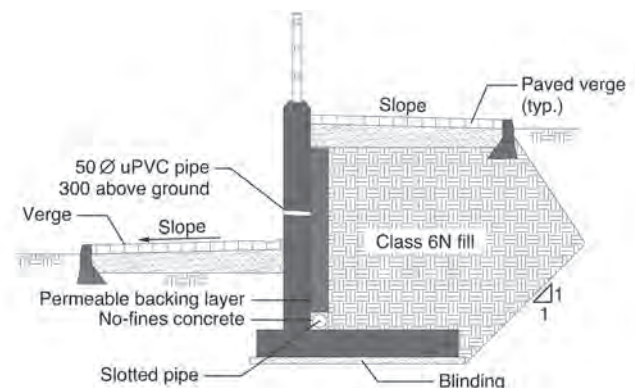


Figure 21.25 Back-of-wall drainage (EGL, exposed ground level)

pervious material, such as gravel or a geocomposite drain and filter fabric, which is a synthetic waffle-like material that channels water vertically downward.

Weep holes should be provided in all high walls, in case the porous drainpipes become inoperative for any reason. They should outfall just above the external paved surface and, ideally, have a reverse fall to prevent continual dripping. In urban areas, small-diameter pipes, say 50 mm at 3 m centre spacing should be used; larger pipes often attract vermin and are also frequently blocked by litter, such as cans or bottles.

The maintenance requirements for the drain must be considered in the design. The pipes should be straight, albeit on a gradient, with catch pits or manholes at each end to enable the pipe to be cleaned by rodding or water jetting. Where the pipe is longer than approximately 80 m, an intermediate manhole should be provided as, above this length, rodding and water jetting becomes difficult in a rough pipe.

Depending on their lengths, wing walls should be treated in the same way as abutment walls. However, for short walls, it is common practice to omit the pipe and allow water to flow to the abutment drainage media through its own drainage medium, for collection by the abutment drain.

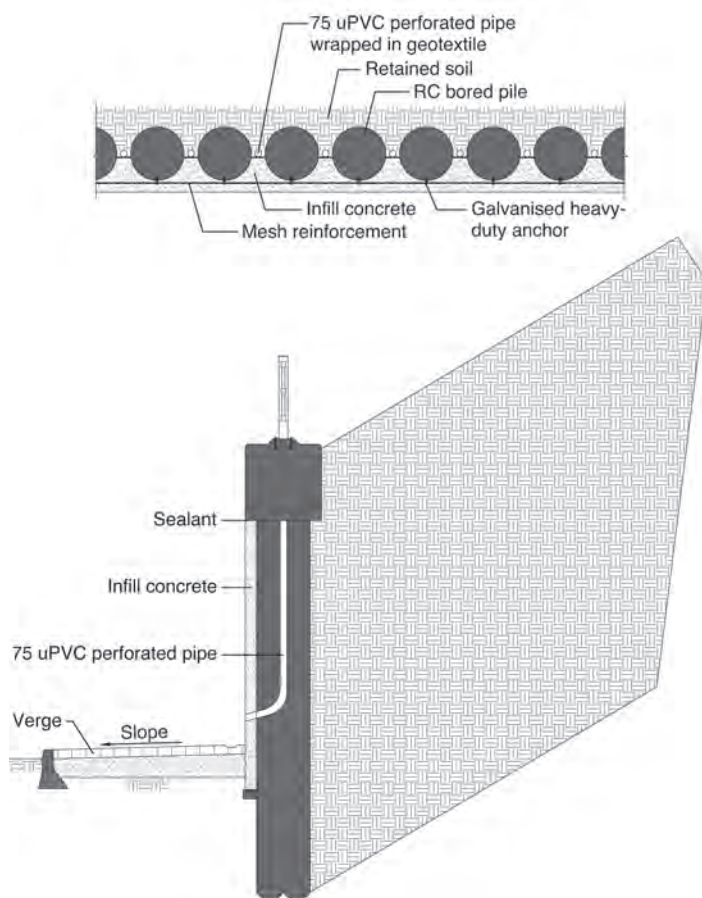


Figure 21.26 Drainage in front of fascia for contiguous wall with in-fill concrete

Drainage in contiguous bored pile wall

Where contiguous bored piles, or similar, are used as the structural abutment or retaining wall, water penetrating the piled wall needs to be collected and discharged into the drainage system on the outer face of the wall (Figure 21.26).

In the case of contiguous bored piles with fascia, the wall drainage system requires porous uPVC pipe to be wrapped with geotextile to prevent fine soil particles from entering and blocking the pipe. These perforations are required only to the back of the pipe; the front face of the pipe is intended to be covered with shotcrete up to a depth of 200 mm. The shotcrete supports the vertical clay soil surface between piles in addition to the perforated pipe (Figure 21.27).

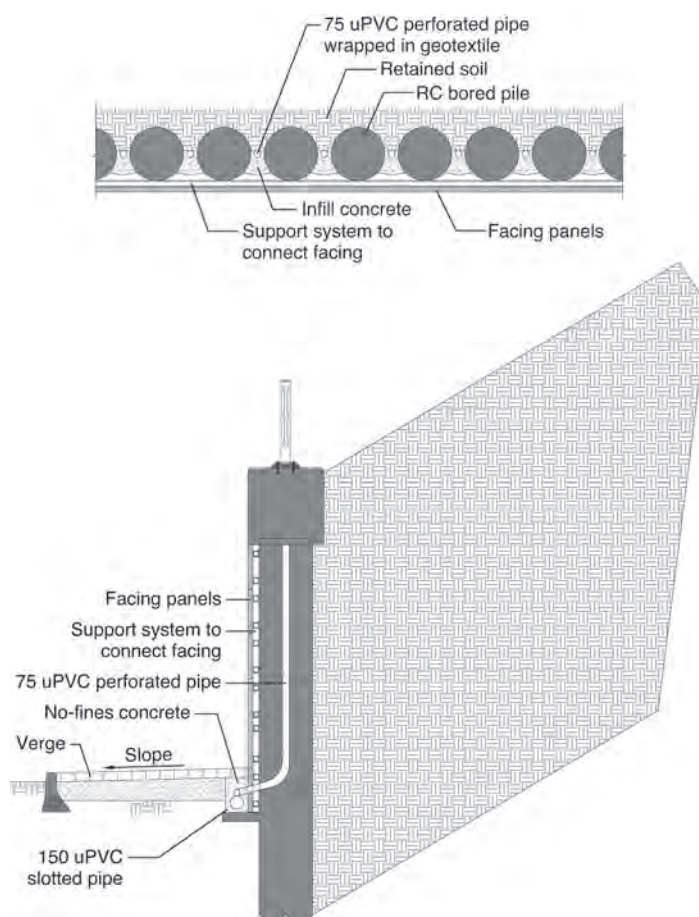


Figure 21.27 Drainage in front of fascia for contiguous wall with shotcrete

These perforated pipes could be connected to the underground drainage or may be allowed to drain freely over the ledge in front of the wall. The option needs to be investigated based on the project requirements.

Alternatively, a gap could be established behind the precast concrete panels and piles so that the drainage system is concealed behind the fascia (Figure 21.28).

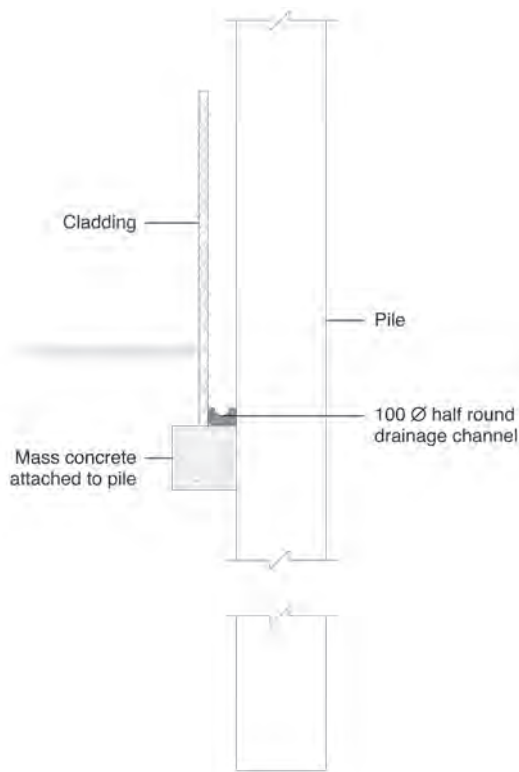
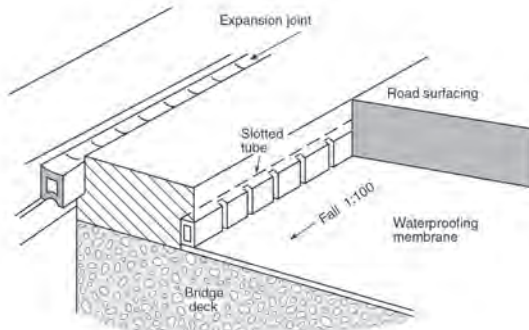


Figure 21.28 Drainage in contiguous wall behind the fascia

Subsurface drainage

Water can collect between the bridge deck surfacing and the waterproofing and cause the surfacing to bubble and delaminate in freeze–thaw conditions. This build-up can be released using transverse slotted drains laid parallel to the joint above the waterproofing (Figure 21.29). Alternatively, a transverse channel in the top of the bridge deck, parallel to the joint with the downpipes can be used to collect the subsurface water and discharge it into the gallery. Ideally, the outlet tubes should be cast into the deck and not drilled through as an afterthought.



Carriageway drainage

The deck drainage must be considered early in the design as it can have a significant influence on the overall bridge configuration. Water should be prevented from entering the bridge at the approaches. Surface water on the bridge deck should be removed by the provision of falls and suitable drainage outlets. Carriageway drainage on bridge decks can cause problems where gullies or downpipes penetrate the deck and its associated waterproofing. Where possible, this should be eliminated by the use of gradients, hard strips acting as reservoirs or drainage channels.

Where drainage is to be provided, the kerb gully inlet type, although more expensive, is favoured because it can be installed on top of the deck and without penetrating the waterproofing. This system is also suitable where porous asphalt surfacing is specified (Figure 20.30). Where this system is not suitable, and a gully and pipe system is deemed necessary, care should be taken to ensure that all penetrations are well detailed to prevent water from escaping around rather than through the system.

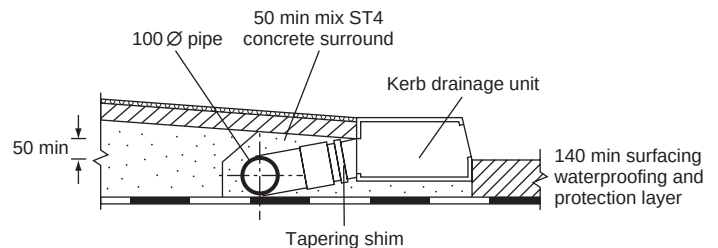


Figure 20.30 Subsurface drainage arrangement (dimensions in mm)

A free-falling drain system at close and regular intervals, located in the roadway adjacent to kerbs, with vertical 150–200 mm diameter pipes of minimum practical length, with no bends if possible, is efficient, self-cleaning and low-maintenance. Large drip moulds on the circumference of the pipes, where they exit the concrete section, prevent water dripping down the deck cantilevers. Free-falling systems should

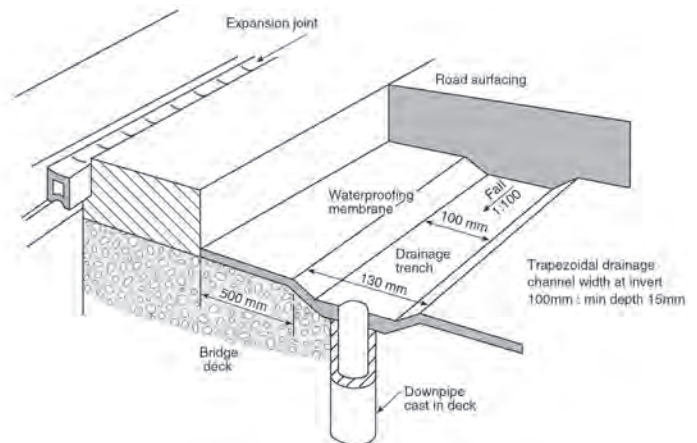


Figure 21.29 Subsurface drainage arrangements

extend below the underside of deck cantilevers or deck soffit and be placed away from the piers and abutment to avoid wind-driven spray on bridge members. Care must be taken that no erosion or damage occurs underneath. Water should never be dumped on a surface that lacks erosion protection, such as rip-rap, a paved slab, splash block or an open basin. The system should be such that water is not allowed to fall freely from the bridge deck, nor should water be discharged into the drainage layers behind abutments. Service bays must have provision for drainage. Facilities for rodding and other necessary maintenance should be incorporated, and systems should be sufficiently robust to withstand damage during cleaning and be resistant to all chemical spillage commonly occurring on the network. Drainage details integral with the structure, such as downpipes cast into piers, should be avoided where possible.

Cases where the drainage pipe beneath the gulley and pipe cannot be avoided; necessary considerations include the following.

- Pipe gradients should be maximised to increase flushing action.
- Consider oversized pipes to reduce the potential for blockages.
- Provide traps with simple access for cleaning wherever feasible.
- Bends and turns must be kept to an absolute minimum.
- Pipe material must be robust and non-corroding (internally and externally).
- Pipe joints must be robust and leakproof.
- Rodding access must be provided to pipe runs.
- Particular attention is required where pipes pass through the structure, particularly at abutments where relative movement is anticipated between the deck and the substructure.
- The length of pipes to be cast into structures should be minimised.

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Chapter 22

Protection

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The protection of bridge structures is central to the operation of any transport infrastructure and bridge owners seek protection methods that represent an efficient use of the available funds while maintaining adequate serviceability. Investigations of deteriorated structures suggest that problems arise from design errors, errors in the materials specification or construction method employed and poor workmanship or quality control. As a consequence, protection methods should be an integral part of the initial design and materials selection, reflected in the quality of construction employed, and linked to planned inspection. Given the importance of water management, good design should incorporate modifications that reduce the risk of unexpected deterioration while facilitating maintenance processes. At the same time, the bridge engineer can select from a number of active and passive methods for protecting bridge structures, including enclosures, coating systems, cathodic protection and the use of corrosion inhibitors.

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Introduction

The existing stock of bridges within the UK, and many other countries, comprises a complex mix of structural form, span size, age and material type, which reflects the ongoing development of transport infrastructure over many generations. With appropriate management, the majority of these structures may be expected to continue to perform their required function for many years but it is an unfortunate fact that bridge failures do occur. Such failures can be sudden and catastrophic, leading to partial or complete collapse, with the associated risk of loss of life (Levy and Salvadori, 1994). Alternatively, failure can be a subtle, and progressive, loss of serviceability, leading to increased management costs, reductions in capacity and the eventual need for strengthening or replacement.

The protection of bridge structures is clearly of prime importance to those responsible for the maintenance of an ageing transport infrastructure and represents a considerable technical challenge and ongoing financial commitment. As a consequence, most operators seek protection methods and maintenance strategies, which represent an efficient use of the available funds, in terms of both cost per structure and serviceability over an appropriate time period. In considering whole-life costs (Ryall, 2001), it must be remembered that bridge failure inevitably carries an implication for the surrounding transport infrastructure in terms of reduced access, delays and longer journey times (and greater distances) for transport users. The indirect costs of these on the economy can be significant and may exceed the cost of bridge repair or replacement.

While the deterioration of an ageing bridge stock is almost inevitable, investigations of deteriorated structures suggest that many problems arise from design errors, errors in the materials

specification or construction method employed and poor workmanship or quality control (Shaw, 1987). As a consequence, protection methods should be considered as an integral part of the initial design and materials selection, reflected in the quality of construction employed and clearly linked to any inspection and maintenance programmes. Indeed, many bridge structures exhibit acceptable durability over their design life. Such structures demonstrate that the application of appropriate design codes and materials selection coupled with good construction practice and an appropriate maintenance programme can yield durable and cost-effective structures.

Causes of deterioration

The deterioration of bridge structures occurs primarily through one of three causes.

- *Chemical reactions* between the material from which the structure is constructed and the environment to which it is exposed. It is an important observation that all of the materials commonly used in the construction of bridge structures are thermodynamically unstable with respect to their environment. For example, metallic iron (which is the basis of steel) is unstable with respect to the Fe(II) and Fe(III) ion species and will readily oxidise to FeO or Fe₂O₃, or their hydrates, in the presence of liquid water and oxygen. Similarly, the calcium hydroxide formed within hydrated Portland cement paste readily reacts with a range of materials in the environment, such as carbon dioxide and sulphates, to form more stable, but often less useful, products.
- *Physical processes* that result from the interaction between the structure and some vector within the environment, resulting in either direct loss of serviceability or producing conditions that allow other deleterious processes to be initiated. For example,

some engineering polymers used as protective coatings undergo physical swelling and disbondment, owing to the adsorption of water over long periods. Similarly, masonry and brick materials can be damaged by the expansion of water as it freezes within the pore structure inherent in such materials. In metallic structures, the initiation and growth of fatigue cracks is driven by the application of cyclic stresses but can be accelerated in the presence of a moist environment containing chloride ions.

■ **Extreme events**, which impose transient forces or displacements that lie outside of the normal design envelope. For example, many bridge structures have suffered partial or complete collapse, owing to the action of earthquakes (Chen and Duan, 1999), wind effects (Brown, 1993) and flooding (BBC, 2005). In addition to these natural phenomena, some of which may be increasing because of changes in the global environment, bridges also find themselves subject to accidental damage by users (e.g. vehicle impact), deliberate vandalism (Scheerhout, 2007) and even terrorist attack (Tran, 2000).

In reality, most bridge structures are at risk from a variety of deterioration mechanisms that can act independently or jointly. The level of protection provided depends on the level of perceived risk that such a mechanism (or combination of mechanisms) can occur over the design life of the structure and the possible consequences for serviceability. Thus, while all-steel bridges are routinely protected from the certainty of aqueous corrosion, only some are designed to resist accidental boat impact acting on their supports, and none is designed to resist meteor impact!

The role of time in deterioration of structures

In most cases, the deterioration seen by the bridge engineer is the result of a combination of physical effects induced by some underlying chemical, or electrochemical, reactions that have been occurring over a considerable period of time. For example, the deterioration of reinforced concrete bridge structures owing to corrosion of the embedded steel manifests itself as physical cracking and spalling of the cover concrete. This results from the formation of voluminous oxide products that have the effect of placing the concrete surrounding the steel in tension, causing the formation of cracks in the cover concrete along the line of the reinforcement. The onset of aqueous corrosion of the steel can occur many years before any outward signs of physical deterioration can be seen (Figure 22.1). The time between initial construction, t_0 , and the initiation of aqueous corrosion, t_1 , represents a period during which a reinforced concrete structure may be considered as being relatively 'maintenance-free'. In practice, the period $t_0 \rightarrow t_1$ can be significantly increased by a combination of good design, quality construction and careful upkeep of the water management system; see the section on material selection and design. Once the initiation of corrosion has occurred, some unplanned cracking of the cover concrete becomes almost inevitable (Concrete Society, 2015). However, if the bridge

engineer takes steps to control the rate of aqueous corrosion, for example, by keeping the structure as dry as possible, then the time before cracking of the cover concrete occurs, t_2 , can be significantly increased (Darby *et al.*, 1996). Thus, by using appropriate methods, a bridge structure can often be successfully operated for many years in a state of controlled deterioration.

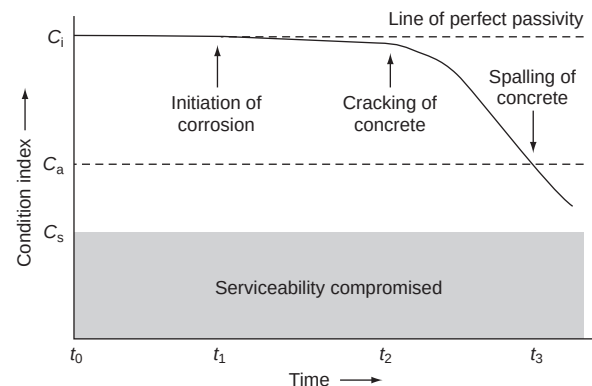


Figure 22.1 Idealised change in the condition index of a reinforced concrete structure as a result of corrosion of the steel reinforcement: during the early stages of the structure's life, $t_0 \rightarrow t_1$, the steel remains passivated by the high pH of the surrounding concrete; initiation of corrosion, whether by carbonation of the cover concrete or the ingress of chloride ions, occurs at time t_1 and the condition index begins to deviate from the line of perfect passivity; subsequent corrosion results in the formation of an expansive oxide film at the steel–concrete interface, inducing tensile stresses in the cover concrete, which eventually cracks, t_2 ; once the concrete has cracked, the rate of corrosion generally increases and eventually spalling of the cover concrete occurs, t_3 (C_i , initial condition; C_a , limit of acceptable condition; C_s , limit below which serviceability is compromised)

The use of controlled deterioration represents a valuable tool in the protection of bridge structures and finds its logical outcome in the process of corrosion allowance. In essence, this is the basis on which all weathering steel bridges are designed; see the section on metal bridges. Similarly, it is arguable that the changes in design codes for reinforced concrete to greater cover depths of stronger concrete, for a given exposure class (BSI, 2000), are intended to allow for the deterioration processes that are expected to occur; for example, the diffusion of chloride ions through the cover concrete. Ideally, such allowance will seek to delay spalling of the cover concrete, or major loss of steel sections, to the point where either the structure is no longer required or planned repairs can be implemented. This approach does not seek to stop the initiation of deterioration but aims to manage its impact on the structural reliability. However, problems can arise when the local rate of deterioration within part of the structure exceeds that shown generally, raising the possibility of unexpected failure. The collapse of the bridge at Ynys-Y-Gwas greatly increased concern regarding the durability of conventional post-tensioned construction and highlighted the fact that very localised deterioration can result in structural instability (Woodward and Williams, 1988).

Strategies for combating deterioration

While there is no universal method for protecting bridge structures from deterioration, it is possible to identify a number of useful strategies for protecting a bridge structure over its design life. Where deterioration is primarily driven by the interaction between a structure and the environment to which it is exposed, three main approaches may be adopted.

- *Remove the environment.* All of the main chemical reactions, and many of the physical interactions, that lead to the degradation of bridge structures require the presence of water. As a consequence, if a structure is either never exposed to a wet environment or designed so that, when wetting does occur, rapid drying can take place, deterioration will be kept to a minimum. This approach is fundamental to all water management systems and represents a key approach in the effective protection of any bridge stock; see the section on water management.
- *Alter the material or structure.* One effective method of protecting bridge structures from deterioration is the selection of appropriate materials for the particular environment to be encountered; see the section on material selection and design. Having selected an appropriate material, care must also be taken in the detailed design of the structure. This should ensure that potential causes of deterioration, such as the possibility of standing water or bimetal contact, are identified and removed from the design. At the same time, the designer should avoid details that are unnecessarily complicated to build under site conditions. This requires the engineer to think beyond the normal limits of the available design codes and take control of the ease, and reliability, with which the structure can be constructed. Finally, good design should ensure safe and simple access to critical parts of the structure (Figure 22.2). This ensures that component parts of a structure can be easily inspected and any maintenance or repairs carried out in a cost-effective manner.
- *Protect the structure from the environment.* For many engineers, the most direct method of protecting bridge structures from deterioration is by the use of surface coatings; see the section on coating systems. Inert barriers rely on the exclusion of the environment but suffer from the inherent problem that where the coating becomes cracked, or broken, any protection is compromised (Hartley, 1994). One solution to this problem is the use of sacrificial layers that actively deteriorate in preference to the underlying structure, for example, galvanised steel; see the section on coatings for steel structures. Hydrophobic surface treatments do not attempt to form an impermeable layer but rather alter the wetting characteristics of the surface. The use of silane and siloxane surface treatments to protect concrete and masonry surfaces is now widespread. These have the advantage of allowing water vapour to pass unhindered, hence promoting drying of the underlying substrate (Nwaubani and Dumbelton, 1997).

In addition to coating systems, the bridge engineer also has available a number of other methods for protecting metallic components from corrosion, including cathodic protection systems and the use of corrosion inhibitors; see the section on water management. The protection of bridge structures from physical processes such as scour is considered in the section on protection from physical processes.



Figure 22.2 Provision of safe and simple access to bridge bearings and half-joints

Water management

Water plays a central role in many of the processes that lead to the deterioration of bridge structures. As a consequence, designers and maintenance engineers must appreciate the need to carefully manage the flow of water as it passes onto, over and eventually away from a structure (Pearson and Cunningham, 1998). An important outcome of water management concepts is the recognition that there are two main approaches that may be adopted by those seeking to protect bridge structures from deterioration.

The first approach seeks to prevent all contact between water and those parts of the structure that are known to be prone to deterioration. Such exclusion systems are commonly based on a number of barrier layers and associated protective measures, with the objective that if water breaches one layer then it will be kept back, for an acceptable period, by the other layers of protection. An inherent danger of this approach is that, should water enter through failures in the continuity of the protective system, it can remain trapped for long periods, providing ideal conditions for long-term deterioration.

The second approach acknowledges that, however well-designed the water management system, water will eventually reach potentially vulnerable parts of the structure. This is especially true if the effects of water vapour and sprays are taken into account. Having accepted that it is impossible to exclude water totally, steps can be implemented to ensure that any water that does enter a structure is removed as quickly as possible. This approach encourages the designer to explicitly consider how water falling onto a structure moves over, through and off it. As a consequence, it tends to produce structures that are more easily inspected since provision has to be made for the maintenance of the drainage system. The success of a design based on this approach can be simply determined by inspecting the structure 2h after significant rainfall. Where water remains ponding on, or adjacent to, critical areas of the structure, the design can be said to have failed.

A variant of this second approach is the use of enclosure systems that are designed to create a local environment around structural members that is relatively benign over the annual cycle, with corrosion rates on bare steel falling to 5%–10% of that experienced outside (McKenzie, 2002). Such enclosures are not impermeable barriers but act to significantly reduce the ingress of contaminants carried onto critical components by rain and wind-blown mist. At the same time, the enclosure system allows any water that does gain entry to be removed by either drainage or evaporation. Current evidence is that painted steel members protected within well-designed enclosures should remain maintenance-free for much longer periods of time than is otherwise possible (Highways Agency, 1996a, 1996b). Such enclosure systems have the advantage that they can be retrofitted to existing structures to both reduce ongoing corrosion problems and provide access for future inspection and maintenance.

Drainage

The primary drainage system has two main functions. First, it should ensure the removal of standing water from the main deck. Second, it should channel water away from the main structural elements so that they remain dry for as long as possible. To achieve these aims, a well-designed drainage system must fulfil the following criteria.

- It must have the capacity to collect any water that arrives on the surfaces of a bridge, including direct rainfall, condensation and spray from moving vehicles.
- It must control the flow of this water from the running surface and divert it through, or around, the structure without it coming into contact with the main structural elements or other critical components.
- It must disperse the water collected away from the structure so as to both protect the substructure and foundations and also ensure that it does not cause problems for adjacent roadways or other structures.

The capacity of a drainage system must reflect both the area of the structure likely to collect water and the rate at which it arrives. While it is possible to calculate the capacity based on annual rainfall data, it is more usual to design for the extreme values likely to be encountered over the design life of the structure, for example, the once-in-a-hundred-year storm. Inadequate capacity of the drainage system can seriously influence the safety of vehicles using the running surface, as standing water can lead to both poor visibility and reduced skid resistance. In contrast, inadequate capacity only indirectly affects the long-term durability of a structure. Ultimately, such problems of deck drainage can be avoided by the use of open, metal-grill style running surfaces, such as are encountered in parts of North America. However, the inherently low skid resistance of such decks makes them unsuitable for many applications.

Once water has fallen onto a structure, its flow can be controlled by a number of methods. First and foremost of these

are the natural gradients that exist both in the vertical and horizontal alignment of the bridge deck and the cambers of the running surface. Where the deck is inclined at an angle, this must be taken into account when designing both the position and capacity of the main drainage points on the deck. Since it is not feasible to rely solely on these natural gradients to control the movement of the water, most successful drainage systems rely on water being directed off the deck by the presence of suitable kerbs and gullies. In utilising such methods, it is important to remember the presence of joints which, if not properly sealed, will allow water to leak onto bearing shelves and other critical components.

Having designed a drainage system that carefully collects and diverts water from a structure, it is important to ensure that the water is channelled away from the structure and not simply allowed to drip over retaining walls or other parts of the substructure. Where this does happen, staining and long-term frost damage can occur. As a consequence, weep holes and drainage pipes should be designed so that their outfall lands away from the structure. This can be achieved by ensuring that such outlets protrude sufficiently to ensure any discharged water does not flow over the lower part of a wall. Alternatively, the drainage pipes should be extended to direct the water into a gully or drainage channel. Water that leaves the drainage system can be deposited into either a main drain, incorporating suitable storm storage capacity, or local soakaways.

In attempting to control the flow of water, it is important to distinguish between the behaviour of bulk water in drains and gullies and that of thin films of water on the surfaces of a structure. Bulk water can generally be relied on to flow in the direction of gravity, following the line of least resistance and hence can be directed with some degree of certainty, although this is subject to limits imposed by the capacity of the drainage system and the vagaries of wind action, coupled with the influence of vehicle movements that can simultaneously transport water onto a structure and create sprays and fine mists of water.

In contrast, the behaviour of water in thin surface films is dominated by the influence of surface tension and will consequently 'hang' upside down on, and flow over, inclined surfaces rather than drip or fall from the surface. On concrete surfaces, this effect is magnified by the capillary absorption exerted by the structure of the cement paste that both encourages the water to spread onto the surface of the structure and helps hold it in place against the pull of gravity. There are a number of important outcomes of this behaviour. The first is the need to include grooves in the soffits of edge beams to encourage water running down the sides of beams to run off the edge of the beams as drips rather than run onto and flow along the bottom of the beams (Figure 22.3). It is necessary to ensure that any water that is forced to drop from the beam does not subsequently fall onto other parts of the structure. Second, it means that water can unexpectedly find its way into parts of a structure that would normally be considered safe from water ingress, for example, bearing shelves. As a consequence, it is useful to design bearing shelves to be self-draining. Typically, this can be achieved by placing the bearing on a suitable raised

plinth that has adequate air space around it to help promote drying (Figure 22.4).

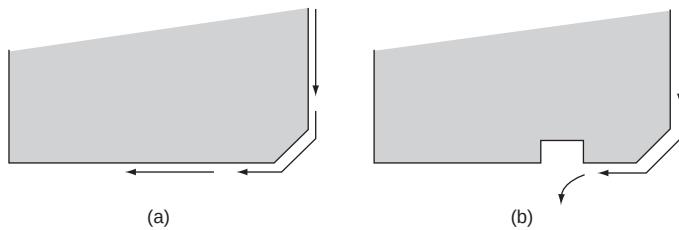


Figure 22.3 Idealised flow of water over the edge of a concrete beam: (a) onto a flat soffit and (b) onto a flat soffit with integral drip feature



Figure 22.4 Raised plinth for protection of bridge bearing: the water ponding around the raised plinth is the result of inadequate design or maintenance of the drainage outlet, which has become blocked with debris (this photograph was taken less than 6 years after the bridge was first constructed)

Having made adequate provision for drainage, consideration should also be given to how the system can be maintained. This is because, even after only a few years in operation, road debris, dead animals, litter and dust can collect in the drainage system and either compromise the maximum capacity or block the drain entirely (Figure 22.4). A drainage system that cannot be adequately maintained using easily available equipment and standard methods will inevitably contribute to poor long-term durability.

Waterproofing membranes

It is the role of the waterproofing membrane to augment the drainage system by forming an impermeable layer over the upper surfaces of a bridge structure that prevents the water from running into the underlying materials, so that it instead passes into the main drains. To achieve this, the waterproofing membrane should be continuous over the part of the structure to be protected and seamlessly fit into the main drainage channels. The effectiveness of the concept of waterproofing layers can be illustrated by comparing concrete bridges in the UK with similar structures in the USA, which, until recently, did not employ such layers and have arguably deteriorated more rapidly than originally envisaged (Lee *et al.*, 2005).

Waterproofing membranes are distinct from the layers of asphaltic or bituminous bound material that make up the running surface of many bridge decks. While such materials are inherently water-resistant, they are not designed to be complete barriers against water ingress. Indeed, porous asphalt surfacing deliberately holds water within the surface of the road to prevent water ponding and reduce dangerous road spray.

Typical materials used for waterproofing are organic in origin and range from simple sprayed bituminous layers to complex binary polymers. All such materials must meet a number of technical requirements.

- They must be capable of forming a continuous film on the surfaces to which they are applied.
- They should have good stability to long-term contact with saline water and common roadside pollutants, such as petrol, diesel and other organic solvents.
- They should be able to resist the effects of construction traffic without failure.
- They should be capable of resisting the application of bituminous and asphaltic layers used to create the final running surface.
- They should be capable of resisting the normal thermal and load-induced movement of the underlying structure without the formation of holes or breaks, which might allow the passage of water.

Waterproofing products currently in use around the world can be classified into two distinct types.

Preformed membrane layers

These are heavy-duty films, typically 1–3 mm thick, manufactured from a variety of polymers that have been reinforced with fibres and other products to improve their mechanical properties and handling characteristics. They are supplied on rolls several metres wide and tens of metres long. These membranes are effectively glued onto the surface of the bridge deck using a compatible primer and adhesive. Care must be taken to ensure that no sharp objects remain on the deck surface that could puncture the membrane under traffic. Once in place, such membranes are normally sufficiently tough to withstand pedestrian movements and limited traffic from rubber-wheeled vehicles. In practice, the asphaltic or bituminous materials that are to be used in the road are applied as soon as possible. A good bond between the membrane and the road material can be generated when the heat from the bitumen melts the topmost surface of the membrane.

Given the finite width of the rolls on which the material is supplied relative to the width of a typical bridge deck, there is a need to lap adjacent layers to ensure continuity of the completed layer. Alternatively, special lapping rolls may be glued into place to seal over the joints. Attention is needed to fit the membrane system directly into the main drains. Careful adherence to cleanliness and the manufacturer's instructions can produce joints that are no more prone to water penetration than

the membrane itself. However, the production of defect-free joints is by no means certain, and subsequent inspection and testing of the completed membrane is mandatory.

Liquid-applied layers

These are, typically, two-part liquid resins that are sprayed using specialised machinery onto the cleaned deck, where they react to form a thin, continuous polymer membrane that is impervious to liquid water. Such coatings have the advantage over reinforced membranes in that they can be sprayed to form seamless layers that are directly lapped into the drainage system. Once in place, however, these relatively thin layers are prone to damage from both foot and vehicular traffic, from which they should be protected until covered by the layers that make up the road above. Since 1986, a red sand asphalt carpet has been used in the UK as a protection for such layers when they are overlaid with mixtures containing large aggregate particles. To be effective, it is important that the red sand asphalt carpet is laid and compacted at temperatures high enough to form a dense layer that bonds firmly to the waterproofing.

Expansion joints

Regardless of the type of waterproofing membrane employed, the need for continuity of the waterproofing layer is compromised by the presence of joints provided either to simplify the construction process or to allow movement of the structure in response to thermal and moisture gradients. The use of half-joints was common practice from the 1960s to the 1980s, allowing for both ease of construction, for example, drop-in prestressed beams on reinforced-concrete (RC) cantilever supports, and simple articulation, to allow for thermal movements. A particular problem affecting such structures is the leakage of water contaminated with de-icing salts through the joints at the end of spans (Figure 22.5). In principle, this should not be

a problem, since all such joints are designed with either static or moving seals. Unfortunately, the development of effective methods of sealing such joints remains problematic, with many joints failing after only 5 to 10 years in service.

Three main types of expansion joint have been in common use in the UK (Figure 22.6). In a survey of the condition of such expansion joints in 250 road bridge structures, only 25% of the joints examined were free of defects and many instances of poor detailing were found (Johnson and McAndrew, 1993). About half the asphaltic plug joints surveyed were found to be leaking, and tracking was common in heavily trafficked lanes, while cracking and debonding were observed more on lightly trafficked areas. Nearly 65% of the reinforced elastomeric joints surveyed were leaking and half had cracked transition strips; many were not flush with the road surface so that ride quality was affected and impact damage occurred on the edges of the joint. The elastomeric joints in metal runners (Figure 22.6(b)) were the most waterproof, with only 40% leaking. It is clear that these findings have significant implications for both the maintenance costs of existing bridges and the design of new structures. One solution to this problem may be summarised as ‘the best expansion joint is no expansion joint at all’ (Iles, 1994). This has led to significant developments in the design and construction of integral bridges. Naturally, the elimination of expansion joints means that thermal changes have to be resisted by thrust into the ground at the ends of the bridge (Highways Agency, 2003).

Material selection and design

New bridge structures in the UK typically have a design life requirement of 120 years. Arguably, the most cost-effective method of ensuring the long-term protection of bridge structures lies in the selection of appropriate materials for the particular environment to be encountered. For example, blended cement concrete incorporating blast-furnace slag has been specified for



Figure 22.5 (a) Failure of expansion joint in road bridge; (b) associated leakage of water through the unprotected half-joint (these photographs were taken less than 6 years after the bridge was first opened to traffic)

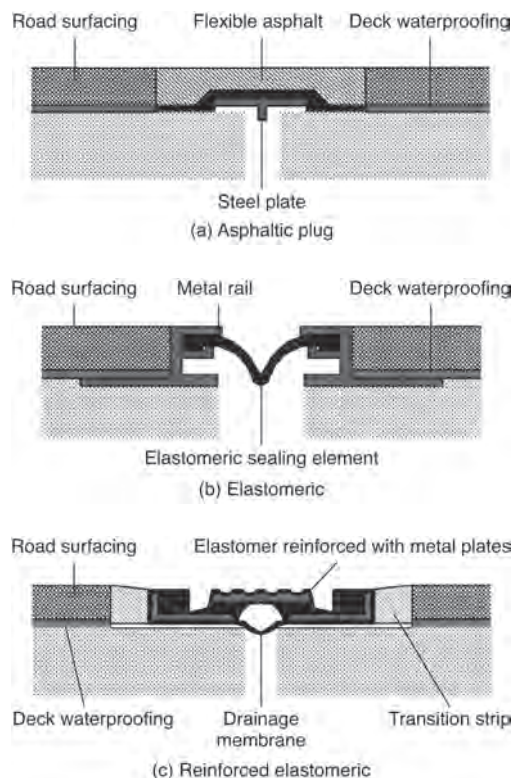


Figure 22.6 General form of expansion joints commonly found in UK road bridges

a number of prestige bridge projects, owing to its potential to resist the ingress of waterborne chloride ions (Deason *et al.*, 1993). However, such blends are inherently prone to poor curing, which can lead to unexpectedly permeable cover concrete and a reduced resistance to rebar corrosion (Parrot, 1991). In selecting a material for a particular environment, there is the risk that the macro-environment will change over the life of the structure (owing to industrialisation) or that micro-environments will develop, owing to the ingress of waterborne contaminants. For example, experience with weathering steel bridges in rural areas suggests that they can provide durable, low-cost solutions (Nickerson, 1995). However, where de-icing salts are able to penetrate and remain in moist areas, local rates of corrosion can match that of unprotected steel (McKenzie, 1990). Thus, specifying a potentially durable material does not guarantee that a structure will demonstrate good durability over time.

Having selected an appropriate material, care must also be taken in the detailed design of the structure to ensure that the key water management objectives are achieved. This will include consideration of the need for joints and their locations, the avoidance of moisture and debris traps and simplification of the design to avoid crevices and other potential problems. Where the design makes part of a structure difficult to construct, there is an increased risk that it will be constructed badly. Experience indicates that this will impact on both the short- and long-term durability performance. As a consequence, any

design that complies with the appropriate design codes but is unnecessarily complicated to build under site conditions may be considered a bad design. Given that all bridge structures will require inspection and maintenance, good design should ensure safe and simple access to critical parts of the structure. Indeed, where deterioration of a component is likely, provision should be made for its replacement. A simple example of this is in the area of bridge bearings, which have a life expectancy considerably less than the bridges they support. Despite this, it is surprising how many bearings have not been designed to be replaced without impeding the flow of traffic. The simple inclusion of a raised plinth, to ensure good drainage, coupled with an integral platform within the bearing support structure, to allow the placement of jacks, can significantly reduce long-term maintenance costs with little impact on the initial cost of construction.

Metal bridges

Bridge structures have been constructed from a wide range of metals and alloys. Many early metal bridges made use of the excellent compressive strength of cast iron elements, which were relatively easy to cast into complex shapes and demonstrated reasonable durability. Where designs required tensile strength, use was made of wrought iron elements manufactured using shaping processes that aligned any defects or inclusions such that they did not restrict the tensile capacity of the material. The introduction of steel in bridge construction dates back well over 100 years with, for example, the Forth Bridge being completed around 1890. Steel exhibits good stiffness and tensile strength and is available in a range of sections. Steel components are relatively easy to join by welding and bolting, reducing the need for the many rivets that were traditionally used to hold together iron structures. A limited number of bridge structures have been manufactured from aluminium alloys. The relatively low density of aluminium enables the design of lightweight bridge decks that can be easily transported. However, the low stiffness and high cost of aluminium, relative to steel, coupled with its poor fire performance and limited corrosion resistance in marine exposures, restricts its wider application.

Steel

Steels are a range of alloys of iron and carbon with small additions of other elements to improve their ease of fabrication and ultimate properties. As a class, steels are characterised by their stiffness, ductility, strength, ease of fabrication, ready availability and low cost. However, they have only limited resistance to corrosion under normal atmospheric exposure. This is because the rust that normally forms on the steel is both porous and non-adherent to the underlying surface so that, in the presence of oxygen and water, corrosion continues unabated. Polluted, industrial environments and marine atmospheres are particularly aggressive and plain carbon steel will deteriorate rapidly if not actively protected by the application of a surface coating system or other means, such as cathodic protection.

Temporary protection of steel can be achieved by the application of a variety of oils and greases that limit the availability of water and present a diffusion barrier to oxygen. More permanent uses of greases include commercial systems such as Denso Paste, a mixture of a petrolatum base, inert fillers and corrosion inhibitors. These are used for the protection of steel cables and other structural elements, such as external prestressing elements. They have the advantage that they can be applied to complex shapes, offer long-term protection against corrosion and, when required, can be easily removed to allow inspection of the protected components and subsequently replaced.

Over the past 30 years, the use of steel in bridge construction has shown considerable growth. In part, this reflects the availability of factory-coated steel members and the associated improvements in coating lifetime being achieved; see the section on coating systems. However, all coating systems imply the need for a continuing maintenance commitment over the life of a structure and this in turn has implications for future financial resourcing.

Weathering steel

By careful control of their composition, fabrication and heat treatment, the mechanical properties of plain carbon steel can be optimised for a range of applications. An extended range of properties can be obtained by the use of low alloy steels that were developed primarily to improve mechanical properties and the ease, and effectiveness, of heat treatment. In general, the corrosion resistance of low alloy steels is not very different from that of plain carbon steels. However, their resistance to atmospheric corrosion can be improved by small additions of such elements as chromium, nickel, copper and phosphorus. A typical example is Corten, in which 2% of copper and small amounts of phosphorus are added to plain carbon steel. This alloy was originally developed by the US Steel Corporation in the late 1920s and was used initially for its increased strength, but its resistance to atmospheric corrosion was also found to be superior to that of mild steel. This is because the presence of the copper and phosphorus modifies the rust that forms on the surface of the steel to one that grows slowly to produce a compact, dark brown protective film. For this reason, such alloys are commonly referred to as weathering steels. The change in the normally porous, non-adherent and non-protective oxide to one that is both adherent and protective is an example of a conversion coating and is distinct from the passive film that forms on the surface of stainless steels.

Weathering steels have been used extensively in the USA, where over 2300 bridges were built with this material between 1965 and 1995 (Nickerson, 1995). It has been found to be particularly suitable for remote rural and agricultural environments that are free from the effects of acid rain and other pollutants. The protective properties of the rust film that forms on weathering steels depends crucially on the exposure conditions and typically takes 2–3 years to develop fully. During this period, the initial rate of corrosion is similar to that of

unprotected plain carbon steel but steadily decreases with time. Consequently, it can be very difficult to predict the long-term performance of such steels on the basis of tests carried out over only a few years. Long-term site exposure tests in the USA on architectural-grade weathering steel produced a loss of only 40 µm after 15.5 years exposure, as compared with 720 µm loss by mild steel (Larrabee and Coburn, 1961).

Work in the UK has confirmed that, under suitable exposure conditions, the rate of deterioration of weathering steels is much lower than corresponding rates for mild steel (Chandler and Kilcullen, 1968). However, great care must be taken if selecting this material for use in industrial and marine environments, where the passive film that should protect the steel can fail to form and consequently the steel corrodes at rates similar to unprotected mild steel. Results from 5 years' exposure in a variety of industrial, marine and rural sites in the UK found the rates of corrosion of mild steel to be significantly higher (9–63 µm/year) than that of structural-grade weathering steel (Kilcullen and McKenzie, 1979). These results suggest that only in the severest environment does the corrosion rate of a weathering steel exceed 25 µm/year after 5 years' exposure and is often considerably less.

Practical experience of bridge structures in the USA and other countries has generally been favourable (American Iron and Steel Institute, 1982) with considerable experience now being accumulated on maintenance coatings that can be applied to salt-contaminated weathering steel (FHA, 1995a). Indeed, the results of a study of 63 steel bridges that had been in service for 18–30 years suggested that uncoated weathering steel bridges designed and detailed in accordance with standard recommendations can perform well (Nickerson, 1995). The positive performance of these bridges indicates that, at a minimum, the original selection of uncoated weathering steel eliminated the need for initial painting and at least one additional maintenance painting.

It is important to appreciate that weathering steel structures must be protected from the ingress, and ponding, of liquid water, as the presence of a film of surface water completely alters the corrosion process and can lead to rapid loss of metal section. Indeed, the inspection of a number of foot and road bridges manufactured from weathering steel has confirmed that, where water leakage and restricted ventilation occur, the steel can develop normal, unrestrained corrosion (Goodman, 1979). The potential of weathering steel to fail to passivate under certain conditions has significant implications for structures designed to incorporate this material. In particular, bolted or riveted joints should be avoided, since they provide ideal conditions for highly accelerated crevice corrosion. Similar problems can be experienced in areas subject to the build-up of debris that acts to shield the surface of the steel. This is a particular problem in marine environments, where the presence of chloride ions in sheltered sections of steel can lead to some localised attack, such as a mild form of pitting. This has been observed both in marine environments and on specimens on the underside of motorway bridges where de-icing salts are used in winter months (McKenzie, 1990).

Stainless steel

In some cases, the film of oxide that forms on the surface of a metal is very dense and closely adherent and so provides a barrier to the diffusion of oxygen and further corrosion. Such passive film formation represents an example of kinetic control of the corrosion process. While generally resistant to chemical forms of attack, the oxide films that characterise this class of material are prone to fretting corrosion, as a result of rubbing contact, and erosion by rapidly moving liquids containing suspended solids. Typical examples of metals protected by passive oxide films include aluminium, steel in alkaline concrete and stainless steel.

Stainless steel is a generic name for a series of alloys containing 11.5–30% chromium and 0–22% nickel, with various other alloying additions. Stainless steels do not resist all environments and some grades are susceptible to localised corrosion, such as intergranular corrosion, stress-corrosion cracking and chloride-induced pitting. This is an important point, since some failures can be attributed to the indiscriminate use of stainless steel on the basis that it is a ‘corrosion-free’ material, when experience suggests it is not (Anon, 1985). Therefore, while stainless steels represent a class of highly corrosion-resistant materials, they should be used with due regard for their limitations.

There are basically four types of stainless steel.

- Martensitic stainless steels can be quench hardened in a similar way to conventional carbon steels. Such steels are magnetic and are used in applications requiring moderate corrosion resistance plus high strength or hardness.
- Ferritic non-hardenable stainless steels cannot be strengthened by heat treatment and so must be cold-worked. Such steels tend to show better resistance to stress corrosion than austenitic steels, especially in chloride-containing waters.
- Austenitic stainless steels, often referred to as simply ‘18–8’ because their basic composition is 18% chromium and 8% nickel, are essentially non-magnetic. Such steels possess the best corrosion resistance of all types of stainless steel.
- Age-hardened stainless steels can be hardened by solution-quenching, followed by suitable heat treatment to produce high tensile strengths. With a few exceptions, the corrosion resistance of this type of steel is less than that of the austenitic varieties, especially in the more severe environments.

Currently, stainless steel is used for only a limited number of bridge components, such as handrails, impact barriers and structural elements, such as prestressing tendons and cables. However, the inherent resistance of some stainless steels to chloride-induced corrosion makes them attractive, despite their relatively high initial cost. One problem in incorporating stainless steel components into new or existing steel structures is the risk of bimetallic, or galvanic, corrosion. This occurs where the passive stainless steel is in contact with a plain or low alloy-steel in the presence of a common electrolyte. This can result in localised dissolution of the carbon steel that becomes anodic

relative to the stainless steel. This problem can be avoided either by insulating the two metals to prevent the metals from coming into contact or building the entire structure from stainless steel.

Reinforced concrete structures

From a theoretical viewpoint, reinforced concrete is an almost ideal combination of materials. The steel carries tensile forces, while the concrete both resists compressive loads and protects the embedded reinforcement from corrosion. Thus, reinforced concrete bridges should be designed, and constructed, to make the best use of this inherent durability and augmented with other methods that, taken together, form a coherent protection system. This includes proper specification of the concrete, adequate cover to the reinforcement, good quality construction and curing and, where appropriate, consideration of the use of coatings or electrochemical techniques.

Concrete quality

The provision of good quality concrete starts with the appropriate choice of constituent materials, including the cement type, aggregate source (and grading) and use of admixtures. These must be combined in an appropriate mix design that ensures a workable concrete with a cement content typically in excess of 300–350 kg/m³ and a free water/cement ratio less than 0.45. Suitable limit values can be found in appropriate design codes and should reflect the likely exposure conditions of the structure being constructed (BS EN 206:2000). Once trial mixes have confirmed the suitability of the mix design, it is important to enforce high standards of batching and mixing. All of this effort will be in vain if the mix is not properly compacted and subsequently cured for an adequate time at the correct relative humidity and temperature.

In designing reinforced, and prestressed, concrete bridge structures, it remains common to specify the concrete in terms of its 28 day cube strength. However, in many instances, this is not the correct design parameter, since it is the permeability and composition of the hardened concrete that tends to control the long-term durability. About 80% of all concrete used in construction in the UK is made using Portland cement. In addition, a number of blended cements are available that can help to improve the long-term durability of the hardened concrete in particular situations and environments (BS EN 197:2002). It may be noted that any gains in durability are often at the expense of short- to medium-term strength gain, owing to the lower reactivity of the components blended with the Portland cement (Dewar, 1988). This has implications for both the speed of construction, by delaying the removal of shuttering, and the type and length of curing required. As a consequence, it is vital that the designer of any concrete bridge structure appreciates the specific properties and performance of available cement blends (Jones, 1994). In general, blends of Portland cement with ground granulated blast-furnace slag (GGBS) show excellent resistance to the ingress of chloride

ions but have extremely poor resistance to carbonation. Such blended cements are commonly used for bridges in marine environments, for example, the Humber Bridge and the Queen Elizabeth II Bridge at Dartford, where their resistance to sulphate attack is an advantage (Deason *et al.*, 1993). However, they are considered unsuitable where the primary cause of deterioration is likely to be carbonation-induced reinforcement corrosion. Properly cured blends of Portland cement with pulverised fuel ash (PFA) resist chloride ion ingress quite well, while retaining reasonable resistance to carbonation. Portland cement concretes, conversely, have the best overall resistance to carbonation attack but show poor resistance to chloride and sulphate bearing environments.

In the design of durable reinforced concrete, it is important to remember that there are a number of options regarding the type of reinforcement. There has been considerable interest in the development of stainless steel reinforcement and tests have shown that this provides excellent resistance to chloride ion-induced corrosion (Treadaway, and Davies, 1988; Treadaway *et al.*, 1980). However, this good long-term performance comes at a high initial cost, since stainless steel reinforcement is typically an order of magnitude more expensive than mild steel ribbed bar of equivalent diameter. The use of galvanised steel reinforcement has been found to produce good resistance to corrosion in marine exposure conditions (Treadaway *et al.*, 1980). An alternative to the use of galvanised steel is epoxy-coated steel reinforcement (Hartley, 1994). However, there are concerns about the long-term durability of such coated bar, especially where chloride ions are present in the concrete, and this type of coated bar is currently not used in the UK. More recently both non-ferrous and non-metallic reinforcement materials have become available. These are still the subject of much research, as many of the composite materials being promoted for use are potentially unstable with respect to their interactions with wet, alkaline environments, such as are found in, and around, concrete bridge structures (Clarke, 1993).

Role of cover concrete

A major cause of the durability problems encountered in concrete structures has been inadequate cover to the reinforcement that reduces resistance to both carbonation and chloride ion-induced corrosion of the reinforcement. Inadequate cover detracts from the effectiveness of specifying low water/cement ratio mixes with pozzolanic, or other, additives and incorporating careful long-term curing. A survey of a number of RC structures in Australia found that corrosion-induced damage commonly occurred at locations when the maximum cover was less than 10mm, the average cover being around 5mm (Griffiths *et al.*, 1987). An analysis of similar data obtained from both bridges and buildings suggests that average depths of cover are much higher in bridge structures (Table 22.1) (Marosszeki and Chew, 1990). This has been attributed to the better control of the construction process usually exercised on bridge projects. In considering the consequences of inadequate cover, it has been argued that it is fundamentally incorrect to

deal with the problem by simply specifying larger covers, or higher-quality concrete, as has occurred in recent revisions of design codes (Beeby, 1993). Instead, it is better to ensure that all those involved in the placing of concrete are aware of the problems of low cover and that they check covers conscientiously during the construction phase. Thus, while design codes may be revised to provide structures that are potentially more durable, such gains can only be achieved if the standard of workmanship is high and good quality control procedures are in place to ensure the reliability of the construction process.

Actual cover/design value	Buildings: %	Bridges: %
1.1	>5	>25
1.0	35–40	50–55
0.9	50–55	85–90
0.8	60–65	90–95
0.7	70–75	>95
0.6	>80	>95

Table 22.1 Typical distribution of actual cover achieved, as a fraction of the design value, for in situ reinforced concrete used in building and bridge structures constructed since 1960

Coating systems

The use of coatings to protect structures from the effects of the environment in which they exist has a long history. Nearly 2000 years ago, the problem of ‘spoiled iron’ was described by Pliny the Elder, who noted that the rusting of iron could be prevented by the application of a coating of vegetable pitch, mixed with gypsum and white lead (Plinius, 1999). This recipe works as well today as it did then but has been superseded by a range of multilayer coating systems that are easier to apply, more effective and less environmentally damaging.

Coatings for steel structures

The aqueous corrosion of steel is driven by the cathodic reduction of oxygen and water leading to the anodic dissolution of iron, producing a loss of metal section. That the hydrated iron oxide that subsequently forms is highly expansive, porous and poorly adherent ensures continued corrosion of the exposed metal surface. Thus most steel structures have an inherent, and absolute, need to be protected from aqueous corrosion. While changes in the composition of the steel can increase its resistance to aqueous corrosion (see the section on material selection and design), the most common form of protection is the application, and maintenance, of a suitable coating. For steel structures, such coatings can be divided into two main types:

- **Barrier layers** act to exclude water and oxygen from the surface of the steel, for example, bituminous membranes, greases, varnishes and paints. Such layers depend on their continuity for effective

protection; where the coating becomes cracked, or broken, any protection is compromised. Even in the absence of cracks, many barrier layers provide only imperfect exclusion of oxygen and water over long exposure periods. Thus, at best, they may be regarded as introducing a large ionic resistance to the corrosion cell. For example, the resistivity of paint films is some 10 000 times that of water and so the corrosion rate is reduced by this amount (Fontana, 1986).

- *Sacrificial layers* act to exclude water and oxygen and, more importantly, provide electrochemical protection to the underlying substrate. The classic example of this type of coating is galvanised steel, in which a thin coating of zinc is applied to a steel substrate. Such a coating has a major advantage over inert barrier layers, in that where the coating is broken, or scratched, the exposed steel is cathodically protected by the preferential corrosion of the more anodic zinc.

In reality, the protection of steel bridges employs multilayer coating systems that incorporate aspects of both barrier and sacrificial methods. Such systems are preferred because the combination can produce a synergistic effect, where the effective life of the coating system, measured in terms of the time to first maintenance, is longer than might be achieved with either a wholly barrier system or sacrificial layer. The use of a number of coatings also helps to ensure complete coverage of the substrate and freedom from breaks, or 'holidays', in the coating.

Selection of coatings

From the end of the Second World War through to the mid 1970s, the majority of steel bridges were protected by multilayer alkyd paint systems applied directly over mill-scale adherent to the steel. These paints contained lead and chromate additives in the primer layers to help prolong the life of the coating. Metallic zinc and aluminium coatings were also used in this period, owing to their superior time to first maintenance. However, such metal coatings found limited application, owing to the relatively slow rate of coating application achievable with the existing technology, higher initial costs and difficul-

ties associated with the in situ maintenance of factory-applied metal layers (Deacon *et al.*, 1998).

Steel bridges can be protected by a range of commercial coating systems incorporating a number of layers of polymer-based paints, metal layers and powders and inhibitive admixtures. Typically, as much of the coating system as possible is applied under factory-controlled conditions, with one or two topcoats subsequently applied on-site to seal any defects created during handling and provide a uniform, and aesthetically pleasing, appearance. The decision to employ factory-controlled conditions for the application of a coating carries a number of potential advantages. First, it enables good control of the initial surface preparation, which is a crucial factor in ensuring adequate bonding between the steel and the primer layers. Second, it ensures that the selected coating can be applied under controlled conditions of temperature and humidity, and to high tolerances of workmanship and thickness. Despite this, it must be remembered that a major requirement of any factory-applied coating system is that it must not be susceptible to damage during transit and handling. Where damage does occur, the coating system must be both tolerant of such defects and repairable, under site conditions, to some close approximation of its original condition.

Within the UK, the Highways Agency (2001) requirements for the protection of steelwork against corrosion are set out in Series 900 of the *Specification for Highway Works*. The specification considers the environmental exposure (e.g. inland or marine), the level of accessibility available on-site, the required durability of the coating (in terms of time before any maintenance is required) and such issues as colour and surface finish. The resulting protection systems consist of three or four distinct layers although in a more protected situation, such as within a box girder, only two layers may be required (Table 22.2).

Environmental concerns and health and safety requirements are becoming increasingly important in the selection process. As a consequence, the use of lead-based additives in coatings is now banned in most countries. Similarly, limits on the allowable levels of volatile organic compounds are steadily reducing,

Environment	Access	Typical arrangement of layers	Thickness of layer: μm
Inland	Ready	Polyurethane finish Micaceous iron oxide, high-build, quick-dry epoxy undercoat Zinc phosphate high-build, quick-dry epoxy primer Steel substrate	} 300
Marine	Difficult	Polyurethane finish Micaceous iron oxide, high-build, quick-dry epoxy undercoat Zinc phosphate high-build, quick-dry epoxy primer Aluminium epoxy sealer Sprayed aluminium Steel substrate	
Interior	Ready or difficult	Micaceous iron oxide, high-build, quick-dry epoxy undercoat Zinc phosphate high-build, quick-dry epoxy primer Steel substrate	} 200

Table 22.2 Typical multilayer coating systems used to protect steelwork from corrosion

with many of the traditional solvent-based paint products being replaced by water-based equivalents. As a consequence, coating selection criteria are not static but are evolving to meet the more stringent regulatory controls while at the same time attempting to ensure adequate, and cost-effective, long-term performance.

While any selection process recognises the importance of starting with a suitable coating system for the particular environment, it must be remembered that in most cases more money will be spent on maintenance of the coating than on its original application (Chandler, 1979). To this end, it can be useful to employ layers that break down in such a way that it is obvious before the breakdown becomes complete, for example, coloured primer coats on sprayed metal coatings. The easy detection of deterioration is only a first step, since the outer layer must also be capable of being repaired to a satisfactory standard under the prevailing site conditions. Of course, any maintenance to the coating system can be rendered ineffective if it is carried out without regard for other defects, such as uncontrolled water leakage, which must also be corrected. However, it must also be remembered that the maintenance of surface coatings is subject to the availability of resources; there can be no guarantee that maintenance will be carried out when the need first becomes apparent. If maintenance is delayed for too long, and corrosion takes a firm hold, any subsequent repair may prove ineffective in stopping corrosion.

Clearly, any maintenance operations that are carried out must not carry any undue risks to either the maintenance operators or the local environment. This is particularly important when dealing with existing coatings that may contain heavy metals, such as lead and chrome. These can be hazardous to human health in even small quantities if ingested or inhaled. As a consequence, it is vital to take measures to protect all workers exposed to such hazards (Hoffner, 1995). The risk to personnel can be reduced by the use of overcoating techniques (Hopwood, 1995). These painting operations require only the partial removal of the existing coating and subsequent application of two, or three, coats of new material over a mixture of clean steel, corroded surfaces and existing coating. However, such coatings are prone to early disbondment, owing to incompatibility with the original layers, and are not a viable alternative to full removal and replacement of a coating system that has undergone significant deterioration.

Paints

Paints are complex coatings and consist of an organic polymer film, or binder, containing a filler or pigment dispersed throughout its thickness. Most polymer binders are initially dissolved in an organic solvent, which prevents polymerisation, aids spreading of the paint and then evaporates as the paint 'dries'. Owing to concerns about the environmental and health risks associated with the uncontrolled release of such solvents, manufacturers have introduced a range of water-based paint systems. An alternative to these new systems is the use of more established solvent-free paints that are catalysed just prior to

application and polymerise on the surface of the steel at a rate depending on temperature and catalyst concentration.

The pigments used in commercial paint systems for bridge structures are essentially of three kinds: inert, inhibitive and sacrificial. Inert fillers simply thicken the paint and so only serve to lengthen the diffusion path of water to the metal surface. Inhibitive pigments act as a controlled source of corrosion inhibitor (see the section on active protection of metals from corrosion) and are used in primer coats in contact with the metal. Sacrificial fillers, such as metallic zinc, corrode in preference to the steel substrate.

Surface preparation is a vital factor in coating durability and involves the production of a clean, rough surface that will provide the best physical and chemical 'key'. Prior to coating, the surface should be firm and free from any layers that will prevent the paint film wetting the surface and adhering. In addition to being physically clean, the surface must also be chemically clean and free from salts and weld deposits. While visual inspection can ensure the absence of major contamination, the presence of thin grease layers and salt contamination are not easily detected by the unaided eye. Fortunately, a range of methods is available for detecting the presence of undesirable contamination (Bayliss, 1979).

When applying several coatings, it is necessary to ensure that successive layers bond to one another. The thinning of high-build paints, to make them easier to apply, is a common mistake, as is the inadequate stirring of heavily pigmented materials and the incorrect mixing of two-component systems. An important outcome of correct application is the achievement of the specified coating thickness, which is an important factor in determining the life of a coating. For any given paint and environment, the time to first maintenance generally increases with coating thickness and decreases with increasing severity of exposure. From a practical perspective, measurement of the wet film thickness can enable the operator to predict the final dry film thickness at a stage when it can still be corrected. This requires an established relationship between wet and dry film thickness.

As noted, a wide range of coating types have been used to protect steel bridges over the past 50 years. Early alkyd paint systems have achieved times to first maintenance ranging from 2 to 15 years. This variable performance reflects the wide range of exposure conditions to which they have been subjected and the inherent limitation of such paints under damp conditions. In terms of overall performance, alkyd paint systems have given similar performance to systems incorporating micaceous iron oxide. Traditionally, these consisted of a lead-based primer with two coats of micaceous iron oxide applied over a grit-blasted surface. More recently, lead has been replaced by zinc phosphate. Chlorinated rubber paints have a good reputation for use in severe environments where they perform better, thickness for thickness, than alkyd-based coatings with times to first maintenance of 5 to 10 years. Despite this, as a class, chlorinated rubber paints do not perform substantially better than alkyd paints under moderate exposure conditions and rarely exceed 10–15 years before requiring maintenance.

Two-part epoxy resin paint systems have shown variable performance over the past 25 years, with early formulations being sensitive to application conditions and overcoating time. More recently, epoxy mastic bridge coatings have gained popularity in the USA (FHA, 1995b). These two-component coatings are based on one of several epoxy resins, cured using either a polyamide or amine curing agent, and are often heavily pigmented with aluminium flakes or powder. They are claimed to offer high-build, low-solvent content, short recoat times and low cost. They are also claimed to be tolerant of surface defects and so require less surface preservation prior to application. However, the high viscosity of some of these products can lead to inconsistent paint thickness if poorly applied. A major problem with this class of coatings is their poor resistance to under-film corrosion at defects or edges. Under marine exposures, epoxy mastic coatings have shown significant failure after only 18 months in the presence of defects (FHA, 1991a). Even after careful application to near-white, blast-cleaned steel, such coatings showed significant failures after only 3 to 5 years (FHA, 1991b).

Metal coatings

Coatings of zinc and aluminium have been used for the protection of a large number of bridge structures (Porter, 1979). These sacrificial coatings do not have to be continuous since, at breaks in the layer, the zinc or aluminium will corrode in preference to the steel. There is little doubt that properly applied metal coatings provide longer lives to first maintenance than conventional paint systems, especially in aggressive environments. In practice, the sacrificial layers are themselves usually sealed with a paint layer that helps to extend the effective life of the coating (van Eijnsbergen, 1976). This, in turn, increases the time to first maintenance and so enables costs to be reduced over the life of the structure. Given the increasing costs of labour for painting operations, and the high indirect costs associated with interruption of service, the long life of sacrificial metal coatings has made them increasingly attractive to bridge engineers. One important feature of sacrificial metal coatings is their inherent resistance to damage introduced during transport, handling and in service. This is in marked contrast to many paint systems, where such damage can initiate the formation of corrosion cells.

It is generally accepted that, for adequate corrosion protection, the thickness of zinc or aluminium applied should be not less than 100 μm , and thicker coatings are often desirable and more economical overall. The choice of coating thickness will depend on the corrosion resistance required and the environment. Thus, in rural environments, zinc corrodes at 1–3 $\mu\text{m}/\text{year}$, rising to 3–6 $\mu\text{m}/\text{year}$ in urban environments, depending on the acidity level, and around 5 $\mu\text{m}/\text{year}$ in marine exposures. These values are one-tenth of those of bare mild steel (Porter, 1979) and one-fifth of that of weathering steels (Kilcullen and McKenzie, 1979) under the same conditions. Based on a

notional 100 μm coating thickness, this equates to a minimum coating life of 20 years even under severe exposure conditions.

Zinc coatings can be applied by a range of techniques but for steel bridges are usually applied by either hot-dip galvanising or metal spraying. The choice of coating method depends on the size and shape of the article to be protected, the severity of the exposure and the requirements for joining the treated components.

The galvanising of bridge components is usually carried out after fabrication and requires careful cleaning of the steel surface by acid pickling, followed by fluxing and dipping in a bath of molten zinc at around 450°C. The zinc coating that forms is bonded to the underlying steel by a layer of iron–zinc alloy, which forms during the coating operation and is as protective as the pure zinc outer layer. The thickness of the alloy layer increases with dipping time and significantly increases the time to first maintenance of the overall coating system. Galvanised coatings have the less obvious advantage of having suitable characteristics for high-strength friction grip joints (Staff, 2003). Despite the many advantages, there are physical limits on the size of component that can be successfully, and economically, galvanised. In addition, great care must be taken to avoid warping or distortion of the component as a result of thermal strains induced by the relatively long contact times with the molten zinc.

Full-size bridge components, and complete bridges, can be coated with both aluminium and zinc coatings, utilising metal spraying techniques. Such ‘metallised’ coatings may be applied either under factory conditions or in special site-spraying shops in the field. The basic technique uses a controlled heat source, such as a gas flame or electric arc, to melt the coating metal in an air-stream, which propels the droplets of semi-molten metal onto the surface being coated. Using modern equipment, it is possible to achieve application rates similar to conventional air-spray paint methods. In contrast with galvanising, there is practically no significant heat input to the material being sprayed. This is advantageous in that it reduces the risk of thermally induced distortions of the component but means that the metal coating does not react with the underlying steel surface, as occurs during galvanising. As a consequence, the bond between the metal coating and the steel is purely mechanical in nature and depends crucially on good surface preparation. The steel must be prepared by abrasive blasting to achieve a near-white surface with a roughened texture that provides a good mechanical key to the sprayed metal. Because of the nature of the spraying process, metal coatings are inherently porous and the final coating must be sealed with a suitable combination of sealer and topcoat paint. The topcoat is necessary partly because of the desire for a uniform appearance but principally because the need for maintenance can be seen when the underlying sealer coat becomes visible. If the topcoat is well-maintained, the coating system will last almost indefinitely but even if it is not recoated the metallic layer still provides many years of protection.

Coatings for concrete structures

In the late 1950s, designers took to the use of reinforced and prestressed concrete with enthusiasm, since it was believed that these were essentially maintenance-free materials, owing to the passivity induced by the high pH of the cementitious materials surrounding the steel. Experience has shown that this belief was incorrect and many of the concrete structures built in the past 40 years are now showing signs of distress. The causes of this failure of durability are well documented and it is becoming widely accepted that concrete structures are as much in need of coatings as their steel equivalents. Indeed, all new concrete structures built in the UK to the *Specification for Highway Works* must be given two coatings of a silane waterproofing treatment. This builds on experience in Germany, where concrete road bridges are coated on all exposed surfaces with a variety of surface treatments designed to reduce the ingress of water, chloride ions and carbon dioxide (Middelboe, 1985). Any such coating can be expected to have a life of 5 to 10 years, depending on its type and application, which clearly implies a continuing maintenance commitment over the life of a typical road bridge.

External protection coatings

External coatings are applied to the surface of concrete structures for a variety of reasons. In considering only those that are concerned with improving durability, it is possible to distinguish three main types of treatment (Figure 22.7), based on the method by which they protect the underlying substrate (Leeming and O'Brien, 1987).

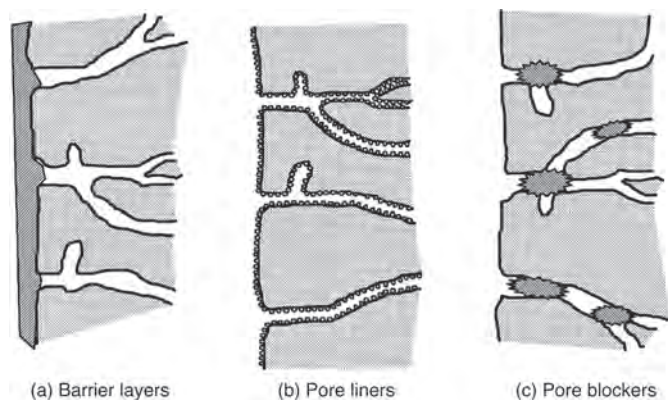


Figure 22.7 The three main types of treatment used to protect concrete structures

Barrier layers

Barrier layers aim to provide an impervious film over the surface of the concrete and can themselves be divided into three main types: coatings, surface sealers and renders. Coatings are similar to the paint layers used to protect steel surfaces in that they are applied as viscous liquids by spray, brush or roller application and subsequently cured, by solvent loss or chemical

reaction, to produce layers with a dry film thickness of between 150 and 500 μm . Typical materials used for coatings include bitumen and synthetic polymers, such as epoxy resins, polyurethanes and alkyds. Sealers are similar to coatings in their physical form and method of application but are distinguished by their claim to penetrate the surface of the concrete. As a consequence, sealers are characterised by their good adhesion to the substrate and are often used as primer layers in multi-coat systems. Sealers are typically based on such materials as low-viscosity epoxy resins, acrylics and linseed oil. Renders are distinguished by their thickness, which can be many millimetres, and by the fact that they are applied by trowel. They are usually extended by the addition of inert fillers that both reduce the cost per unit volume and increase the resistance to weathering. Owing to both the viscosity of such materials and their method of application, their adhesion to the substrate is usually inferior to coatings and sealers. Most modern renders are based on polymer-modified cement mortars.

The most common form of barrier layer used to protect concrete bridges is coatings. For such surface coatings to be effective, good workmanship is essential and the concrete surface must be carefully prepared, by filling and levelling, before the coating is applied. The use of multilayer coating systems is generally preferred over one-coat, high-build, coatings that are prone to defects and pinholes when poorly applied. Regardless of the type or quality of coating specified, it is important to remember that long-term exposure to weathering produces a stiffening of the material and reduction in elongation at break (Le Page, 1996). This implies a need to maintain and, where necessary, replace such coatings over the life of a structure.

Coatings are commonly used for their ability not only to prevent the ingress of liquid water but also to restrict the passage of carbon dioxide. Such anti-carbonation coatings are carefully formulated to allow some permeability to water vapour to avoid a build-up of vapour pressure behind the coating and allow long-term drying of the structure. This is assisted by the fact that the molecular size of carbon dioxide is larger than that of water. Such coatings can be characterised in terms of their diffusion resistance, u , which is a dimensionless parameter indicating how many more times impermeable a coating is than air under equal conditions.

Preventing carbonation and water or chloride ingress is clearly beneficial in reducing the risk of reinforcement corrosion (Robery, 1988) and, by keeping the concrete dry, is also beneficial in restricting damage from both freeze-thaw cycling and alkali-silica reaction. However, such protection relies on the coating remaining unbroken. As a result, the ability of a coating to bridge over a crack as it forms in the underlying substrate is one of several parameters influencing coating selection (Harwood, 1990). Having bridged a newly formed crack, the coating must also be capable of withstanding the repeated cyclic opening and closing of an active crack. The ability of a surface coating to both bridge a crack that forms in the substrate and accommodate subsequent movement of the crack is a complex function of coating thickness, material type, bond strength and temperature. Figure 22.8 shows the influence of

temperature on the behaviour of a 300 μm thick coating subject to an initial crack opening of 0.3 mm, which subsequently varies in width between 0.3 and 0.6 mm. It is clear that there is a critical temperature above which the coating accommodates many thousands of crack-opening cycles prior to failure but below which it fails to either accommodate or bridge the crack. Generally, coatings made up of several individual layers are more tolerant to cracks in the underlying concrete, since a defect in one layer will not necessarily initiate failure of the adjacent layers (Le Page, 1996).

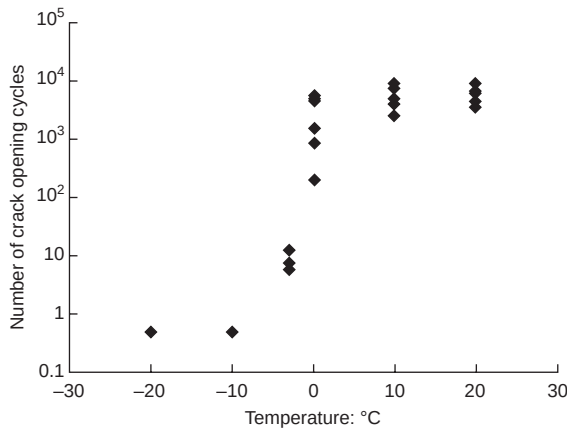


Figure 22.8 Influence of temperature on the crack accommodation behaviour of a 300 μm thick coating subject to an initial crack opening of 0.3 mm, which subsequently varies in width between 0.3 and 0.6 mm (Le Page, 1996)

Pore liners

Pore liners are low-viscosity fluids that are able to penetrate the concrete, where they are adsorbed onto the surfaces of the capillary pores and react to form a water-repellent layer. Typical penetration depths on normal concrete under site conditions are in the range 2–10 mm, although penetration depths exceeding 50 mm are attainable under laboratory conditions. The materials used as pore liners are generally based on monomeric alkylalkoxysilane and oligomeric alkylalkoxysiloxane, commonly known as silane and siloxane, respectively, although certain silicone compounds can be used to produce the same effect. Silanes become reactive in the presence of moisture, the speed of reaction being governed by the surrounding pH. In alkaline concrete, silane reacts with the pore lining quite rapidly (McGill and Humpage, 1990). However, the silane molecule is very volatile and can evaporate before it has time to react with the surface. This is less of a problem with the oligomeric siloxanes, which are less volatile and so have a longer time to react with the surface. Work on stone faces has demonstrated that the efficiency of a hydrophobic surface treatment is greatly increased by curing at elevated temperature and humidity (Bohris *et al.*, 1996).

All pore liners work by altering the contact angle, θ , between the concrete surface and the liquid water (Figure 22.9). Under

normal conditions, water readily wets the concrete surface, $\theta \approx 0^\circ$, and capillary forces promote the ingress of water into the pore system within the cement paste. By treating the surface to produce a hydrophobic lining, the concrete effectively becomes unwettable, $\theta \approx 90^\circ$ – 180° . As a consequence, any water falling on vertical and inclined surfaces simply runs off. This significantly delays the ingress of water into a structure under all but standing water conditions. Unlike pore blockers, these treatments allow the passage of water vapour, promoting long-term drying of the concrete, and so reduce the likelihood that any embedded steel will corrode (Darby *et al.*, 1996). Another advantage of these materials is that they are clear and colourless and so do not prevent subsequent visual inspection of the treated structure for cracks and other defects. Against this must be set the disadvantage that, once applied and cured, these treatments are effectively invisible and can only be detected by their impact on the wetting behaviour of the surface. For this reason, an initial surface absorption test (ISAT) of the surface before and after treatment is often used to ensure that the application process has been adequate. Since the ISAT is effectively non-destructive, it can be repeated at intervals to determine the effectiveness of the treatment over time and identify any need for re-application.

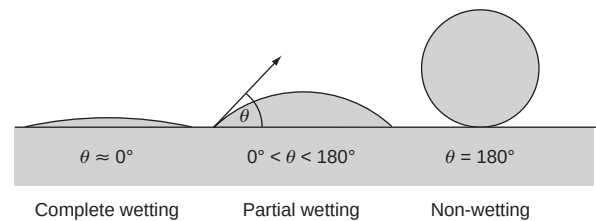


Figure 22.9 Wetting characteristics of a surface as a function of the contact angle, θ , between the liquid and the solid

Pore blockers

Pore blockers are a family of products, based originally on calcium and sodium silicates, which are claimed to penetrate 1–3 mm into the surface of the concrete and form reaction products that block pores within the concrete and densify the matrix. Owing to the partly crystalline nature of the products that form, these have become known as ‘crystal growth’ materials. There is little evidence that such treatments are effective in reducing either carbonation or chloride ion ingress in the bulk of the concrete, although more recently introduced products based on lithium polysilicates appear to offer improved performance. Other pore blocker treatments make use of low-viscosity resinous materials, such as epoxy resins and acrylics, which penetrate the pores and harden in situ.

Internal protection coatings

Recognition of the fact that concrete cover sometimes fails to provide the expected protection to embedded reinforcement

has given rise to the development of coated rebars (Walker, 1989). The objective is to ensure that, should chloride ions reach the depth of the steel, there is another layer of defence preventing the onset of corrosion and the subsequent development of cracking and spalling of the concrete cover. However, a designer considering the use of coated reinforcement must weigh the perceived benefits against the extra cost of coated steel products, which are typically twice that of equivalent uncoated steel bar.

In the USA, many concrete road bridges have been manufactured using fusion-bonded epoxy-coated reinforcement with a coating thickness of 1–2 mm (Hartley, 1994). Such an impervious coating relies on being a pinhole-free barrier if it is to provide adequate protection. There has been some concern that such coatings can become damaged during the construction process, exposing small areas of steel. This is potentially disastrous, since such defects could become the sites of active anodes, with the remainder of the coated bar becoming cathodic. This unfavourable anode–cathode area can lead to significant localised pitting. An assessment of structures containing coated reinforcement in the Florida Keys area – characterised by a marine environment – has revealed considerable damage to the coated bars after periods of 10–15 years.

In Japan, there has been interest in the use of galvanised steel reinforcement, with a 60–100 μm thick layer of zinc. Such coatings are as equally likely as epoxy coatings to be subject to damage during the construction process but, because the zinc is anodic relative to the steel, any steel exposed by cracks in the coating will be protected by preferential dissolution of the zinc. Again, long-term evidence regarding the effectiveness of such systems is limited, although there is evidence that galvanised reinforcement can increase the time to the initiation of corrosion of rebars in chloride-infested concrete (Treadaway *et al.*, 1980). However, as the concentration of chloride ion increases, the rate of loss of the zinc layer increases appreciably, reducing its effective lifetime (Treadaway and Davis, 1988). In addition, there remains the possibility of an interaction between the zinc and the steel to form brittle compounds, which reduces the ability of the reinforcement to deform in a ductile manner (Andrade *et al.*, 1995).

Active protection of metals from corrosion

Owing to the electrochemical nature of the metallic corrosion process, it is possible to control the reaction by the application of a suitable potential, for example, cathodic protection. This is an example of the thermodynamic control of the deterioration process, one previously mentioned method being the use of galvanised steel. It is also possible to interfere with one or more of the rate-determining reactions that occur at the anode or cathode of the corrosion cell. This can be achieved by the application of corrosion inhibitors that exert kinetic control over the rate of corrosion.

Electrochemical methods – cathodic protection

The use of cathodic protection for the control of rebar corrosion in both new and existing reinforced concrete structures is becoming increasingly common, since it offers a secure means of stopping corrosion of the embedded steel (Hayward, 1997). Cathodic protection is based on the principle of altering the normal rest potential of a metal (point A in Figure 22.10) to bring it into the immune region of the Pourbaix diagram (point B in Figure 22.10), where the metallic state is the stable one. In this region, the metal is effectively held in a state where anodic dissolution is impossible, since only cathodic reactions can occur on the surface of the protected metal. There is an interesting side effect of doing this, since the cathodic reaction results in the production of hydroxyl (OH^-) ions at the metal surface. This has the effect of raising the local pH (point C in Figure 22.10) and as a consequence the surface of the metal becomes protected by the build-up of basic salts. In the event of failure of the cathodic protection system, this provides additional protection by ensuring that the steel surface remains in the passive condition (point D in Figure 22.10).

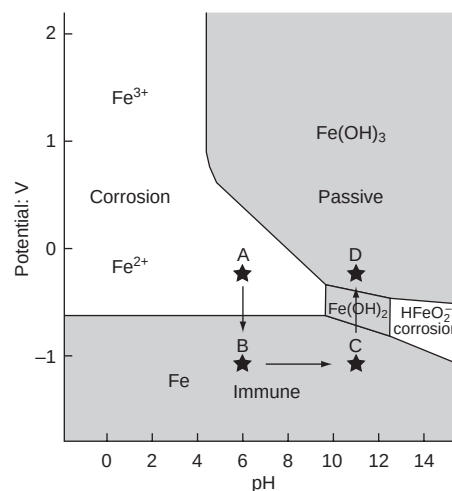


Figure 22.10 Pourbaix diagram for iron in oxygenated water showing the process of cathodic protection

There are two ways to cathodically protect steel reinforcement within a concrete structure.

- **Galvanic coupling (sacrificial protection).** By electrically connecting the reinforcement to a more anodic metal in the electromotive force series, the steel will be protected by preferential corrosion of the sacrificial anode. Galvanised steel reinforcement works in this way and typically has 50–150 μm of zinc coating on the surface of the steel (Treadaway *et al.*, 1980). The sacrificial corrosion of the layer of zinc lowers the potential of the steel so that any exposed parts of steel do not corrode. The system is not as wasteful as it first appears, as the zinc soon acquires a coating of basic salts. In addition, the cover concrete acts as a buffer zone, reducing the supply of oxygen, water and chloride ions to a level where the thin zinc coating can last for many years (Treadaway and Davis, 1988;

Treadaway *et al.*, 1989). The problem with this approach is that normal corrosion of the steel can begin once the galvanised layer has been consumed, albeit with a much delayed onset. Some concern has been expressed that the hot-dipping process used to cover the steel bar with the zinc can lead to the formation of brittle intermetallic compounds within the interfacial layer between the steel and zinc, potentially leading to embrittlement (Andrade *et al.*, 1995).

- **Impressed current.** The cathodic protection of steel reinforcement by means of an impressed current supplied from an external power supply involves connecting the negative terminal of the power supply to the reinforcement cage tank, and the positive to an inert, titanium anode mounted on the surface of the concrete (Figure 22.11). The electric leads connecting the steel reinforcement and titanium anode are carefully insulated to prevent current leakage. The current passes to the steel, which becomes the site of cathodic reduction reactions, with the simultaneous formation of hydroxyl ions and a rapid rise in the local pH.

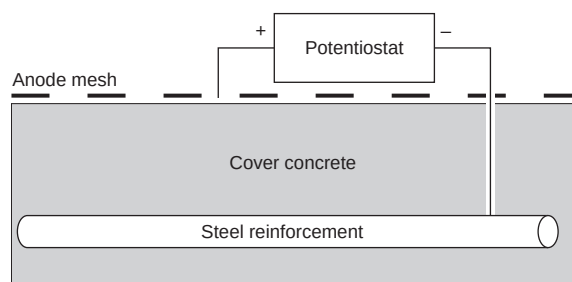


Figure 22.11 Cathodic protection of steel reinforcement in concrete

Corrosion inhibitors

One method of increasing the tolerance of reinforced concrete to carbonation and the presence of chloride ions is the application of corrosion inhibitors. These are substances that, when present on the surface of the steel, decrease the rate of corrosion without significantly changing the concentration of other corrosion agents (ISO, 1999). This differentiates them from coatings and other materials that act to restrict the supply of water, oxygen and chloride ions around the steel bars. There are a number of commercially available corrosion inhibitors, which can be categorised into three main groups:

- *inorganic inhibitors*, for example, nitrites
- *organic inhibitors*, for example, amines
- *vapour phase inhibitors*, for example, amino alcohol.

All these materials act to interfere with one or more stages of the corrosion process and can, under ideal conditions, reduce the corrosion rate by at least one order of magnitude. Of course, a reduction of only 50% in the corrosion rate might provide a cost-effective increase in the life and reliability of a particular structure.

In attempting to slow the corrosion rate, inhibitors can either suppress anodic dissolution of the steel (anodic inhibitors) or

suppress the various electron-consuming reactions that can occur (cathodic inhibitors). One problem inherent in the use of anodic inhibitors is that it is essential that they are present in sufficient concentration to close down all the anode sites. Failure to achieve this can result in a few anodes remaining active, the subsequent anode/cathode ratio being such as to promote aggressive, and highly localised, pitting of the steel. In contrast, cathodic inhibitors are essentially safe at all concentrations, since they always act to increase the anode/cathode ratio, reducing the total available current for anodic dissolution. Some 'mixed' mode (ambidodic) inhibitors are capable of suppressing the reactions at both anode and cathode sites simultaneously.

When used to protect steel in concrete, corrosion inhibitors can be either added as admixtures to the fresh concrete or applied to the surface of the hardened concrete (Treadaway and Russell, 1968).

Admixed inhibitors

Admixing allows a uniform distribution of inhibitor throughout the concrete; in high performance concretes, the low permeability of the cover concrete prevents the inhibitor from being lost. The main problem to overcome is that of preventing any adverse effect on the fresh and hardened properties of the concrete. Several types of inhibitor are specifically used as admixtures for new concrete, the most common being based on calcium nitrite. It is now well established that nitrites are anodic inhibitors that interfere with reactions at the steel–concrete interface and are consumed as they compete with chloride ions to block the anodic sites. However, with a sufficient dosage, they should be able to maintain a constant availability at the steel surface and have been found to be effective as long as the chloride/nitrite ratio stays below about 1.8 (Virmani and Clemena, 1998). A dosage rate of 10–30 l/m³ is generally specified, depending on the expected maximum chloride level at the rebar. This is usually the benchmark against which other inhibitors are tested.

Figure 22.12(a) illustrates the corrosion behaviour of steel bars embedded at 10, 25 and 40 mm in a Portland cement concrete (w/c=0.60) and subject to weekly wet/dry ponding with a 5% solution of sodium chloride solution. The bars at 10 mm start to corrode after only four to six cycles; the bars at 25 and 40 mm start to corrode after 12–16 cycles and 54–58 cycles, respectively. This clearly demonstrates the role of cover depth as a means of protecting steel from the aggressive action of ingressing chloride ions. This behaviour can be compared with that of similar bars in an equivalent concrete containing 4% (by weight of cement) admixed calcium nitrite (Figure 22.12(b)). It can be seen that the presence of the inhibitor increases the time to initiation of corrosion for the bars at all the depths tested. It is of note that linear polarisation measurements showed that, once initiated, the subsequent rates of corrosion of the bars in the concrete containing the inhibitor were less than those in plain Portland cement concrete (Mulheron and Nwaubani, 1999).

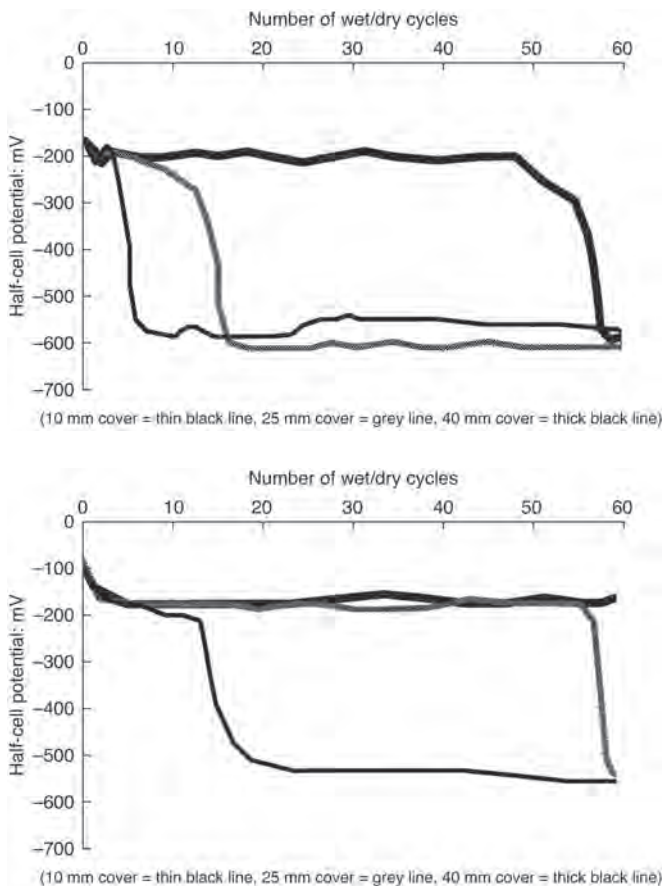


Figure 22.12 Change in half-cell potential with number of wet/dry cycles for steel bars embedded in: (a) Portland cement concrete (free w/c = 0.60); (b) Portland cement concrete (free w/c = 0.60) containing 4% admixed calcium nitrite

Surface-applied inhibitors

The main advantage of surface-applied corrosion inhibitors is that they can be added after construction is complete. Thus, if a concrete structure is showing signs of reinforcement corrosion, the inhibitor can be applied as a surface treatment, which subsequently penetrates the concrete, repassivating the reinforcement and stopping further corrosion. This requires the inhibitor molecule to be sufficiently mobile to move through the pore structure of the cover concrete; for this reason, these are sometimes referred to as migratory corrosion inhibitors. It is commonly assumed that such materials rely on capillary action. However, vapour phase inhibitors, which are characterised by their high vapour pressure, can also migrate as vapours, provided the pores are sufficiently empty to allow vapour transport mechanisms to operate.

Amino alcohols are ambidodic, film-forming inhibitors, which block both anodic and cathodic reactions. They can be applied as a coating on the surface of the concrete or a 'plug' of material in a hole, or mixed into repairs. Under laboratory conditions, such inhibitors are able to migrate through the concrete cover of a relatively porous, poor-quality concrete (Mulheron

and Nwaubani, 1999). One issue with corrosion inhibitors lies in determining how effective they are. The results of laboratory tests suggest that both admixed and surface-applied inhibitors can significantly reduce corrosion rates in the presence of chloride ions (Berke, 1999; Treadaway and Russell, 1968; Virmani and Clemena, 1998). The results of field trials have been more variable with often little, or no, reduction in the long-term rate of corrosion reported (Broomfield, 1997; Prowell, 1993; Sohaghpurwalla *et al.*, 1996; Sprinkel and Ozyildirim, 1998). In part, this reflects the practical problems of ensuring that the inhibitor is able to reach the surface of the steel. However, in some trials (Broomfield, 1997), the level of chloride ion present was greater than 1% by weight of cement, which is beyond the limit at which many inhibitors can reasonably be expected to suppress corrosion (Mulheron and Nwaubani, 1999). In addition, some of the structures that had been treated already showed signs of reinforcement corrosion, with associated cracking and spalling of the cover. Such structures are beyond the point at which a non-invasive repair using a migratory corrosion inhibitor would be considered practical by their manufacturers. Indeed, the balance of evidence suggests that surface-applied corrosion inhibitors are most effective when applied to a reasonably permeable cover concrete that is either carbonated or has a chloride ion content below 1% by weight of cement. Where reinforcement corrosion is well established and cracking of the concrete cover has taken place, the application of surface-applied corrosion inhibitors is of little value and invasive repair methods must be employed.

Protection from physical processes

Where deterioration is primarily driven by physical processes that depend on the interaction between a structure and some vector within the environment, there are two key approaches that may be adopted.

- *Reduce the risk of occurrence.* The risk of physical impact on structures can sometimes be reduced by improving the level of visibility of the vulnerable components. For this reason, many bridge structures are protected by high-visibility reflective signage that aims to provide drivers with a clear indication of the available height and width. Reducing the probability that the material will be saturated when frost occurs can decrease the risk of frost damage to brickwork. Similarly, the adoption of appropriate designs and suitable construction methods can mitigate the risk of erosion of bridge supports located in, or adjacent to, rivers.
- *Reduce the effect of an occurrence.* When there is a significant risk that physical damage can occur by extreme processes, such as large-scale impact, excessive ground movement or flooding, it is possible to adopt designs that ameliorate the worst effects. For example, protective barriers or other cladding layers can be used sacrificially to absorb impact stresses or redesigned so that the bridge is isolated from damaging ground movements. Alternatively, structural elements at high risk can be strengthened to resist peak stresses and designed to ensure continuity of the load path,

even under extreme displacements. Where the risk of an extreme event is deemed high, the design should also allow for ease of replacement of failed elements.

Scour and erosion

Many bridges are required to span rivers, waterways and flood valleys. As a consequence, such bridges and their supporting piers, columns and abutments must resist the action of flowing water. This resistance must encompass both the normal variations in seasonal flows and also those that occur under flood conditions. The maximum flow depends on geographical location, the topography of the local catchment area and the prevailing weather system. However, in designing a structure to resist the forces exerted by the most severe 100-year flood, account must also be taken of the cumulative effect of the much smaller forces exerted by the normal flow over the life of the structure.

Where a bridge and its approaches exert no significant constriction of natural flows, the risk of long-term damage is relatively small. In contrast, where a bridge restricts the flow of water or where a bridge is situated at a narrow point in the watercourse, there is a risk of severe hydrodynamic forces on the structure and scouring of the supports. This risk is greatly magnified under flood conditions that can, in the extreme, lead to failure of the structure and loss of life. For example, the collapse of the I-90 highway bridge crossing over Schoharie Creek on 5 April 1987 resulted in the loss of ten lives when two spans of the bridge fell into floodwaters after a pier was undermined by scour. A further seven lives were lost on 10 March 1995, when the two I-5 bridges over Los Gatos Creek near Coalinga, California, failed because of scour from a large flood (Richardson *et al.*, 1997).

Scour may be defined as the removal of riverbed material such as sand and rocks by the action of river or tidal currents. It occurs on a continuous basis over time but is especially strong during flood conditions, since the swiftly flowing water has more energy to lift and carry material downstream. It is a particular problem around bridge structures as it has a tendency to expose or undermine foundations that would otherwise remain buried (Figure 22.13). The term *scour* is general and may be divided into a number of distinct types (Noble and Boles, 1989).

- Degradational scour is the removal of sediment from the riverbed by the flow of water. This removal of material and resultant lowering of the river bottom is a gradual process, but may remove large amounts of sediment over time.
- Contraction scour is the removal of material from the riverbed caused by the acceleration of water approaching and flowing under a bridge. This process occurs when the width of the bridge opening is narrower than the natural river channel. The faster flow enables the water to carry a greater volume of material from under the bridge than would occur if the flow were uncontrolled.
- Local scour is the removal of sediment associated with vortex systems induced by obstruction to the main flow, such as from around bridge piers, abutments and embankments. This results in the highly localised removal of material adjacent to the structure to form so-called scour holes. The vortex that contributes to the formation of these holes originates at the upstream nose of the obstruction, where the flow acquires a downward component that reverses direction at the stream bed. The resultant spiralling eddy is frequently referred to as a horseshoe vortex and rapidly removes material from around the foundation. The depth of scour is affected by the geometry of the pier and its foundation and depends on the pier width, length, shape and alignment. Other factors include the velocity and depth of flow, the type and size of bed material and the rate of bed transport. Pile foundations are often assumed to give more security against scour than spread footings (Lagasse *et al.*, 1995).

In most cases, the scour observed at any given bridge is likely to reflect a combination of these three types of scour, as well as other, less common, effects.

In assessing the failure of the Schoharie Creek Bridge, it was discovered that the footings were particularly vulnerable to scour because of inadequate rip-rap around the base of the piers and a relatively shallow foundation (Fotherby, 1993). Thus, problems of failure can be addressed either by changing the structure or by replacing any material that is washed away. The first solution typically involves altering the foundation. Enlarging the footing, strengthening or adding piles or providing a sheet piling barrier around the pier foundation can accomplish this. Replacement of material generally involves the placement of erosion-resistant material, such as rip-rap or broken concrete around the pier or abutment to offer a barrier to scour. In addition to alterations to the existing foundations, many bridge sites require the use of some type of training works to protect

the bridge and its approaches from damage by floodwater. They are used to stabilise eroding riverbanks and channel location in the case of shifting streams, and also to direct flow parallel to abutments and piers, thereby minimising local scour.

Summary

The selection of inherently durable materials is always an important part of the design process but it must be acknowledged that all known construction materials suffer

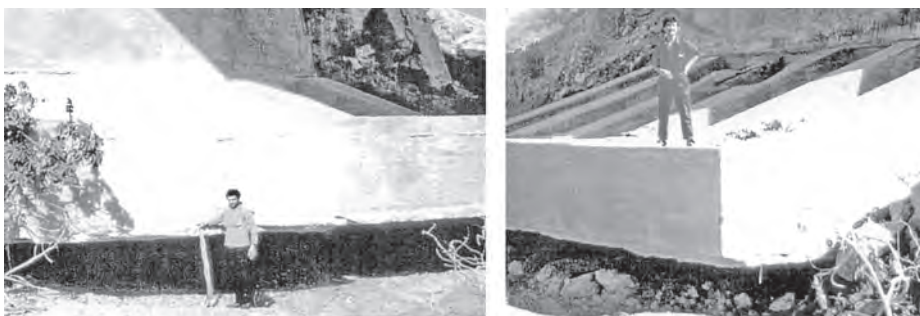


Figure 22.13 Example of flood-induced scour under bridge foundation (Alhinewa Bridge, Libya, courtesy of F Gergab)

from some form of deterioration over time. As a consequence, it is necessary to select materials that will provide adequate times to first maintenance based on the expected exposure conditions and the available funds for both initial construction and ongoing maintenance. Thus, the decision to use blended cement concretes, weathering steel or even stainless steels requires a good understanding not only of the potential benefits of such materials but also of the situations under which their use might lead to unexpectedly high rates of deterioration. In this respect, a sound knowledge of the causes of deterioration of structural materials and the methods that can be used to limit that deterioration is an essential first step.

It can be stated unequivocally that no available construction material is maintenance-free and so the bridge engineer remains responsible for ensuring that the final structure will be durable. Good design, irrespective of the choice of material, involves not just a knowledge of structural behaviour but also an appreciation of the effect of small design details on water management and, in turn, the likely long-term performance. This should allow the designer to incorporate modifications that will reduce the risk of unexpected deterioration and, at the same time, make the routine inspection and maintenance that will inevitably be required as simple and cost-effective as possible.

In conjunction with good design, there exists a range of methods for protecting both new and existing bridge structures from the extremes of any natural exposure. Perhaps the most common, and universally applicable, of all protection techniques is the use of surface coatings. The use of coatings, whether as barriers or sacrificial layers, implies an ongoing need for maintenance; this must be acknowledged when considering the whole-life cost of a bridge asset.

Finally, it is true to say that bridge engineers have never been faced with a greater array of materials and methods for protecting bridge structures from deterioration. As a consequence, it is important not to forget that bridges that are intelligently designed and constructed in accordance with normal standards and recommendations should be capable of providing adequate, cost-effective, performance over their design life.

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Chapter 23

Bridge management

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The objectives of a bridge manager are to ensure that bridges achieve their design life, that they remain open to traffic continuously and that their risk of failure is always very low. These objectives are to be achieved sustainably and at minimum lifetime cost. The chapter divides the subject into project- and network-level bridge management: the former is concerned with individual bridges, whereas the latter deals with the management of bridge stocks. Project-level management includes such aspects as inspection, testing and deciding maintenance requirements and appropriate prevention and remedial methods and monitoring strategies. Network-level management includes estimating the rate of deterioration, the prediction of future condition using Markov chain models, planning optimal maintenance programmes, prioritising maintenance and assessing the effectiveness of different maintenance strategies. Maintenance affordability, backlogs and long-term plans are also discussed, as are some important techniques used by bridge managers, for example, whole-life costing, probability modelling and sustainability assessment. The chapter concludes with a brief review of recent research on bridge management systems and the results of a survey on the type and functionality of bridge management systems used by local authorities and other bridge owners in the UK.

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Introduction

Bridge management (Das, 1996, 1999a) is concerned primarily with existing bridges; the objectives are to ensure that they achieve their design life, that they remain open to traffic continuously throughout their life and that their risk of failure is always very low. These objectives are to be achieved at a minimum lifetime cost.

The term *bridge management* encompasses a wide range of activities that are commonly encountered in the day-to-day management of bridges, such as inspection, assessment of load-carrying capacity and various types of testing. The results from these activities are used to prioritise the maintenance requirements. These aspects are covered comprehensively in other chapters of this book, so only those features that are pertinent to bridge management will be discussed in this chapter. Other important subjects associated with the management of individual bridges are the evaluation of their current condition and their rate of deterioration. This information is needed to help decide the most appropriate time to carry out maintenance work.

Many aspects of bridge management relate more specifically to the management of a stock of bridges. Examples of topics in this area include maintenance planning, prioritising and budgeting. Techniques that have been developed to aid the management of bridge stocks include whole-life costing, cost-benefit analysis, sustainability assessment and risk analysis.

These techniques often make use of economic or probabilistic models.

Bridge management involves making decisions (Frangopol and Estes, 1997), such as when a bridge should be maintained and what type of maintenance should be carried out. These are complex decisions that can have a major influence on lifetime costs and serviceability.

To make the best decision, the consequences of all possible decisions must be evaluated against criteria based on the objectives. Decisions are sometimes made from limited or inappropriate information, in which case they will tend to be conservative, with the result that more maintenance will be recommended than is actually required. Studies of bridge management decisions have established the type of information and algorithms that are needed to make sensible decisions. These data and the algorithms form a vital part of computer-based bridge management systems that have been developed during the last decade. How to make decisions is the key theme of this chapter.

Project- and network-level bridge management

Some aspects of bridge management are concerned primarily with the management of individual bridges (project-level bridge management), whereas other aspects are concerned with

the management of a stock of bridges (network-level bridge management).

Project-level aspects, such as the bridge condition, load-carrying capacity and non-destructive test results, clearly have a major influence on the timing and type of maintenance because they relate to a particular bridge. These decisions do not, however, depend entirely on factors associated with the particular bridge; they also depend on factors associated with other bridges in the stock. There are two ways in which conditions within the stock of bridges often affect the timing of maintenance for a particular bridge.

- When the maintenance budget is insufficient to undertake all the identified maintenance requirements, the work is prioritised; in a particular year, maintenance on one bridge may be deferred in favour of another bridge with a higher priority.
- The flow of traffic around the road network can be influenced by construction work and maintenance work to the pavement, bridges and street furniture; hence, these factors can influence the timing of maintenance work to a particular bridge.

The type of maintenance depends more closely on factors related to the particular bridge but, when there is more than one feasible maintenance type, the decision can depend on the maintenance management policy for the stock.

Network-level bridge management is more closely associated with the overall condition and serviceability of the stock and somewhat less concerned with the maintenance of individual bridges, although it is important to note that most of the input information for network-level algorithms is derived from project-level inspections, assessments and test results.

In this chapter, project-level aspects of bridge management will be considered next, followed by the network-level features, but it is to be remembered that both are vital parts of bridge management and are closely inter-related.

Project-level bridge management

Inspection

The main purpose of bridge inspections (DoT, 1983; Highways Agency, 1997a, 1997b) is to note any defects that could affect the safety or serviceability of the bridge or lead to a reduction in the life by accelerated deterioration. In the UK, three types of inspection are normally carried out. These are called general, principal and special inspections. General inspections are based entirely on visual observations made from any readily accessible position using aids such as binoculars and lamps to observe distant and dark locations. General inspections are made every 2 years.

Principal inspections are carried out every 6 years and are broadly similar to general inspections, except that observations must be made at a distance of less than 1 m from every part of the bridge. This requirement means that it is often necessary to use a hoist. The close proximity of inspector and bridge means that more detailed observations can be made, so the principal

inspection report is more comprehensive than the report for a general inspection. Sometimes a limited amount of testing is carried out during the principal inspection of concrete bridges to test for reinforcement corrosion. These tests measure the cover depth and half-cell potential of the reinforcement, the chloride content of the concrete and the depth of carbonation.

There are several ways to evaluate bridge condition; perhaps the most useful for bridge management purposes is to associate the condition with one of the major phases in the deterioration of a bridge element or component. Each phase is associated with a particular type of maintenance. The main phases of deterioration are:

1. progressive breakdown of protective systems such as paint, waterproofing membranes or expansion joints
2. physical deterioration of bridge elements or components leading to a reduction in life, which begins after protection is lost
3. significant damage, with possible hazards to users, for example, from falling lumps of concrete
4. substantial damage, which may have affected the strength of the bridge, producing a request for a special inspection and assessment of load-carrying capacity.

For a bridge in phase 1, preventative maintenance techniques should be effective; these are normally quick and easy to carry out, with the result that costs and disruption to bridge users are low. For a bridge in phases 2, 3 or 4, preventative maintenance will not be effective and the objective of maintenance is to stop or drastically reduce the rate of deterioration, reinstate the protective system and make the bridge safe for users. If the load-carrying capacity is assessed at less than 40 t, strengthening will be required. In general, as the deterioration progresses, the complexity, cost and associated disruption arising from the maintenance increases substantially. Deterioration in phase 1 is called primary deterioration, while that in phases 2, 3 or 4 is called secondary deterioration.

The condition assessment for different elements of a bridge recorded in the bridge management system (BMS) at each inspection provides three useful pieces of information.

- It is the input for algorithms to calculate the rate of deterioration and predict the future condition.
- It indicates the type of maintenance that is appropriate.
- It confirms that the inspection has been carried out.

This information helps the bridge manager to decide when to carry out maintenance and which method to use, and to confirm that inspections have been carried out on schedule. Inspections are the only routine way by which the bridge manager can be assured that bridges are in a safe condition and it is all too easy for the inspections of a few members of the stock to be overlooked. Using the BMS, it is straightforward to list any bridges where an inspection is overdue so that the problem can be corrected. In general, the conditions of the individual elements and components of a bridge are not combined to give

an overall condition for the bridge because it would be possible for the poor condition of one element to be masked by generally good condition of other elements, resulting in necessary maintenance being overlooked. An overall bridge condition can, however, be useful for assessing the general condition of the whole stock of bridges.

The frequency of general and principal bridge inspections is currently prescribed but, as more knowledge is gathered about the rate of deterioration, it may be possible to vary the frequency of inspections based on the bridge age and type of construction. This would enable the resources for inspections to be targeted at bridges with rapidly deteriorating elements or components. The improved information obtained would enable the most appropriate time for maintenance to be determined with greater precision.

The final type of inspection is called a special inspection and is carried out on an as-required basis. Special inspections are usually triggered by an adverse principal inspection report and are used to determine the cause and extent of deterioration. This information is needed prior to maintenance work to decide the extent and type of repair. Special inspections usually involve extensive non-destructive testing and material sampling.

Assessment

The assessment of the load-carrying capacity (Highways Agency, 1997a, 1997b) of a bridge is usually made when deterioration is considered to be enough to have reduced the strength or when the loading standard is changed. An assessment failure is a vital piece of information because it implies that essential maintenance is needed. Essential maintenance means that one of the following options must be selected without delay.

- Strengthen or replace the bridge so that it passes the assessment.
- Restrict the traffic with lane or weight restrictions so that only safe loads are carried.
- Monitor the bridge frequently; this option should only be considered if the failure would not be catastrophic and early indications of failure would be readily visible.

Clearly, an assessment failure has a major influence on both the time when maintenance is carried out and the type of maintenance.

Testing

Sampling and non-destructive testing (Bungey, 1982; DTI, 2006) are used in special inspections to establish the cause of deterioration, to locate latent defects and to estimate their rate of development. Latent defects can occur inside concrete or under protective coatings, such as waterproofing membranes on concrete or paint films on steel. Latent defects can also occur on parts of bridges that are not usually accessible for inspection, such as foundations, half-joints and deck ends. The

most common latent defect is corrosion of reinforcing steel. Corrosion of reinforcing steel ultimately disrupts the concrete but sometimes remains hidden, for example, in post-tensioning ducts or where pitting corrosion results from macrocell action. These tests are useful for confirming the phase of deterioration for a bridge element.

This information is primarily needed to determine appropriate maintenance methods and to find the extent of the repairs that are required. The information on the rate of the deterioration process can also act as an input to the algorithm predicting the best time for maintenance. The ability to look ahead in order to estimate the future condition of bridge elements can be useful for planning maintenance programmes and budgets. Estimates of the future condition of a concrete bridge is likely to be helpful in the following.

- Measurement of the chloride content of concrete at different depths from the surface, together with the cover depth of the reinforcing steel, enables the time to corrosion to be estimated. This is the time when the chloride concentration exceeds the threshold for corrosion to begin. Preventative maintenance is largely ineffective after this time.
- Measurements of corrosion rate, cover depth and concrete strength can be used to estimate the time when the concrete cover will crack owing to general reinforcement corrosion.
- Measurement of the rate of localised corrosion can be used to estimate the time when sufficient reinforcement cross-section has been lost to make the structure substandard.

An estimate of the timing of these three events can be invaluable for deciding the most appropriate maintenance strategy.

Although it is possible to measure the rate of reinforcement corrosion, an outstanding problem is to establish how much steel has been corroded in the period before testing started. One technique that can be used for this purpose is magnetic flux leakage, but it is still under development. Acoustic emission methods can be used to detect cracks or breaks in post-tensioning strands. Infrared thermography, impact echo, ultrasonics and ground-penetrating radar have been used to locate latent defects in concrete elements with varying success. Examples of commonly used tests are given in Table 23.1.

Deciding maintenance requirements

The information from inspections, assessment and tests should be sufficient to determine the maintenance requirements in most cases (Kreugler *et al.*, 1986). For example, an assessment failure indicates the necessity for essential maintenance, as previously described. The inspection data give the condition and some estimate of the rate of deterioration, which can be used to indicate the type of maintenance (preventative or repair) and its urgency. Test results establish the cause and extent of deterioration and hence suggest where maintenance should be carried out and the appropriate maintenance technique.

The bridge management objective that controls maintenance requirements is that serviceability must be fully maintained

Test	Purpose
Paint film thickness	To check for inadequate paint coverage on structural steelwork
Ultrasonic thickness gauge	To check the thickness of structural steel sections
Pit depth gauge	To measure the depth of pits in structural steelwork or reinforcing steel
Chloride sampling	To measure the chloride content of concrete at different depths from the surface
Carbonation depth	To measure the thickness of the carbonated layer of concrete near the surface
Cover depth	To measure the depth of the reinforcing steel
Delamination soundings	To locate areas of steel–concrete delamination
Half-cell potential	To locate areas where the reinforcing steel is corroding
Corrosion probes	To assess the corrosivity of concrete and monitor the progressive ingress of chlorides and carbonation
Corrosion rate	To estimate the rate of loss of steel cross-section
Amount of corrosion	To estimate the total reduction in cross-section of reinforcement

Table 23.1 Examples of sampling and non-destructive tests used for special inspections

throughout the design life of the structure at a minimum life-time cost. At one extreme, it could be decided not to do any maintenance work. In this case, no maintenance costs would be incurred, but the bridge may become unserviceable at some point during its design life. When the bridge fails its assessment, traffic restrictions will be necessary in the absence of maintenance. The costs associated with the management of traffic and the delays to users can be very large on busy roads and are often much greater than maintenance costs.

At the other extreme, it may be decided to carry out the maintenance work as soon as deterioration is detected. This is not usually efficient because if the rate of deterioration is low there are few adverse consequences of delaying maintenance and the money saved can then be used to maintain another bridge that is deteriorating quickly and has a higher priority for maintenance.

Except in the situation when assessment failure means that essential maintenance is required immediately, the decision about maintenance requirements should be based on a consideration of the consequences of (a) not maintaining, (b) preventative maintenance and (c) making repairs now and at various times in the future. For example, if, on the basis of the current condition and the rate of deterioration, it is decided that the condition at the end of the design life will not lead to an assessment failure, then maintenance can be deferred indefinitely. This decision is only valid for a limited period and does not commit the manager to not organising any maintenance on the bridge until the end of its design life. The circumstances are reappraised from time to time when new evidence from inspections, assessments and tests can be considered; if the rate of deterioration is shown to have increased, the previous decision can be altered.

The purpose of preventative maintenance or repairs is to increase the age of the bridge when essential maintenance eventually becomes necessary. In other words, preventative maintenance and repairs reduce the rate of deterioration.

Selecting an appropriate maintenance method

First, it is necessary to decide the type of maintenance (preventative, steady-state repair or essential strengthening) required, on the basis of assessment and condition information (Al-Subhi *et al.*, 1989). Routine maintenance, which includes such operations as clearing blocked drains and removing vegetation, is not considered here because it is carried out regularly, regardless of the findings from inspections, assessments and tests, as the name implies. Second, an appropriate maintenance method (OECD, 1981) needs to be selected; this depends on the cause and rate of deterioration, the current condition and the element of the bridge affected.

Essential maintenance generally involves strengthening or replacement of bridge elements. Strengthening techniques include welding, plate bonding and external post-tensioning, which increase the stiffness of bridge decks. Replacement of elements has been used for deck slabs and beams, piers and columns. The primary purpose of essential maintenance is to increase the load-carrying capacity; the reason for the inadequate capacity is secondary. If the reason is simply increased loading, the maintenance can be limited to increasing the capacity, but if the reason is deterioration then maintenance must also include repairs and preventative maintenance.

The selection of the maintenance method for steady-state repairs and prevention depends primarily on the cause of deterioration. For steel construction, the main cause of deterioration is corrosion; regular maintenance painting should be carried out to prevent the steel from corroding. If corrosion does occur, the only repair option is to grit blast back to shiny metal before repainting. An assessment of load-carrying capacity should be carried out if corrosion has resulted in a significant reduction of steel section.

The selection of repair and prevention methods for concrete construction is more complex because there are numerous

causes of concrete deterioration. The deterioration of reinforced concrete can be conveniently subdivided into deterioration of the concrete and deterioration of the steel reinforcement. The main causes of concrete deterioration are sulphates, freeze–thaw cycles and alkali–silica reaction (ASR). Deterioration can also be related to poor mix design and construction processes, such as compaction and curing. These types of deterioration can only be prevented by actions taken at the time of construction; there are no effective preventative actions that can be taken after construction. For example, where the environment is known to contain significant quantities of sulphate or sulphide, it is sensible to consider the use of sulphate-resisting Portland cement. In regions experiencing large numbers of freeze–thaw cycles, frost damage to concrete can be prevented by adding air-entraining agent to the concrete mix. Frost damage is worse in concrete that is saturated with salty water, so such techniques as waterproofing membranes and silane treatments may be helpful. Alkali–silica reaction between aggregates and the alkali in cement can be prevented by avoiding the most reactive types of aggregate and by keeping the alkali content of the cement below the designated limit. To set up damaging stresses in concrete, the ASR requires water, so procedures to reduce the water content of concrete, such as waterproofing membranes and silane treatments, may help. If these forms of concrete deterioration take place, the only viable repair method is concrete replacement, which may be extensive, especially for ASR, where entire sections can be affected. Sulphate and freeze–thaw damage normally occur only in the cover zone of the concrete. It is important to note that deterioration of the concrete will increase the risk of corrosion to the reinforcement because steel de-passivators, like chlorides and carbon dioxide, will be able to move more easily through the concrete to the reinforcement.

Deterioration of the reinforcing steel is caused by corrosion and can be prevented by actions taken at the time of construction and for a period after construction. Preventative techniques that can be applied at construction include the use of epoxy-coated mild steel, stainless steel or carbon or glass fibre reinforcement, inhibitors, cathodic protection, anti-carbonation coatings, silane treatments and waterproofing membranes. All but the last three of these techniques directly protect the reinforcement against corrosion and, to date, have been used only occasionally, largely on grounds of cost. Waterproofing membranes, silane treatments and anti-carbonation coatings are applied to the concrete and are designed to slow down the ingress of carbon dioxide and chlorides into the concrete, thereby increasing the age of the structure at which the reinforcement begins to corrode. These techniques can be used after construction because they are applied to the concrete surface; they should be effective, providing corrosion of the reinforcement has not already begun. It is important not to overlook the importance of well compacted and cured low water/cement ratio concrete in preventing reinforcement corrosion.

When corrosion of the reinforcement occurs, it results in a loss of steel section or cracking, spalling and delamination of the concrete, owing to the stresses produced as a result of

the low density of rust compared with the density of the steel. Reinforcement corrosion repair methods have two main functions; to stop the corrosion and to repair the damaged concrete. A number of techniques are available:

- concrete replacement
- cathodic protection
- desalination
- realkalisation.

Concrete replacement must be used to repair the damage caused by corrosion, regardless of which technique is used to stop corrosion. Concrete replacement can also be used to stop corrosion, although this involves the removal of all the carbonated and chloride-contaminated concrete even if it is physically sound. This often means that concrete repairs to stop corrosion are not economically viable. Cathodic protection can be applied at any time to stop corrosion caused by carbonation or chlorides. It functions by making the reinforcing steel cathodic with respect to an external anode system. Cathodic protection requires a permanent electrical installation. Desalination can be used to stop corrosion caused by chlorides; it works by migrating chloride ions towards an external anode and away from the reinforcing steel in an electric field; this process takes about 6 weeks. Realkalisation stops corrosion caused by carbonation, and works by migrating sodium ions from an external anolyte into the concrete where, in combination with the hydroxyl ions generated on the reinforcing steel due to the electric field, the alkalinity is raised to a level where the steel re-passivates. Realkalisation takes about 4 weeks. Desalination, realkalisation and concrete repair are normally used in conjunction with a preventative treatment, such as silane or an anti-carbonation coating, to increase the life of the repair. Cathodic protection does not require additional preventative measures because it is a permanent installation, but the anodes do require periodic replacement.

Bridge monitoring and maintenance strategies

Previous sections of this chapter have described the types of information that can be obtained from inspections, assessments and non-destructive testing, and how this can be used to decide the maintenance requirements and appropriate maintenance methods. In this section, monitoring strategies for concrete bridges suffering from salt-induced reinforcement corrosion will be discussed. This example is chosen because nearly all highway bridges are affected by reinforcement corrosion at some time during their life and because reinforcement corrosion often results in latent defects and is mechanistically complex.

A major challenge for bridge managers is to make the best use of the resources available to achieve the objectives. If resources were unlimited, there would be little difficulty in

using inspections, assessments and tests to locate defects and apply remedial measures. One way in which attempts have been made to conserve resources is by utilising preventative maintenance methods, such as waterproofing membranes for the top surface of bridge decks and silane treatments to other concrete surfaces exposed to de-icing salt in order to reduce future maintenance requirements. Even so, in practice, resources have been tightly constrained and the information on which to base decisions has been limited. Decisions based on limited information are usually either conservative or result in an increased risk of failure. Conservative decisions lead to expenditure on unnecessary maintenance work. An increased risk of failure can arise when latent defects are not detected. The philosophy of the maintenance strategy described here is the targeting of resources to generate more relevant information.

In the strategy that is currently typically adopted, bridges are inspected regularly every 2 years (general inspection) and 6 years (principal inspection). Both types of inspection are almost entirely visual, the distinction being that the inspector should be within arm's length of each part of the bridge for a principal inspection, whereas for a general inspection any convenient viewing position may be used. The first principal inspection is supplemented by a very limited amount of non-destructive testing in two small test areas vulnerable to corrosion, such as the base of a pier and a location under a leaking expansion joint. The result of this inspection process is that reinforcement corrosion is detected but only after it has reached an advanced stage and caused substantial damage. There is also a concern that pitting corrosion, which causes significant loss of strength, could remain undetected.

When the damage caused by reinforcement corrosion has been detected, an extensive non-destructive test survey is carried out to locate latent areas of corrosion in order to specify the overall extent of the repair work (Pearson and Patel, 2002). The repairs have three main functions; namely, to reinstate the damaged concrete and reinforcing steel, to stop the corrosion process and to prevent its reoccurrence. Damaged concrete is cut out and replaced with a suitable concrete repair material. Corroded steel can be cleaned and severely damaged bars can be supplemented with additional steel. The corrosion process is normally stopped by applying an electrochemical treatment, such as cathodic protection. This will also protect against corrosion reoccurrence over the area to which the treatment is applied. There remains, however, a significant chance that corrosion will occur in the future, outside the region of cathodic protection. The life of concrete repairs and cathodic protection is thought to lie in the range 10 to 25 years.

The weaknesses of this strategy for combating reinforcement corrosion are (a) that maintenance and repair are not carried out at the most advantageous stage of the deterioration process and (b) insufficient account is taken of the fact that corrosion could easily occur in the future on another part of the bridge that has not been repaired and protected. This results in more extensive repair work and a higher frequency of repair visits for a bridge.

To improve the strategy, it is necessary to carry out maintenance to prevent corrosion from initiating and to consider the

maintenance of the bridge as a single entity rather than as a collection of constituent parts.

Clearly, the easiest way of achieving this would be to apply preventative maintenance to the entire bridge stock and perform regular non-destructive testing surveys to find the best time to renew the preventative treatments, thereby preventing corrosion initiation. In practice, this would be very expensive and disruptive. It is also not efficient because some bridges or parts of bridges can tolerate a certain amount of corrosion without serious consequences and the case for preventative maintenance in these locations is, therefore, weak. The main consequences of allowing corrosion to initiate are reduced strength, danger from spalling concrete and the high cost and disruption resulting from repairing damage caused by corrosion. The seriousness of these consequences depends on where the corrosion occurs and will vary from bridge to bridge.

The structurally critical areas of a bridge can be determined from the design calculations or from a load assessment of the bridge during service. The reserves of strength available at these critical locations will vary from bridge to bridge; in cases where the reserves are high, it is unlikely that corrosion could cause sufficient weakening to make the bridge unsafe. In some cases, even when the reserves are low, it is possible that the critical location will not correspond to a location vulnerable to reinforcement corrosion and the rate of deterioration at this location will therefore be low, so again it is unlikely that there will be sufficient weakening to make the bridge unsafe. Thus, a simple risk model that considers the probability and consequences of corrosion occurring at structurally critical areas will help in deciding where monitoring and maintenance will be beneficial.

Risk is a much-used word that is often informally substituted for the probability of an event occurring. This usage is, however, technically incorrect because risk is actually a variable that combines the consequences and probability of an event occurring with equal weighting. In this study, the event to be considered is reinforcement corrosion at a particular location on a bridge. The locations are the most structurally critical points on the bridge that can be found from the design calculations or a load assessment. The consequences of corrosion fall into two categories, safety and economic. Five main types of consequence are considered – collapse, spalling, repair cost and user disruption during repairs. The probability of corrosion occurring at a given location depends on factors such as cover depth, water/cement ratio of the concrete, level of exposure to salt water and the age of the bridge. The relationship between risk, probability and consequences is shown in Figure 23.1, where risk is divided into four bands (very high, high, moderate and low).

The rankings from the risk model will permit the effective targeting of monitoring and preventative maintenance, thereby overcoming the high cost and low efficiency of blanket monitoring and preventative maintenance applied to all the exposed surfaces of every bridge. The interpretation of risk data requires a degree of engineering judgement. For example, the authors judge that monitoring probes should be installed at all

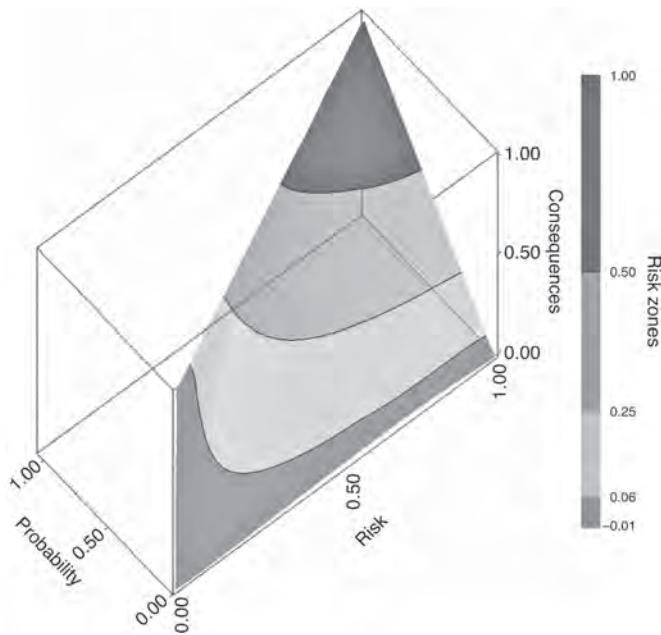


Figure 23.1 Relationship between probability, consequences, risk score and risk zone

structurally critical points rated to be at very high or high risk.

The next question to consider is when preventative maintenance should be renewed in these high-risk locations. The effective life of a preventative maintenance treatment ends when the rate of chloride ingress into the concrete increases. The progress of chloride ingress can be monitored automatically using ladder probes (Schiesl and Raupach, 1992) embedded in the concrete at high-risk locations. A schematic diagram of a ladder probe is shown in Figure 23.2.

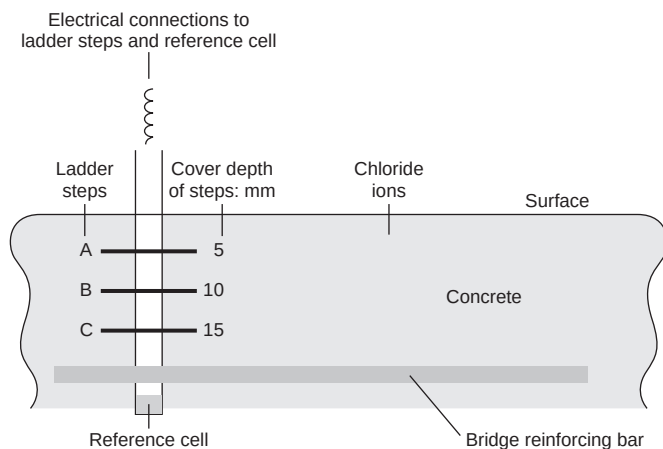


Figure 23.2 Ladder probe in concrete

A ladder probe consists of a number of electrically isolated pieces of reinforcing steel, each one at a different cover depth. The half-cell potential of each of these pieces of reinforcement can be automatically recorded and logged using an embedded silver–silver chloride reference electrode that forms a part of

the ladder probe. As the chloride passes into the concrete, each piece of steel will activate in turn, as indicated by its half-cell potential, starting with the one nearest to the concrete surface, as shown in Figure 23.3. Knowledge of the cover depth of the bridge reinforcement at the probe location together with the ladder probe data will provide an early warning of the onset of bridge corrosion and a measure of the rate of chloride ingress.

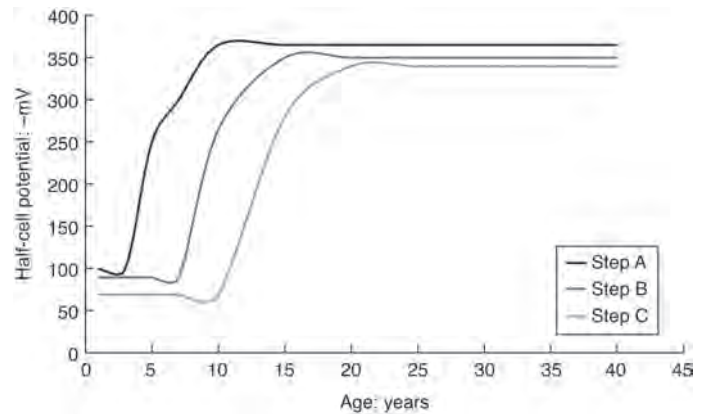


Figure 23.3 Typical results from a ladder probe

For example, the ingress of chloride into concrete approximately follows a square law; hence, for a ladder probe with equally spaced steps, if the time for the first probe to activate is x years and the bridge reinforcement coincides in depth with the n th step of the ladder, then the time to corrosion of the bridge reinforcement in that location will be about x^n years. As a numerical example, if the time to activation of the first step of the ladder is 3 years and the reinforcement coincides with the third rung of the ladder, the time to corrosion would be 3^3 , that is, 27 years. Since the risk analysis can be carried out at the design stage of a bridge, the location of the ladder probes can be included in the design and installed during construction. The use of a ladder probe is much more reliable than the results of models for chloride ingress based on Fick's second law of diffusion. These models give only a very approximate prediction of future chloride concentration at the depth of the reinforcement. The errors mainly arise because of the assumption that corrosion will initiate when the chloride concentration attains the threshold level. In practice, the threshold level can vary from 0.2% to 1% chloride ions by weight of cement and there is no easy way of determining its value for a particular location. This uncertainty leads to a wide variation in the possible values of time to onset of corrosion. Another problem is that chloride ion transport in concrete occurs by capillary absorption as well as by diffusion; hence, Fick's law is not appropriate for modelling. The ladder probes combine information about the rate of chloride ingress and the thermodynamics of corrosion initiation; hence, they do not suffer from the problems associated with the Fick's law model. A disadvantage of the ladder probe is that the number of positions that can be examined is limited to the number of probes that can be installed. The extent of deterioration can be checked using a non-destructive

test (NDT) in the vicinity of the ladder probe when the latter indicates that the corrosion front is approaching the reinforcing steel or the rate of deterioration is increasing. The main advantages of the embedded ladder probe over a surface NDT is that measurements can be made automatically as often as required without needing access to the bridge. Figure 23.4 describes the improved strategy for managing reinforcement corrosion in bridges.

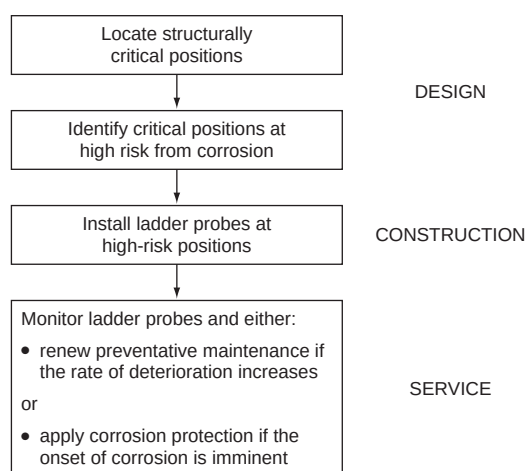


Figure 23.4 Process for preventing reinforcement corrosion of structurally critical locations on bridges

To summarise, non-destructive tests are available for monitoring reinforcement corrosion and can be used to apply preventative maintenance at the appropriate time so that corrosion and the damage it causes to bridges does not occur. Currently, these tests have not been adopted, largely because of the cost of their widespread application. It has been shown that combining structural and corrosion information in a simple risk model, applied at the design stage and with limited data requirements, enables the testing budget to be targeted at locations where there is a high probability that corrosion could cause substantial damage to the bridge. The reduction in testing sites that results from using the risk model means that it is feasible to

embed monitoring probes, enabling automatic monitoring at any desired frequency without the need for access or traffic management.

Network-level bridge management

The development of relational database systems (Gusella *et al.*, 1996) during the past decade has enabled bridges to be managed as a stock. This has a number of potential advantages.

- Design features and materials associated with defects can be identified.
- The cost-effectiveness of different maintenance methods can be established.
- The influence of bridge maintenance on traffic flow can be examined.
- The rate of deterioration of a particular bridge can make use of information on the deterioration of bridges of similar type and age elsewhere in the stock.
- Maintenance work on different bridges in the stock can be prioritised rationally.
- Maintenance programmes and budgets for the stock can be planned.
- The performance of a maintenance programme can be evaluated in terms of changes in the overall condition of the bridge stock.

Computer-based databases and their associated analysis algorithms are called bridge management systems (BMSs) (FHA, 1987). They consist of a number of modules, such as

- inventory
- inspection, assessment and test records
- maintenance records
- economics of maintenance methods
- economics of traffic disruption and management
- rate of deterioration
- optimised and prioritised maintenance programmes.

Inventory category	Example
General	Bridge number, bridge name, structure type, grid reference, road number, obstacle crossed, year built, maintenance agent, main material, number of spans
Route	High vehicle route, heavy vehicle route, salted, weight restriction, traffic flow, heavy goods vehicles, minimum width, minimum clearance
Span	Span number, span length and width, skew, deck type, main and secondary materials
Supports	Support reference, type, substructure type, foundation type
Public utility	Type of utility, date notified
Components (joints, bearings, parapets, waterproofing membranes)	Component reference, span cross-reference, support cross-reference, component type, date of installation
Prestressing	Prestressing type, location

Table 23.2 Typical inventory categories

Originally, a BMS consisted only of an inventory, whose primary function was the secure storage and easy retrieval of data on individual bridges. The primary function of these first-generation BMSs was to facilitate the day-to-day management of bridges by generating database reports. An inventory is an essential feature of all BMSs and is the basic store of data for all bridges in the stock.

A BMS inventory typically consists of about 200 data fields. Examples of data fields in a simple inventory are listed in Table 23.2. The data can be used to generate an almost unlimited number of reports to answer questions that may arise in the day-to-day management of bridges. A typical example of a report is given in Table 23.3.

Query	List the post-tensioned bridges built before 1960 that have no waterproofing membrane.
Report fields	Bridge no Bridge name Grid reference Year built Road Post-tensioning type

Table 23.3 Example of an inventory report from a simple BMS

Subsequent developments were made in response to the need to store information about inspections, assessments, tests and maintenance work. These are second-generation BMSs. They enable the manager to check for bridges where planned maintenance work and inspections had been inadvertently overlooked, a situation that can easily occur when managing a stock of a thousand or more structures. This feature is essential for providing assurance that all the bridges in the stock are safe and serviceable. The information stored in these additional modules allows more complex reports to be generated (see Table 23.4) and provide vital input data for the algorithms used in third-generation BMS.

Query	List all overbridges built before 1990 where a pier in the central reservation has a condition state less than 3 and has not been treated with silane.
Report fields	Bridge no Bridge name Grid reference Road Year built Condition state

Table 23.4 Example of a report from a second-generation BMS

Third-generation BMSs have the general objective of maintaining serviceability of the bridges in the stock at a minimum lifetime cost. This objective requires (a) modules providing

information on the economics (cost and lives) of maintenance methods and their impact on the flow of traffic and (b) algorithms for finding the rate of deterioration and optimised and prioritised maintenance programmes for the bridges in the stock. The optimised programme minimises lifetime costs, whereas the prioritised programme minimises lifetime costs subject to the constraint that the budget available for maintenance work is less than the cost of the optimised maintenance programme. Most of the remainder of this chapter will concentrate on the data and algorithms required in order to predict future deterioration and generate optimised and prioritised maintenance programmes.

Rate of deterioration

To produce a maintenance programme for a bridge, it is essential to know how quickly it will deteriorate. If deterioration is slow, maintenance can either be deferred until the bridge is older or a simpler and cheaper type of maintenance can be adopted. Conversely, if deterioration is rapid, it may be necessary to bring the maintenance work forward. For a particular bridge with good inspection, maintenance and test records, it should be possible to estimate the rate of deterioration up to the present time and maybe slightly into the future using extrapolation. By using information from the inspection, maintenance and test records of other bridges in the stock, of a similar type of construction and material, when they were the same age as the bridge under consideration, it should be possible to make a better estimate of the rate of deterioration and predictions about the condition in the future. Rates of deterioration (Hogg and Middleton, 1999) can, in principle, be determined from either inspection or test data. The results from physical tests offer the most direct approach, although this is far from straightforward. It is possible to devise models describing deterioration processes. These models are complex, reflecting the nature of the physical processes, and a specific model is required for each deterioration mechanism. Furthermore, a deterioration process may involve a number of different mechanisms operating simultaneously or at different stages of the process.

Consider, for example, the deterioration caused by corrosion of reinforcing steel in concrete by de-icing salt. This process occurs in two distinct stages. In the first stage, chloride ions from the de-icing salt slowly penetrate the concrete cover until they reach the reinforcement. In the second stage, corrosion begins, when the chloride concentration of the concrete adjacent to the reinforcement exceeds the threshold value, and then propagates at a rate dependent on such factors as the concrete resistivity, potential difference between anodic and cathodic sites, cathode/anode area ratio, rate of diffusion of oxygen and the electrochemical corrosion mechanism. In the first stage, there are two mechanisms of chloride ingress (diffusion and absorption); in the second stage, corrosion can result in a non-uniform reduction in reinforcement cross-section (localised corrosion) or a loss of bond between the steel and concrete, owing to cracking, spalling or delamination of the concrete (general corrosion).

From this discussion, it can be seen that at least four models would be needed to describe the reinforcement corrosion process. A large body of data would also be required and, as this could only be obtained by expensive non-destructive testing, the inevitable conclusion is that the physical model approach is not feasible for finding the rate of deterioration of thousands of bridges. Nevertheless, considerable work has been undertaken to establish physical models, but so far the results have been very inaccurate. The root cause is a lack of knowledge and real data about the physical processes involved, but the modelling work has demonstrated the variability of deterioration rates among nominally similar bridge elements.

The other approach (Saito and Sinha, 1990) is to use the condition state assessments made during bridge inspections. The information already exists and can be collected cheaply. Condition measuring using the condition state assessments made by bridge inspectors is normally based on a set of discrete states, which represent different stages in the deterioration process. There is a close association between the condition state and the appropriate type of maintenance that simplifies the interpretation of the condition measure. Different materials have different deterioration processes; hence, it is necessary to set up a condition state scale for all the common deterioration processes and materials of construction. A condition state scale consists of a number of states, each of which is given a numerical value and a definition describing the stage of the deterioration process. The number of states is normally between 3 and 10. If the assessment is based entirely on visual observations, the number of states normally lies at the lower end of this range, whereas more states can be used if non-destructive tests and material sampling are used. When too few states are used, the deterioration process is not adequately described, but if too many states are used it becomes difficult to differentiate between them, resulting in different inspectors making different condition assessments on the same element. Deterioration sometimes results in the formation of latent defects; in these cases, it is recommended that the condition state should be based on visual observations, NDT and sampling. An example of a typical condition state scale for corrosion of reinforced concrete is discussed earlier in this chapter.

The general procedure is as follows.

1. Subdivide the bridge stock into sets of bridge elements with characteristics that indicate that they should deteriorate by similar mechanisms. Factors that are most likely to influence the deterioration process are the construction material, geographic location and when and how the element was last maintained. As more information is obtained about deterioration processes, the subdivision can be refined, with the proviso that the number of bridges in each set is statistically significant.
2. For each set of bridge elements, the data on condition state as a function of age for each element are aggregated to generate a function relating the average condition state of the set with respect to age. This function cannot be used to predict the future condition of a particular element because

its current (condition state, age) ordered pair is unlikely to lie on the average line.

3. Create a Markov chain (Jiang *et al.*, 1989) for the deterioration process starting from a known situation, such as 'the condition state at age 0 has the value 1'. (For a five-point deterioration scale, condition state 1 represents an undeteriorated state, whereas condition state 5 represents an unserviceable state.) The Markov chain calculates the probability of a typical element being in a particular condition state at a particular age.
4. An optimisation procedure is utilised to minimise the difference between the condition states found for a given age by the procedures described in steps 2 and 3. The condition state probabilities from the Markov chain associated with this minimal difference represent the condition state transition probabilities for the deterioration process based on the historic condition state–age data used in step 1. These transition probabilities are the most accurate representation of the deterioration process based on the available data. A condition state transition probability is the probability that an element currently in state x will move to state y by the next inspection. Transition probabilities are used to represent the rate of deterioration because the condition state scale consists of discrete points rather than a continuum. Two reasonable assumptions are made to simplify the mathematics: (a) the condition state does not decrease with increasing age and (b) between consecutive inspections, the condition state either remains constant or increases by one unit. These assumptions produce a relatively sparse transition state matrix that reduces the number of computations needed to predict future condition state values.
5. The predicted condition state in future years is obtained by matrix multiplication between the current condition state vector and the transition probability matrix. The condition state vector relates to the current condition of the particular bridge for which the prediction is being made. It is a five-element row vector representing the probability of the element being in each condition state. The transition matrix represents the historic evidence on how elements of a similar type and age have deteriorated. Thus, predictions are based on the most recent data on the condition state of the particular element and the historic information about how similar elements have deteriorated. This is a rational basis for making predictions.

This description of how deterioration rates are obtained from inspection data is unavoidably complex but it is intended to give a flavour of the process for those with limited mathematics.

A mathematical description follows, which some readers may prefer. The same step numbers are used.

Step 1 is the same.

2. The average condition state against age function usually fits quite well to a polynomial equation, such as

$$C(t) = a + bt + ct^2 + dt^3 \quad (23.1)$$

where $C(t)$ is the average condition state of the stock at time t years and a, b, c and d are polynomial coefficients.

- The beginning of the Markov chain is represented by the tree diagram shown in Figure 23.5. The numbers at the nodes represent the condition state and the numbers associated with the branches of the tree, such as p_{xy} , represent the transition probability of going from state x to state y between consecutive inspections. Note the simplifying assumptions

$$y \geq x \quad \text{and} \quad y = x \quad \text{or} \quad y = x + 1 \quad (23.2)$$

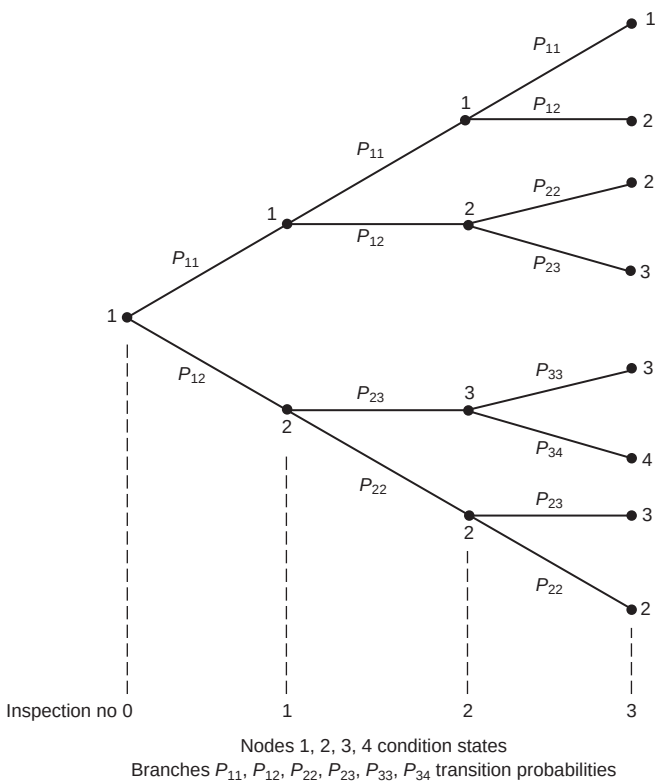


Figure 23.5 Markov chain diagram

Figure 23.5 can be used to determine the probability of being in a given state at a given time and to determine the average condition state at a given time. For example

- the probability of being in state 2 at the second inspection is given by

$$p_{11}p_{12} + p_{12}p_{22} \quad (23.3)$$

- the average condition state at the second inspection is given by

$$C_m(t, w) = p_{11}^2 + 2(p_{11}p_{12} + p_{12}p_{22}) + 3p_{12}p_{23} \quad (23.4)$$

where $C_m(t, w)$ is the average condition state at time t determined by the Markov chain and w is the set of probabilities p_{xx} .

Thus, to find $C_m(t, w)$, it is necessary to know the values of p_{11}, p_{22}, p_{33} and p_{44} .

For a five-point condition state scale, $p_{55} = 1$, by definition. It follows from the simplifying assumptions that

$$p_{x,x} + 1 = 1 - p_{xx} \quad (23.5)$$

The transition probabilities are usually combined in a matrix, P , called the transition state matrix.

Thus

$$A = \begin{bmatrix} p_{11}p_{12} & 0 & 0 & 0 \\ 0 & p_{22}p_{23} & 0 & 0 \\ 0 & 0 & p_{33}p_{34} & 0 \\ 0 & 0 & 0 & p_{44}p_{45} \\ 0 & 0 & 0 & 0 & p_{55} \end{bmatrix} \quad (23.6)$$

Note that for a five-point condition state scale the transition matrix is 5×5 .

- The optimisation to find the transition state probabilities is based on the minimisation of objective functions, such as

$$\min \sum_{t=0}^{t=4} |C(t) - C_m(t, w)| \quad (23.7)$$

The value of the set w corresponding to the minimisation gives the transition probabilities. Thus

$$w_{\min} = \{p_{11}, p_{22}, p_{33}, p_{44}, p_{55}\} \quad (23.8)$$

The minimisation is obtained by cycling through the values of p_{11}, p_{22}, p_{33} and p_{44} from 0 to 1, using a step size of 0.01, until the minimum is reached.

Consider a condition state row vector $V = [p_1 \ p_2 \ p_3 \ p_4 \ p_5]$, where $p_1, p_2, \dots, p_5$ are the probabilities of being in condition states 1, ..., 5 at a particular time. For a five-point condition state scale, the condition state vector has five elements.

The condition state vector after n inspections is given by

$$V(n) = V(0)P^n \quad (23.9)$$

where $V(0)$ is the condition state vector at age zero, for example, $[1, 0, 0, 0, 0]$, indicating that a new bridge element is normally in condition state 1 and not in any other condition state.

Multiplying the condition state vector $V(n)$ by a five-element column vector representing the condition states gives the predicted condition state after n inspections $C(n)$.

If $V(n) = [p_1 \ p_2 \ p_3 \ p_4 \ p_5]$ then $C(n) = V(n)S$, where $S = [1, 2, 3, 4, 5]^T$.

This completes the description of how the condition state of an element in future years can be predicted. It is important to remember that predictions become less reliable the further one extrapolates into the future. It is, however, only the predictions for the next few years that will have a bearing on immediate maintenance activities; these predictions should be reliable. The predictions should be recalculated periodically, including the results of the most recent inspection assessments of condition state in the input data for the algorithm. The results of

the predictions are normally represented as a condition state trajectory, where a condition state is matched with each future inspection. An example of a condition state trajectory is

Condition state 1 1 1 2 2 2 3 3 4 4 5
Inspection number 0 1 2 3 4 5 6 7 8 9 10 (23.10)

The technique described here for predicting the condition of an element in future years is restricted, by the simplifying assumptions, to elements that have not undergone maintenance. Although the restriction includes most bridge elements, there are a significant number that have been maintained and will be excluded. The technique described could be used for maintained elements if the assumptions were relaxed. In particular, the assumption stating that the condition state cannot decrease would need to be relaxed. The amount of condition data from bridges following maintenance is currently very limited and an alternative approach is needed at present. One approach is to associate two parameters with each maintenance method. These parameters are

1. The improvement in condition resulting from maintenance work

$$\text{Improvement } (I) = \text{condition state before maintenance} - \text{condition state after maintenance} \quad (23.11)$$

2. The life, L , which is the number of years following maintenance that the condition state remains constant, before deterioration recommences.

For example, consider a bridge element of age 40 years with a condition state of 3 that undergoes a maintenance treatment with $(I, L) = (1, 20)$, it is predicted that the condition state will improve to state 2 ($3 - 1$) following the maintenance and then maintain this value for 20 years until the element is 60 years of age. A comparison of condition state trajectories with and without maintenance provides a measure of the benefits of maintenance that can be compared with the costs (Table 23.5).

Without maintenance condition state	3	4	4	4	5	5
Age: years	40	45	50	55	60	65
With maintenance condition state	3	2	2	2	2	3

Table 23.5 Condition state trajectories with and without maintenance

The experience for most maintenance methods is limited because they have only been used widely during the last decade; therefore, while knowledge of bridge improvement is good, information about life is less reliable at present. It is important to revise the values of improvement and life for maintenance methods regularly to take account of new data recorded in the inspection and maintenance modules of the BMS. Now that a procedure has been established to predict the future condition of bridge elements, we are in a position to investigate the optimisation of lifetime maintenance costs.

The optimal maintenance programme

The optimal maintenance programme (Jiang and Sinha, 1989) states which elements of which bridges should be maintained by specified methods each year such that the lifetime maintenance cost of each bridge is minimised. The general approach to optimisation and the various options that are available are best understood by considering the tree diagram in Figure 23.6. Each node of the tree is associated with a particular condition state, calculated using the techniques described in the previous section. Each vertically aligned set of nodes corresponds to a particular inspection of the bridge element. The branches emanating from a node represent a number of maintenance options, of which one is a 'no maintenance' option. The condition state of the node determines the type of maintenance that is appropriate and hence limits the number of possible maintenance methods to about two or three. The number of possible maintenance methods equals the number of branches emanating from a node. The number of branches from each node in Figure 23.6 is fixed for convenience only; in practice, the number of branches can vary with the condition state. Each branch has an associated cost, discounted at a rate selected by the operator.

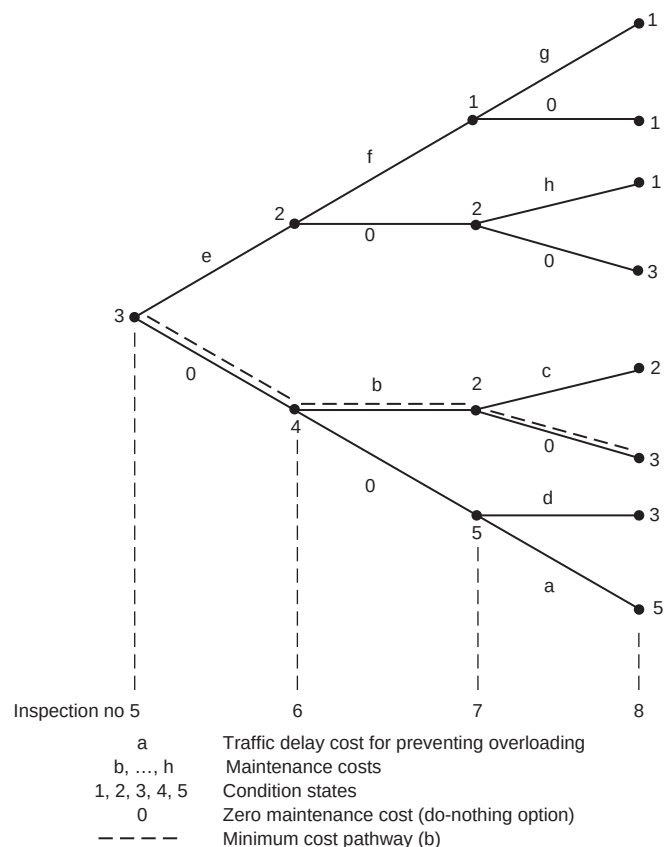


Figure 23.6 Optimisation tree diagram for two maintenance options per node

Thus, an important part of the optimisation module is a library of maintenance costs. These costs are subdivided into three types:

- engineering
- management of traffic
- traffic delays.

The operator can use any combination of these three types of cost. The optimisation is based on the sum total of the cost types selected, although each cost can be itemised separately. These options for the operator are necessary because sometimes the influence of maintenance work on traffic delays will not be required. Engineering costs depend primarily on the maintenance method, whereas the traffic-related costs depend mainly on an element being maintained and traffic density.

The optimisation process consists, basically, of calculating the total cost of every path through the tree and identifying the path with the lowest total cost. The year and maintenance method associated with the branches of this path of minimum cost give the optimal maintenance programme. In practice, this would result in an excessive number of computations, taking a large amount of computer time. For example, if each node has three branches and optimisation is required over the next 20 inspections, then $320 \sim 3 \times 10^9$ pathways would have to be calculated. This number can be reduced by a factor of about a million by using a technique called dynamic programming (Winston, 1991), together with the provision of some information about the last maintenance treatment.

The operator of the optimisation module has the option of applying constraints to the optimisation. When no constraints are used, preventative maintenance and repairs are only carried out if they reduce the lifetime cost. Essential maintenance has to be done, but it could involve traffic restrictions instead of engineering work; therefore, it is important to include traffic management and delay costs if it is decided to use optimisation without constraints. Optimisation without constraints is the purest form of optimisation and will deliver a programme that minimises lifetime costs.

Sometimes managers wish to use a constraint that the condition state must not exceed some threshold value, thereby ensuring that no bridge element will deteriorate beyond a certain level. The effect of this constraint on the tree diagram is that when at any time the condition state at the next inspection is predicted to exceed the threshold value, the 'no maintenance' branch emanating from the node is eliminated. This ensures that some maintenance is carried out, keeping the condition state below the threshold. The important aspect of this optimisation procedure is that the rate of deterioration forms an integral part of the procedure and, in particular, the effect of maintenance treatments on the rate of deterioration is fully taken into account.

The optimised maintenance programme represents an idealised situation; in some cases, it is necessary to defer or bring

forward maintenance work for a variety of reasons, such as to maintain the flow of traffic through the network. A frequent reason for having to modify the optimal programme is shortage of money; the budget is rarely large enough to carry out all the maintenance work specified in the optimal programme. In these circumstances, it is necessary to prioritise the maintenance programme (Darby *et al.*, 1996). A general principle for prioritisation is that the difference in the lifetime costs of the optimal and prioritised maintenance programmes is minimised. A prioritised maintenance programme is, by definition, suboptimal. Deferring optimal maintenance work is likely to result in more complex, extensive and costly maintenance in the future, even when discounting is taken into account. Furthermore, if traffic disruption occurs, delaying maintenance will lead to more disruption, owing to the growth in traffic with time. The next section describes a prioritisation procedure.

Prioritisation

The procedure uses cost-benefit analysis to minimise the difference in lifetime cost between the prioritised and optimised maintenance programmes. All maintenance work has a cost and a benefit. The prioritisation algorithm investigates the consequences of delaying maintenance work specified in the optimal programme. Thus, in a particular year, if a maintenance job specified in the optimal programme is not carried out there would be a benefit and cost. The benefit would be the money saved that year by not doing the work. The cost would be the increase in lifetime cost that would occur if the work is not done that year compared with the lifetime cost of the optimal programme.

For this exercise, costs are discounted back to the year when the prioritisation is carried out. In practice, the optimisation algorithm is run twice, in both cases starting at the year when the prioritisation is required, but in one case maintenance work is allowed that year whereas in the other case it is not permitted. This procedure is carried out for every element that was scheduled for maintenance on the basis of the original optimised maintenance programme and the elements ranked in order of cost/benefit ratio. The elements with the lowest ratio have the highest priority for maintenance because the costs of deferring maintenance are relatively low compared with the benefits. The prioritised maintenance programme for the year is then obtained by maintaining bridges in order of increasing cost/benefit ratio until the budget is consumed. The bridges that are not maintained will have their deterioration calculated on the basis that no maintenance was carried out; they will be considered for maintenance again on the next occasion that the optimised programme specifies. This will not necessarily be in the next year because the deferral of optimal maintenance the previous year could have resulted in a condition state transition that would necessitate more complex and expensive maintenance action. In these circumstances, it is quite likely that the next optimal maintenance action could be some years in the future.

In the unlikely event that essential maintenance work specified in the optimal programme has too low a priority for the work to be done, appropriate traffic restrictions must be imposed. The costs of such restrictions should be included in the cost–benefit analysis; therefore, traffic delay costs should be included when the prioritisation algorithm is used.

The effectiveness of a maintenance strategy

It is often asked whether the money spent on bridge maintenance provides good value. This depends on how closely the objective of the maintenance strategy is satisfied. The objective of the optimised maintenance strategy is to maintain serviceability of the bridges in the stock during their design life at a minimum lifetime cost. The optimised maintenance programme produced by the algorithms described earlier in this chapter will fully satisfy this objective because the mathematics are so defined. This is an ideal situation; when budgets are limited and maintenance work needs to be prioritised, a different sort of objective is appropriate. The prioritised maintenance programme minimises the difference in lifetime costs between the prioritised and optimised programmes. This gives assurance that good value is being obtained from the money actually spent on maintenance, but it does not indicate the consequences of the reduced level of maintenance. To investigate the effect that the maintenance expenditure has on the condition of the bridge stock, it is necessary to set objectives, such as

- the average condition of the stock in 20 years' time will be in the range 2 to 3
- the proportion of bridges requiring essential maintenance during the next 20 years will not exceed 0.5%.

One of the advantages of the algorithms described earlier in this chapter is that they can be used in a hypothetical or 'what if' mode to investigate how closely the prioritised maintenance programmes for a range of budget values satisfy objectives of the type mentioned. The prioritisation algorithm ensures that any money spent on bridge maintenance achieves good value, while an investigation of the consequences of a specific prioritised maintenance programme shows how close the programme gets to satisfying the condition target for the stock. The three graphs in Figure 23.7 show different ways of monitoring the effectiveness of prioritised maintenance programmes against practical objectives. In each case, budget B2 satisfies the objective most closely. Budget B1 is clearly too small to meet the objective in each case, whereas, although budget B3 satisfies the objective, it is larger than necessary.

This section concludes the discussion of the main algorithms employed in network-level bridge management. The next section briefly describes some other techniques used in the management of infrastructure.

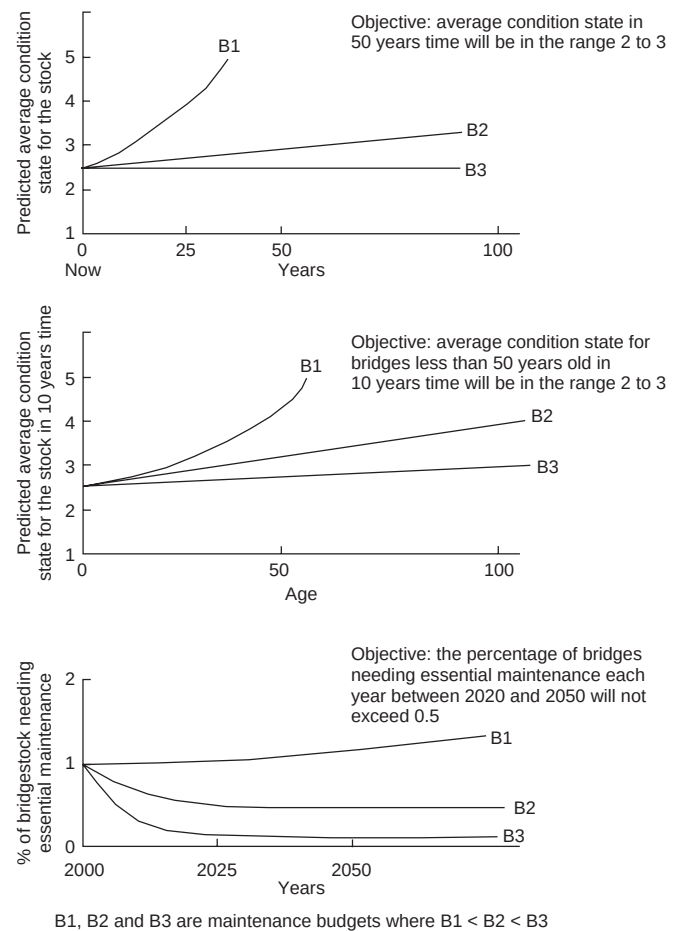


Figure 23.7 Varying the size of the maintenance budget to satisfy objectives

Maintenance affordability, backlogs and long-term plans

The need for cost-effective maintenance strategies for highway bridges is becoming increasingly important as these structures age and the demands for funding expand. Shortfalls in the annual budget to complete all the identified work also require that rational procedures be formulated regarding when maintenance work on different structures within a stock should take place.

By definition, an optimal maintenance programme should minimise lifetime maintenance costs based on the information available at any given time. In practice, however, the optimal programme often proves to be unaffordable, owing to competing demands for infrastructure expenditure. The problem of affordability arises because usually when attempts are made to implement an optimal maintenance programme, a relatively high proportion of the bridge stock is already in a condition that requires immediate maintenance work. In many cases, the importance of maintenance work has not been properly appreciated or funded prior to trying to implement optimal

maintenance. The result is that a backlog of maintenance develops, so the condition of the bridge stock for its mean age is poorer than it would have been had an optimal maintenance strategy been applied to the stock from the time of construction. In these circumstances, the optimal maintenance programme will usually recommend quite a lot of work at first to improve the condition of bridges in order to delay the need for essential maintenance and to do the essential maintenance that is already required. Thus, when an optimal maintenance programme is initially adopted, the cost in the first few years tends to be high. In an ideal world, with a stock of bridges all built at the same time, an optimal maintenance strategy applied from new would be affordable, with a rational funding policy. In practice, this situation does not arise; consequently, optimal programmes are often not affordable. In these circumstances, bridge stock managers frequently have to prioritise maintenance work.

The purpose of steady-state maintenance is to reduce the rate of deterioration, thereby increasing the age of a structure when it will eventually require essential maintenance, and in some cases avoiding this altogether.

Ideally, the maintenance budget should be subdivided between routine, steady-state and essential maintenance in a ratio that does not vary with time, as shown in Figure 23.8.

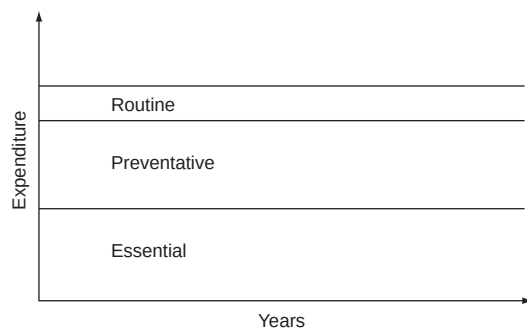


Figure 23.8 Ideal bridge maintenance programme

The constancy of this ratio can only be achieved if the budget is sufficient. If the budget is inadequate, the rate of deterioration will increase and before very long the numbers of bridges requiring essential maintenance each year will increase and consume the entire budget (Das, 1999a). The absence of routine and steady-state maintenance will further increase the rate of

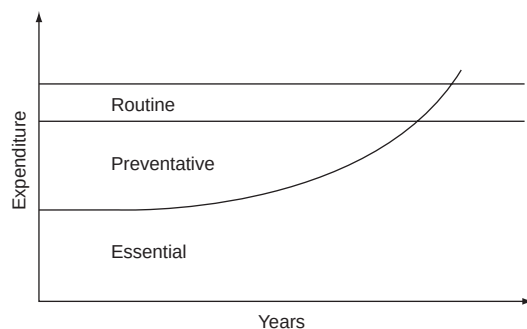


Figure 23.9 Effect of long-term underfunding

deterioration, resulting in a vicious circle effect. Eventually, the budget will not even be sufficient to fund all the required essential maintenance (Figure 23.9) and the number of bridges with traffic restrictions will rapidly increase.

The consequences of underfunding have an immediate effect on bridges needing essential maintenance, since any delay in carrying out the work will necessitate traffic restrictions, such as lane or weight restrictions. This will lead to substantially increased traffic delay costs, as shown in Figure 23.10 for a range of budget deficits (Wallbank *et al.*, 1999).

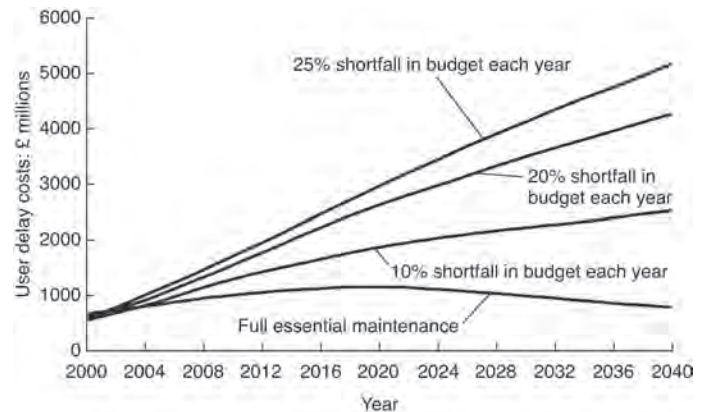


Figure 23.10 Traffic delay costs due to underfunding

The consequence of a maintenance budget deficit is the generation of a maintenance backlog of bridges, the size of which will depend on the degree and duration of underfunding (Table 23.6). To eliminate the backlog and return to stable management conditions requires

- a substantially increased budget
- a long-term plan
- prioritisation of the order of maintenance within the long-term plan.

Year	Bridges needing maintenance ^a	Bridges actually maintained ^b	No of bridges in backlog
1	100	20	80
2	180	20	160
3	260	20	240
4	340	20	320

^aThe number of additional bridges needing maintenance each year is 100.

^bThe budget is sufficient to maintain 20 bridges per year.

Table 23.6 Underfunding leading to a maintenance backlog, an example

Surprisingly, perhaps, long-term underfunding of maintenance has occurred in many organisations responsible for infrastructure management at both local and national level, in both developed and developing countries. In nearly all cases, the underfunding is not intentional. The insidious effect of deferring steady-state maintenance, whereby the immediate

consequences are trivial but grow progressively, often producing an unstable management situation within a decade, probably explains how these situations occur.

In Table 23.6, the budget for maintenance is expressed, for simplicity, in terms of the average number of bridges that can be maintained each year. The other pieces of information required are the number of bridges initially in the backlog and the average number of additional bridges that will require maintenance each year. The table shows how quickly the backlog can grow.

The effect of a long-term plan with an increased budget on the size of the backlog (Vassie and Arya, 2006) is shown in Table 23.7. It can be seen that the backlog is eliminated within 10 years and that the maintenance budget can subsequently be reduced. The duration of the long-term plan should not exceed 20 years and ideally should be about 10 years. If the duration exceeds 20 years, the number of additional bridges needing maintenance each year will start to increase, negating the assumption that this number is constant over the duration of the long-term plan.

Year	Bridges needing maintenance ^a	Bridges actually maintained ^b	No of bridges in backlog
0	—	—	500
1	100	150	450
2	100	150	400
⋮	⋮	⋮	⋮
9	100	150	50
10	100	150	0
11	100	100	0

^aThe number of additional bridges needing maintenance each year is 100.

^bThe budget is sufficient to maintain 150 bridges per year.

Table 23.7 Eliminating a maintenance backlog: example

The relationship between

- initial size of the backlog at the start of the plan, C
- the required annual budget, B (in terms of the average number of bridges for which there is maintenance funding)
- the duration of the long-term plan, N
- the average number of additional bridges needing maintenance each year, A
- is represented as

$$B = (C/N) - A \quad (23.12)$$

This equation can be used to estimate the annual budget needed to eliminate the backlog within the duration of the long-term plan. The backlog will only diminish if the budget is more than sufficient to maintain the additional average number of bridges needing maintenance each year.

It can be seen from the example in Tables 23.6 and 23.7 that in order to eliminate the backlog within 10 years it was neces-

sary to increase the budget by 650%. This is a major increase and may not be affordable. A somewhat lower budget could be adopted if the duration of the plan were increased, although it must be appreciated that a budget increase of 400% would be needed just to keep the backlog constant. Of course, this is only an illustrative example, designed to show the dangers of allowing a maintenance backlog to develop. In practice, the difference between the number of bridges joining and leaving the backlog each year, prior to the installation of the long-term plan, may be smaller than in this example, with the budget increase needed being correspondingly smaller.

When it has been decided to eliminate the backlog using a long-term plan, the next step is to decide which bridges are to be maintained during each year of the plan. This stage of the plan will need to be reviewed each year, as additional bridges will require maintenance each year. This decision will require a prioritisation scheme to rank all the bridges needing maintenance.

Other techniques used in the management of bridges

The previous sections have described, in some detail, deterioration modelling, prediction of condition and the optimisation and prioritisation of maintenance. A number of other techniques are sometimes used:

- whole-life costing
- probabilistic modelling
- risk analysis
- sustainability assessment.

These techniques are described briefly in the following subsections.

Whole-life costing

Until the last decade, most infrastructure investment decisions were based on the lowest initial cost. Structures like bridges, which are expected to have a long life, inevitably require some maintenance in order to achieve continuing serviceability. It is, therefore, considered to be more rational to base expenditure decisions on estimates of lifetime cost instead of initial costs. The most important feature of whole-life costing (Vassie and Rubakantha, 1996) is the estimation of future costs. The main purpose of whole-life costing is to appraise the economics of a number of possible options, to aid decision-making. The type of option is unrestricted but the whole-life costing of bridges has usually been adopted to compare construction options for improving durability or to compare alternative maintenance methods. Since the objective is to appraise the economics of a range of options rather than determine the actual expenditure needed over the life of a bridge, the process of estimating future costs is simplified by neglecting the effects of inflation.

The process of whole-life costing consists of a number of key steps:

- list the types of event leading to expenditure over the life of the structures
- estimate the cost of each of these events at today's prices
- establish whether the cost of each event will vary significantly with time over and above the effects of inflation
- estimate the age of the bridge when these events are likely to occur.

The types of event leading to expenditure over the life of a bridge could include:

- initial cost of design, land and construction
- preventative maintenance work
- repairs
- replacement of components, such as expansion joints, parapets, bearings and waterproofing membranes
- strengthening
- replacement of elements, such as decks or piers.

Some of these events may result in disruption to users of the bridge. The value of this disruption can be calculated using the computer program QUADRO (DoT, 1982). These costs can be much higher than the maintenance costs for a bridge on a busy road. Their value is sensitive to the flow rate of vehicles; since vehicle flows have been increasing progressively over the last few decades, this provides an example of a cost that is increasing with time.

There is bound to be some uncertainty associated with some of these steps because decisions about what may happen in the future are involved. Whole-life costing models can be deterministic or probabilistic. Where considerable uncertainty about the values of the input variables to the model exist, it is better to use a probabilistic model, where instead of inputting a single value for a variable, such as the mean, a probability distribution is used as the input for the variable (see the next subsection for further details).

Whole-life costing is an economic analysis involving expenditure at different times. Economists consider that expenditure that can be deferred to a later date saves money. The idea is that the money could be invested during the period of deferral and therefore has an increased value when the expenditure is eventually made. This process is called discounted cash flow, but for maintenance work account must also be taken of any increased quantity of maintenance required at the later date, owing to continuing deterioration. The factor that defines the time value of money and hence the relative costs of work carried out at different times is called the discount rate (Spackman, 1991). The discount rate depends not only on investment interest rates, but also on the risk of a bridge not achieving its design life. This risk is evidently higher for bridges with a long required life than for a car tyre, for example, which has a relatively short

life and value. Occasionally, bridges have not reached their design life before demolition for such reasons as redundancy, road realignment, rapid deterioration or increased loading. Discount rates are also affected by commercial factors, such as the rate of return required on the initial investment; a high rate of return is associated with a high discount rate. In the UK, discount rates for infrastructure such as bridges are currently around 3% but ranged between 6% and 8% per annum during the last decade. In other countries, discount rates ranging from 2% to 20% have been used.

A discount rate of 6% per annum implies that costs incurred after an age of 40 years have a value of less than 10% of the value at age zero and are therefore often neglected because of the small contribution they make to the whole-life cost. This simplification is not always justified, particularly where the base cost is high or is increasing with time. Nevertheless, it is generally true to say that high discount rates do not favour options designed to improve durability and minimise future maintenance needs. This has resulted in much controversy; some people consider that infrastructure with a long design life should have a correspondingly low discount rate.

The base year for discounting is the year when the investment decisions are made. For durability measures introduced at the time of construction, the base year would be the year of construction, whereas for maintenance options the base year would be the year when the maintenance work was carried out. The formula for calculating the net present value (NPV) of maintenance work carried out in the future is

$$NPV = C_0(1 + 0.01i)^{-n} \quad (23.13)$$

where C_0 is the cost of the work in the base year, i is the discount rate expressed as a percentage and n is the year in which work is carried out, relative to the base year.

The whole-life cost (WLC) consists of the summation of all the NPVs over the design life of the bridge

$$WLC = \sum_{n=0}^{n=120} (NPV)_n \quad (23.14)$$

where $(NPV)_n$ is the net present value of any expenditure made in year n .

The best way to incorporate WLC principles into the tendering process (Arya and Vassie, 2004) is to provide a transparent method of comparing the competing options. This normally involves comparing bills of quantity, although in recent years their application in costing projects has been in decline. The cost of construction of individual elements can be easily estimated if the items in bills of quantities are listed by element type rather than type of work. In addition to returning priced bills, the contractor would be encouraged to vary the design of elements or components in order to reduce whole-life costs. Bills of quantities would also be required for all proposed alternatives. Future maintenance work on particular elements or components of the structure would have to be identified and costed in order to estimate the whole-life costs of the designs.

Tenders could be awarded on a combination of the initial and lifetime costs of the structure, depending on the client's financial requirements. Presenting the information in this way

would allow the key assumptions to be compared between designs and allow initial and whole-life costs to be compared.

Apart from achieving the major goal of reduced whole-life costs, the introduction of such a procedure would offer a number of other benefits, including

- providing a mechanism for introducing new design concepts, new materials and new construction systems
- encouraging all those involved in the construction industry to contribute ideas and expertise in design and construction
- encouraging or rewarding companies investing in research and development.

Probabilistic methods

It is known that, among a set of nominally similar bridges, the rate of deterioration and the need for essential maintenance in a particular year, for example, vary markedly from bridge to bridge. This variability results from a limited knowledge of the complexities of the deterioration processes occurring in bridges. The uncertainty about the precise value of a variable means that it is often better to represent a variable by a probability distribution (Sundararajan, 1995) rather than a single value, such as the mean or a combination of mean and variance. The use of a mean value is clearly unsatisfactory, since it provides no indication of the dispersion of values for individual bridges. The use of the mean and variance, although it provides a measure of dispersion, also assumes a Gaussian distribution. A probability distribution that accurately describes the dispersion of individual results is the best option, but necessitates a considerable body of data. Simple distributions based on limited data can also be useful where great accuracy is not needed.

Probabilistic models have been used to predict whole-life costs, the number of bridges requiring essential maintenance each year and the cross-section of steel reinforcement remaining at different ages, when corrosion is occurring.

For example, triangular probability distributions have been used to estimate the number of bridges in a stock that will need essential maintenance each year. Only three pieces of information are required to set up a triangular distribution:

- the lowest age at which any bridge in the stock needs essential maintenance
- the highest age at which any bridge in the stock needs essential maintenance
- the mode of the distribution, that is, the age at which the need for essential maintenance is most likely.

Such approximate distributions will clearly only provide approximate results, but for broad distributions, where the result is not particularly sensitive to the shape of the distribution, the results can be adequate.

Probability distributions enable simulation techniques to be used, of which Monte Carlo simulation is the best known.

Simulation techniques use a large number (typically about 1000) of 'what if' questions and express the answers as a probability distribution. A single 'what if' question could be, for example, 'What is the net present value of a particular maintenance action if it is carried out at age t years?' A distribution of ages when the maintenance action is needed can be randomly sampled to give an age when maintenance is needed. The usual procedure is to generate random numbers in the range 0 to 1, the same as the probability range, and then to read off the age corresponding to the random number from the cumulative version of the probability distribution. This age is then used to calculate one value of NPV. This procedure is repeated until the distribution of NPVs converges to within user-specified limits. This sampling procedure ensures that the proportion of samples of a particular age corresponds to the value of the probability that maintenance will be needed at that age. Thus, if the probability of maintenance at a particular age is low, few cases corresponding to this age will be selected, and vice versa.

Risk analysis

The risk (Shetty *et al.*, 1996) of an event occurring is generally defined as some combination, often the product, of the probability that the event will occur during a given time period and the consequences of it occurring. The probability of occurrence, although often difficult to calculate, has a clear meaning. The consequences of occurrence are, conversely, open-ended and less well defined. For example, the consequences of a bridge parapet failing as a result of vehicle impact could include injury to people in the impacting vehicle, injury to people underneath the bridge, damage to the bridge, damage to the features under the bridge, such as a road or railway, damage to the impacting vehicle and vehicles under the bridge and closure or partial closure of the bridge and the service underneath while repairs are effected. Consequences can be expressed as a monetary equivalent or on an arbitrary scale. It is not normally possible to make any more than a fairly rough estimate of the probability and consequences of an event occurring, since these depend on many variables of uncertain value and interdependence.

The main advantage of carrying out a risk analysis is to ensure that the factors influencing the probability and the potential consequences are properly considered. The analysis should identify the factors to which the probability of the event occurring is most sensitive and the circumstances in which the potential consequences are most severe. This information can then be used to introduce modifications to reduce risks that are considered to be too high.

There are many ways, of varying thoroughness, of carrying out a risk analysis. The simplest type of risk analysis consists of a somewhat cursory qualitative assessment of some of the factors affecting probability and some of the consequences. This type is only occasionally adequate. At the other extreme, a full quantitative model, calculating the probability and consequences of the event occurring, is required. The events that engineers want to analyse are usually complex and the factors

that affect the probability and their inter-relationships are poorly understood. Even when a reasonable probability model can be developed, there is usually an insufficient availability of data to apply it. A full risk analysis is, therefore, often not feasible. A semi-quantitative approach has been used with some success for a number of events. This approach uses a points scoring system for the probability and consequences and consists of a number of main steps.

1. Set up a group of engineers with experience relating to the particular event.
2. Get the group to list the factors affecting the probability and the potential consequences.
3. Get the group to rank the factors affecting probability and the consequences in order of importance; then allocate a weighting to each factor and consequence, by consensus.
4. Set up criteria for each factor and consequence based on quantitative measures to provide an indicator of their importance in a particular situation; each criterion is associated with a points score.

These steps set up the risk analysis system. Step 3 provides an assessment of the importance each factor and consequence in general, while step 4 is an assessment of the importance of each factor and consequence in relation to a particular case. Consider the example in Table 23.8, which considers only two factors and two consequences, for reasons of simplicity and presentation. In practice, many more factors and consequences would be considered.

In the simplified example in Table 23.8 for a case where the membrane is 10 years old, 10% of tests indicated a defective condition, the road is heavily salted and the cover depth is 40 mm

$$\begin{aligned}
 \text{total probability score} &= 4 \\
 \text{normalised probability score} &= 0.5 \\
 \text{total consequences score} &= 6 \\
 \text{normalised consequence score} &= 0.6 \\
 \text{risk score} &= 30
 \end{aligned}
 \quad (23.15)$$

This example has taken no account of inter-relationships, although this is possible. For example, if the road is not salted, early corrosion would not occur. The risk system could take account of this by introducing the following restriction: 'If a zero point score is awarded for the consequence "salt entering the bridge deck" then a point score of zero must be awarded for the consequence "early corrosion of deck reinforcement".' A detailed example of a semi-quantitative risk analysis is provided in ICE (1998).

Sustainability assessment

Sustainable development is a key policy objective in many countries. Its aim is to promote systems or practices that provide a good balance between the conflicting demands of modern society, namely economic growth, protection of the environment and social progress for all. In the UK, this concept came to the fore in 1999 with the publication of *A Better Quality of Life* (DETR, 1999), which set out the Government's agenda on sustainable development and outlined a framework for implementing this policy in all areas of its responsibility (DETR, 1998a). A supplementary paper published about that time discussed the implications for the construction industry (DETR, 1998b). This document also established the key performance indicators that would be used to monitor progress of the construction sector towards meeting this objective. These performance indicators have been elaborated by a number of other organisations over the years, for example, the Construction Industry and Research Information Association, the Building Research Establishment and the Institution of Civil Engineers.

The construction industry is one of the largest consumers of natural resources and one of the largest polluters (DTI, 2006). The construction process can also have a significant impact on local communities (Considerate Constructors Scheme, 2021). Yet investment decisions on infrastructure projects, such as roads and bridges, continue to be dominated by initial costs, although over the past decade decisions have sometimes been

Failure of bridge deck waterproofing				
Event	Probability factors	Weighting	Criterion	
			0 points	1 point
				2 points
Age of membrane	1	<0 years	5–20 years	>20 years
Condition of membrane	3	<5% defective	5–15% defective	>15% defective
Consequences				
Salt entering concrete deck	1	Road not salted	No of saltings per year <10	No of saltings per year >10
Early corrosion of deck reinforcement	4	Cover depth >50 mm	cover depth 30–50 mm	cover depth <30 mm

To apply the risk analysis, the following steps are carried out for the particular case under consideration.

1. For each probability factor and consequence, a points score is awarded depending on the criterion satisfied.
2. The scores from step 1 are multiplied by the weightings and summed to give a total probability score and a total consequence score.
3. The total scores are normalised by dividing by the maximum possible probability and consequence scores, giving a range of 0 to 1 for each total.
4. The risk is then obtained by multiplication of the normalised probability and consequence scores; further multiplication by 100 gives a risk scale of 0 to 100.

Table 23.8 Simple example of risk analysis system

based on lifetime costs. Lifetime assessments involve aspects of maintenance planning and are therefore of relevance to bridge managers. Lifetime economic appraisal improves sustainability in providing constructed facilities because it takes account of the entire construction process from design to demolition. This economics-based approach does not, however, fully consider sustainability issues because, at present, the cost of materials, energy, transport and waste or emission treatment, among others, only partly accounts for resource or energy reserves and the consequences of the generated waste and emissions. Thus, the use of lifetime costing will go some way towards achieving efficiency gains in economic, environmental and social terms, but progress will be inadequate, particularly with regard to environmental issues, where most commentators now agree that the prevailing world situation is too serious for modest incrementalism to be sufficient. A report by the Sustainable Development Commission (SDC) on the UK Government's progress on sustainable development in the period 1999–2004 found that both the public and private sectors act on issues of social justice and environmental protection, provided that they do not conflict with the primary objective of economic growth (SDC, 2004). In response to this, the Government have vowed to lead by example and ensure that all goods and services procured in the future will be done sustainably (DEFRA, 2005). As a further demonstration of their commitment to this goal, they have stated that future progress on sustainable development will be assessed by the SDC, which will act as an independent watchdog of Government progress. Increased public spending (affordability) is recognised as a barrier to sustainable procurement. Nonetheless, there is a commitment to make sustainable procurement a reality across all departments and agencies. This in turn requires access to tools to assist evaluation of design, construction, operation or maintenance and end of life options, that is, the construction process, on a sustainable whole-life value basis.

It is interesting to consider how government policy with regard to sustainable development could affect the decisions made by bridge managers. At present, decisions are made mainly on the basis of economic factors but in the future more emphasis will have to be given to environmental and social factors. For example, when considering maintenance strategies, the two extreme positions are

- to carry out maintenance as soon as inspections identify the need
- only to carry out maintenance on safety grounds.

In the first instance, the life of the bridge will be maximised and the consumption of resources minimised, but the frequency of maintenance visits to the bridge, many of which will require traffic management and lead to traffic congestion and delays, could be high. In the second case, the life of the bridge could be significantly reduced, although the number of maintenance visits would be minimised.

Taking account of environmental and social factors in addition to economic factors could influence the bridge manager's decision-making in the two cases, as follows.

- In the first case, it would be necessary to balance the benefits of the reduced consumption of natural resources, such as cement, aggregate and steel, against the increased costs associated with frequent maintenance visits and the traffic congestion so caused, resulting in delays and inconvenience to the public, increased wear and tear to vehicles, increased fuel consumption and emission of greenhouse gases and pollutants, such as oxides of nitrogen and sulphur.
- In the second case, it would be necessary to balance the increased consumption of natural resources due to the reduced life of the bridge (e.g. if the life were halved, the consumption would be approximately doubled, depending on the degree of recycling achieved on demolition) against the avoidance of the costs and disadvantages of regular maintenance visits, as described.

It is clear from a consideration of these maintenance strategies that a bridge manager's decision-making process will be much more complex when account has to be taken of sustainability. It will be necessary to have information about environmental and social factors, as well as economic factors over the projected lifetime of a bridge. A procedure for quantitatively assessing sustainability that is reasonably simple and accurate will also be required and indeed some tools are now available to aid bridge engineers in this process (as described next). The extreme examples of maintenance strategy discussed here are chosen to illustrate some of the factors that need to be considered in order to achieve good sustainability. In practice, an intermediate strategy would normally be selected by the manager, but the decision-making process would be similar.

Another situation that often faces bridge managers is the case when a strength assessment shows that a bridge is sub-standard but funding is not immediately available to carry out the necessary strengthening work. In these circumstances, it is often possible to achieve safe passage of traffic by erecting barriers to limit the deck area available to traffic, thereby reducing loading to a safe level. While this is a cheap and easy interim action that keeps the bridge open, it can result in traffic congestion on busy roads. When sustainability is being assessed, it is necessary for the bridge manager to weigh these advantages against the delays and increased vehicle emissions caused. It may be reasonable to keep these interim measures in place for a considerable time on a lightly used bridge, whereas this may not be justified for a busy bridge and there would be a case for funding to be brought forward. In practice, these are complex decisions where it is inadequate to rely solely on engineering judgement. A procedure or tool for assessing lifetime sustainability is needed to help the manager make consistent and good decisions.

A sustainability assessment essentially involves two main steps. The first concerns the separate assessment of all relevant indicators in order to evaluate lifetime economic, environmental and social impacts. The second step involves combining these three impacts to determine the overall sustainability. Early tools developed for assessing the sustainability performance of construction projects include the Civil Engineering Environmental Quality Assessment and Award Scheme (CEEQUAL) and the Environmental Impact Assessment (EIA). An examination of these methods shows, however, that many are, in

fact, heavily biased towards the evaluation of environmental impacts and do not necessarily consider social or economic aspects. Other tools, such as the Building Research Establishment Environmental Assessment Methodology (BREEAM) and Leadership in Energy and Environmental Design (LEED) are primarily aimed at building structures and cannot readily be adapted for bridges.

Fortunately, some tools are available that could be used to appraise the sustainability of bridge works, such as Sustainability Accounting and Sustainable Project Appraisal Routine (SPeAR). An analysis of these methods shows that they are largely qualitative; this does not, in general, appeal to engineers who are used to precision. Moreover, there is no acceptable way of combining information about economic, environmental and social factors to obtain an overall assessment of sustainability (Amiri *et al.*, 2006).

In an effort to resolve these drawbacks, the System for Appraising the Sustainability of Structures (SASS) has been developed by Arya *et al.* (2015). Figure 23.11 shows the set of indicators used to measure sustainability. The impact of some of the indicators, such as climate change, resource use and costs, can be evaluated in purely numerical terms, respectively, tonnes of carbon dioxide, weight of materials and pound sterling. The method of assessment used for other factors, such as heritage or biodiversity and aesthetics is more subjective and involves addressing a series of questions and scoring the responses.

To obtain an overall sustainability, the scores for each indicator must be combined. Unlike some other appraisal tools, such as BREEAM and CEEQUAL, SASS is designed to give a relative measure of sustainability rather than an absolute measure. This is not seen as a drawback of this method, as it is usual for engineers to propose a number of alternative solutions and select the best for the prevailing conditions. The comparison is carried out using a normalisation technique. The normalised scores are dimensionless and can therefore be compared with similar dimensionless scores for the other indicators. The normalised scores are multiplied by weighting factors based on the sustainability theme and the number of indicators per theme. This method also allows weightings to be varied in order to reflect priority themes or indicators for a particular development, for example, economics, aesthetics or climate change.

The work undertaken to develop this tool has shown that changes to the construction and maintenance of a bridge can have far-reaching and diverse effects on its lifetime sustainability (Arya *et al.*, 2010, 2015). It is not intuitively possible to know how such changes affect sustainability; therefore, it is important that a standard method such as the one described here for calculating sustainability is used.

Recent developments in the field of bridge management systems

Bridge management procedures around the world

Bridge inspection practices and bridge management programmes in China, Japan, Korea and the USA have been studied by Jeong *et al.* (2018). They discuss a synthesis report of the Bridge Management Committee of the International Association for Bridge Maintenance and Safety (IABMAS), who surveyed 21 BMSs from 16 countries. This research showed that transportation agencies throughout the world take into account bridge safety and risk, as well as cost models corresponding to different deterioration types (Mirzaei *et al.*, 2012). Improvements in deterioration models using probabilistic methods are used to establish optimal bridge maintenance strategies (Orcesi and Frangopol, 2011). The key findings of Jeong *et al.* (2018) are that the BMSs of most countries have the capability of performing essential functions, such as data storage, deterioration model or cost information, optimisation models and updating functions, but the BMSs of some countries have not yet activated all these features.

Mirzaei and Adey (2017) have determined the most sustainable bridge work programmes using improved structure-level considerations; they show that while some impacts are attributable to elements, others are attributable to the bridge as a whole and therefore require more modelling effort than is currently done in many BMSs. Mirzaei and Adey (2017) introduce a methodology to determine the most sustainable work programmes, systematically accounting for both element- and structural-level impacts, and demonstrate its effectiveness by applying it to an example bridge.

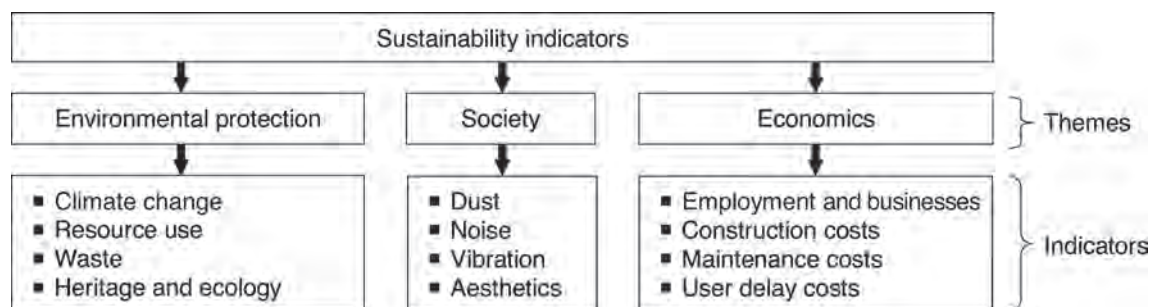


Figure 23.11 SASS sustainability indicators

Sustainable work programmes should be designed to minimise negative economic, societal and environmental impacts. Impacts can be classified by attributing them to one of four stakeholder groups, namely, owner, user and the directly and indirectly affected public (Adey *et al.*, 2012). All these impacts need to be accounted for to achieve a sustainable work programme.

A methodology based on a cost–benefit analysis over a 5 year period for overall bridge condition was proposed by Elbehairy *et al.* (2009), while Huang and Huang (2012) proposed a methodology that determines work programmes by first defining element intervention strategies using predefined ranges of element condition states. The element-level impacts considered in this latter methodology were those incurred by the owner. Structure-level impacts were considered as additional impacts incurred by users, owing to traffic disruption delays, vehicle damage and accidents.

A methodology based on the concept of structure-level performance states to achieve an optimal bridge intervention strategy is described by Fernando *et al.* (2013). In this methodology, impacts are considered to be directly associated with structural performance states, which are determined from the element condition states. Thus, both element- and structural-level impacts are taken into consideration.

Life-cycle costing on many BMSs has only been applied during the bridge operation phase to support decisions about existing bridges. However, life-cycle costing has many useful applications within the entire life of a bridge. This cradle-to-grave approach is used in the Swedish Bridge and Tunnel Management system (BaTMan) described by Safi *et al.* (2013).

Information on the basic components of a BMS is given in the AASHTO report (AASHTO, 2001). The NCHRP report 590 (Patider *et al.*, 2007) describes the development of methodologies for network- and project-level optimisation.

Algorithms for bridge management systems

Ang and Tang (2007) consider the performance of bridges under a number of hazards using a risk assessment procedure in a lifestyle context. They show that the life-cycle assessment of deteriorating highway bridges include aleatory and epistemic uncertainties associated with randomness and inaccuracies in the prediction of reality.

During the last two decades, several studies introduced probabilistic techniques, which can assist the bridge management process (Biondini and Frangopol, 2016; Biondini *et al.*, 2014). Reliability performance indicators deal with uncertainties associated with loads, resistance and modelling methods but do not account for the consequences of bridge failure. Risk-based indicators enable the probability of structural failure to be combined with the consequences associated with failure (Ang and De Leon, 2005; Ellingwood, 2006).

Methodologies that consider sustainability as a performance indicator are discussed by Frangopol *et al.* (2017). The use of sustainability life-cycle performance assessment and management procedures permits the integration of economics, social and environmental aspects. A sustainability performance metric has been established using multi-attribute utility theory, which allows several risks to be considered, while taking account of the risk attitude of the decision maker (Jiménez *et al.*, 2003).

Several deterministic and a few probabilistic studies have analysed the resilience of individual bridges and bridge networks. Resilience, as defined by the Presidential Policy Directive (PPD, 2013), is a structural performance indicator that accounts for structural performance together with recovery patterns under hazards (Bruneau *et al.*, 2003).

Risk and sustainability management of bridges under the effect of sudden and gradual deterioration have been investigated by Frangopol *et al.* (2017), who addressed quantifying risks, life-cycle performance and sustainability of bridges at the component and network levels. The effect of climate change on probabilistic performance was also examined. Bridge management planning and optimisation under a restrictive budget and performance constraints are achieved using a probabilistic management framework. This approach (Frangopol *et al.*, 2017) provides optimal intervention strategies for the decision maker, allowing risk- and sustainability-informed decisions about individual bridges and bridge networks during their lifetime.

The main effects of climate change on the performance prediction of bridges were also investigated by Frangopol *et al.* (2017). Measurements over the last decade have shown that sea levels, global temperatures, ocean temperatures and atmospheric carbon dioxide concentrations are all rising at an enhanced rate. These trends are likely to continue; therefore, it is important to determine the effects of climate change on the performance and life-cycle assessment of deteriorating bridges.

The Pontis (Thompson *et al.*, 1998) and BRIDGIT (Hawk and Small, 1998) bridge management systems consider discrete condition states and Markovian deterioration modelling. The main element of a Markov chain model is the transition matrix that specifies the probability that the state of a component changes to another state within a specified time period. It should, however, be noted that the condition index is a subjective measure that may not reflect the true load-carrying capacity of members (Saydam *et al.*, 2013). These component-based approaches do not provide information about the interaction between components and the overall performance of the whole structure (Saydam and Frangopol, 2011).

Overall, the key objectives of a probabilistic framework are to determine the likelihood of successful performance from the total expected cost accrued over the entire life-cycle and make optimal risk-, resilience- and sustainability-informed decisions related to design, construction, inspection, maintenance, monitoring, repair and replacement of bridges under a number of objectives and constraints (Frangopol *et al.*, 2017).

Survey of the functionality of BMSs used by bridge owners in the UK

The survey printed in Box 23.1 was sent to local authorities and other bridge owners in the UK. The objective was to obtain a rough idea of the functionality of the BMSs used in the UK. It was hoped that the questions posed were simple enough to be answered quickly by yes or no responses. Completed questionnaires were received from 36 authorities. Table 23.9 gives a summary of the responses for each of the 13 questions.

A survey on bridge management systems in the UK

1. Do you use a computer based bridge management system? If so which one.
2. Does it include an inventory of your bridge stock containing information such as name, location, type of construction, list of elements, material of construction, and components (bearings, joints, etc.)?
3. Do you use a BMS to plan and prioritise your inspection programme?
4. Do you record condition and test results on your BMS?
5. Does it use real-time data?
6. Do you use the information from (4) to estimate the rate of deterioration?
7. Do you use the information from (4) and (5) to plan a programme of repair and maintenance?
8. Do you record the maintenance and repair work carried out?
9. Do you use the information in (7) and subsequent condition measurements to assess the effectiveness of the maintenance repair and prevention work?
10. Do you record the cost of maintenance and repair work on a whole-life cost basis?
11. Do you include traffic delay and traffic management in the cost of maintenance and repair?
12. Do you prioritise your maintenance or repair work on the basis of the condition of each bridge or on the basis of the conditions of all the bridges in the stock?
13. Do you feel your management system provides you with the information required for managing bridges in the format required or do you have to process it to effectively manage your bridge stock?

Box 23.1 Questionnaire

Question	Yes	No	Question	Yes	No	Question	Yes	No
1	35	1	6	15	21	11	16	20
2	35	1	7	25	11	12	16	19
3	23	13	8	26	10	13	13	23
4	31	4	9	12	24			
5	15	21	10	9	27			

Table 23.9 Summary of responses from 36 bridge authorities in the UK to each of the 13 questions in Box 23.1

Commentary on the answers to the questionnaire

1. Only one authority did not use a BMS. Surprisingly, ten different BMSs were being used. The three most commonly used systems were BridgeStation, WDM and Confirm. A few authorities had designed their own systems using Access databases or Excel spreadsheets.
2. All the BMSs in use contained an inventory of the bridge stock, although some needed additional fields to make them comprehensive.
3. Most BMSs planned and prioritised inspection programmes. Some just used the standard interval for general and principal inspections and did not take account of backlogs in the programme or priority for bridges in poor condition.
4. Almost all the BMSs recorded condition scores but only a few recorded test results. These were often recorded on a separate spreadsheet, making them less easy to coordinate with the condition scores.
5. Some authorities used tablets to enter the data obtained on-site to give real-time data. Other authorities completed forms on-site, subsequently entering data to the BMS in the office by the day following the inspection.
6. Some BMSs calculated rates of deterioration from condition assessments made during inspections. However, most BMSs did not have this facility, although estimates of rates of deterioration were made manually outside the BMS.
7. Most authorities used their BMSs to plan maintenance or repair programmes based on condition data recorded during inspections. Other authorities prepared their maintenance programmes manually.
8. Most authorities kept a record of maintenance and repair work completed, either within the BMS or on spreadsheets.
9. Most authorities did not assess the effectiveness of maintenance or repair work on the subsequent rate of deterioration.
10. Most authorities did not yet assess the cost of maintenance on a whole-life cost basis.
11. Most authorities included traffic management costs in the overall maintenance or repair costs within the BMS. Only a few authorities included calculated traffic delay costs in the BMS, although many took account of them in other ways.
12. The numbers were roughly evenly divided between those authorities that prioritised maintenance or repair work based on the condition of an individual bridge and those that prioritised based on the condition of all the bridges in the stock.
13. Most authorities said that their BMSs did not provide them with all the information needed in the correct format and that they needed to carry out additional processing of the data. It was noted that most of the authorities who were content with the output of their BMSs used BridgeStation.

Comments on the practical use of bridge management systems

There is a conception that the people who develop the algorithms for bridge management systems take an overly theoretical approach to a practical problem without themselves having experience of managing large bridge stocks. Some engineers consider that success will only be achieved if BMSs are written by engineers. This would avoid a black-box approach. Engineers would like to know how the BMS algorithms work and how engineering judgement is taken into account. Indeed, some engineers do not want maintenance priorities to be automated by the BMS. They would prefer the BMS to provide information to help them to prioritise using their own engineering judgement. For example, when considering safety aspects of bridges, some engineers prefer to use engineering judgement to make a qualitative assessment rather than a reliability or risk evaluation.

Engineers may find a document recently produced by engineers from the UK Bridges Board (2015) called the *Structures Asset Management Planning Toolkit* (version 2.01) useful for managing their bridges. This document is supported, endorsed and recommended by the Department for Transport, Highway Asset Management Financial Information Group (HAMFIG), Association of Directors of Environment, Economy, Planning and Transport (ADEPT) and Chartered Institute of Public Finance and Accountancy (CIPFA). The purpose of this toolkit is to support bridge engineers and managers in management and other related issues, such as financial planning, prioritisation, life-cycle planning and asset valuation.

Some engineers consider that such factors as the importance of a bridge within a network, results of a structural assessment or consequences of a failure are not amenable to assessment by computer algorithms.

Traffic delay costs are generally not included in BMSs because they tend to be very high compared with actual repair costs, so are difficult to include in whole-life costing procedures.

The prioritisation of repair projects is difficult to achieve using cost-benefit analysis because of difficulties in quantifying the benefits of repair projects. Often, prioritisation is based less on economic factors and more on the number of complaints from the public. For example, the public may complain about a defect that looks bad but may not matter structurally. This may be obvious to the engineer but not to the general public. Factors such as these are difficult to incorporate into BMS algorithms.

Nevertheless, bridge managers require BMSs that allow them to make informed and rational decisions, thereby making better use of limited time and resources.

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Chapter 24

Deterioration, investigation, monitoring and assessment

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With age, aggressive environment and steadily increasing traffic, bridge decks wear out their strength and durability, especially where design, construction or maintenance errors are made. This chapter first analyses the major sources of material and structural degradation: concrete deterioration, steel corrosion, excessive magnitude or repetition of applied stresses or strains, temperature-level fluctuation and non-linear diffusion. The principal types of cracks and weakened areas are shown and commented on. For material and structural appraisal, numerous powerful methods are presented, ranging from simply conventional to highly advanced, selected for each case after a thorough desk study and a detailed visual inspection. The last presented method illustrates the feasibility of permanent monitoring and active control of structural safety, approaching the performance of smart systems. Finally, the residual strength evaluation procedures are outlined for a potentially distressed bridge. They involve a well-prepared semi-probabilistic recalculation process, based on site investigation results and on the concepts of serviceability and ultimate limit states.

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In spite of their relatively simple structural systems and well-defined supports, bridges suffer heavily from the effects of age, climate and, especially, traffic that often exceeds the codified limits. Their strength and durability depend on the type and quality of the constituent materials, structural design, construction and maintenance. This latter factor is the only one that can be influenced for existing structures. Maintenance, however, can be effective only in the light of adequate inspection, monitoring and assessment procedures, facilitating, if need be, an appropriate preventative action (Calgaro and Lacroix, 1997). This chapter will consider the main causes of damage, certain material and structural investigation methods and assessment procedures. Only concrete and steel bridge decks will be discussed.

Main causes of degradation

The deterioration of bridges may have different causes and degrees of gravity, with or without visible signs. It can materialise in unusual cracks, excessive strain or changes in geometry. Other serious types of damage, such as embedded steel corrosion and concrete alkali–silica reaction, become apparent only at an advanced stage. Before presenting investigation methods, the probable causes of the most frequently observed types of degradation will be classified. The classification is not exhaustive.

Deterioration of constituent materials

Concrete

While generally known for its durability, concrete suffers at times the consequences of gas and water infiltration into its capillary system. Physical, physicochemical and chemical mechanisms may deteriorate its properties when external agents react with concrete hydrates to form expansive or soluble compounds. Concrete can also be subjected to destructive internal swelling, owing to chemically incompatible ingredients.

Physical or mechanical agents

Frost–thaw cycles act on the free water content in unprotected areas of concrete bridges. Scaling, swelling and often crack patterns affect the surface and deeper layers with a variable intensity, depending on the porosity and degree of saturation. Road de-icing salts cause additional penetration of chloride ions, hence steel corrosion.

When surfaces of concrete decks and masonry piers are exposed to water flow, they are liable to abrasion and erosion; impacts aggravate the consequences.

Physicochemical agents

If no appropriate precaution is taken, intrinsic concrete shrinkage may favour various forms of cracking, in different

directions, at consecutive formative stages of the material. The first cracks appear within hours from casting, owing to packing down and sedimentation. After shuttering removal, other widely spaced shallow hair cracks may follow, owing to self-desiccation shrinkage; they grow wider and deeper with high hydration heat. At much longer times, drying shrinkage cracks may appear, owing to subsequent free water loss.

Chemical agents

The following reaction products are observed under the electron microscope (EM).

- **Carbonation.** Atmospheric carbon dioxide dissolves in water to form carbonic acid, which reacts with most cement hydrates (Figure 24.1). This reaction, called carbonation, may very slowly move into the unsealed concrete pores, cancel alkalinity and set up a steel corrosion process. In general, it would take many years, and even decades, for carbonation to reach a normally covered reinforcement.
- **Sulphate reaction.** Sulphates in concrete may be of external geological or biological origin, for example, industrial pollution. They also have an internal origin, such as the use of gypseous or pyrite aggregates (Figure 24.2) and of quick hardening heat treatment, producing what is known as delayed ettringite formation (Divet and Randriambololona, 1998). The sulphate destructive action on cement constituents occurs in two steps: sulphates first react with calcium hydroxide to form secondary gypsum. This, in turn, reacts with aluminates to form secondary ettringite (Figure 24.3), a very expansive crystalline substance inducing high internal, hence bursting, pressures, leading to concrete disintegration (Figure 24.4).
- **Marine environment.** Seawater attacks by its sulphates and chlorides. These go through a complex chain reaction with calcium hydroxide in the cement, to finally form the aforementioned expansive ettringite (Figure 24.5). Chloride ions, remaining free after diffusion into the concrete, can reach and corrode the reinforcement.
- **Alkali-silica reaction.** In a concrete mix, the presence of incompatible ingredients can start a complex chemical reaction with destructive mechanical consequences (Hobbs, 1988; Swamy and Al-Asali, 1986). Among the known mechanisms, the most frequent is the alkali-silica reaction (ASR). It takes place between the interstitial alkaline solution, mainly from the cement, and a reactive type of silica particles from the aggregates. Favoured by lime and humidity, an alkali silicate gel is thus formed; it further combines with the calcium hydroxide of the mortar paste to produce a highly water-absorbing, hence swelling, gel. The role of water has been thoroughly investigated (Larive, 1998). The resulting concrete expansion, together with tendon or reinforcement overextension, leads to crack patterns that may attain or exceed a 10 cm depth, with 2–5 cm spacing (Figure 24.6). Observed under the scanning electronic microscope, the ASR products vary from a smooth gel (Figure 24.7) to different groups of crystals of crackling aspect (Figure 24.8). The gel is formed and often stays around the aggregates but may also migrate in the concrete mass and even leak out through surface cracks.

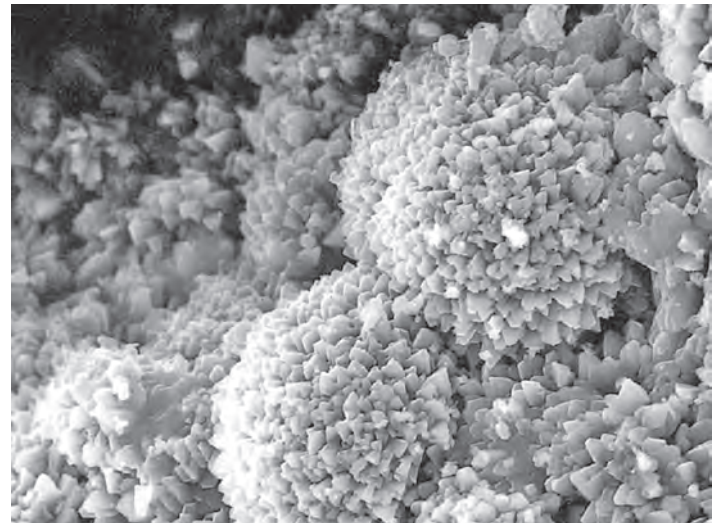


Figure 24.1 Carbonation: calcium carbonate (CaCO_3) crystals (EM) (photograph courtesy of LCPC)

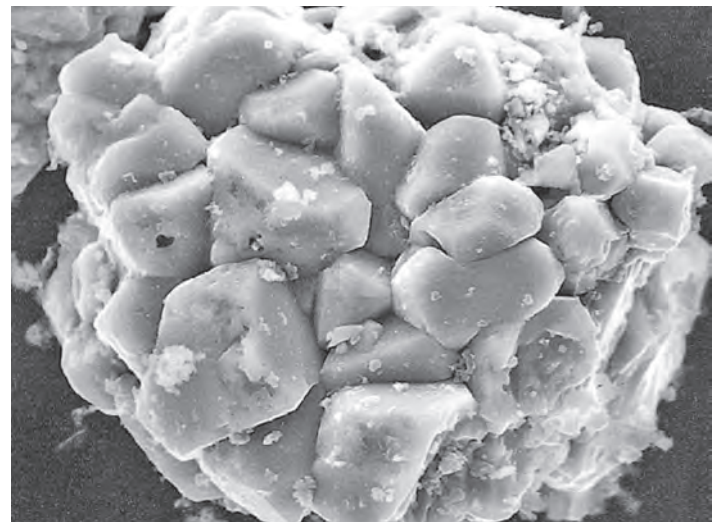


Figure 24.2 Pre-existing pyrites (EM) (photograph courtesy of LCPC)

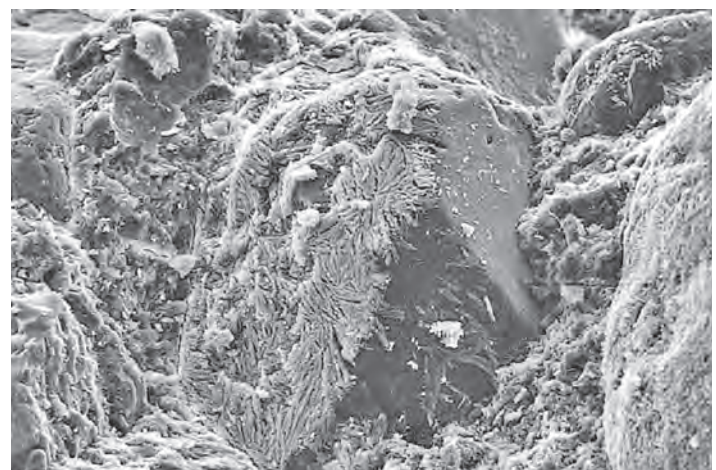


Figure 24.3 Expansive ettringite on aggregate surfaces (EM) (photograph courtesy of LCPC)



Figure 24.4 Destructive sulphate reaction, causing cracking in concrete (photograph courtesy of LCPC)

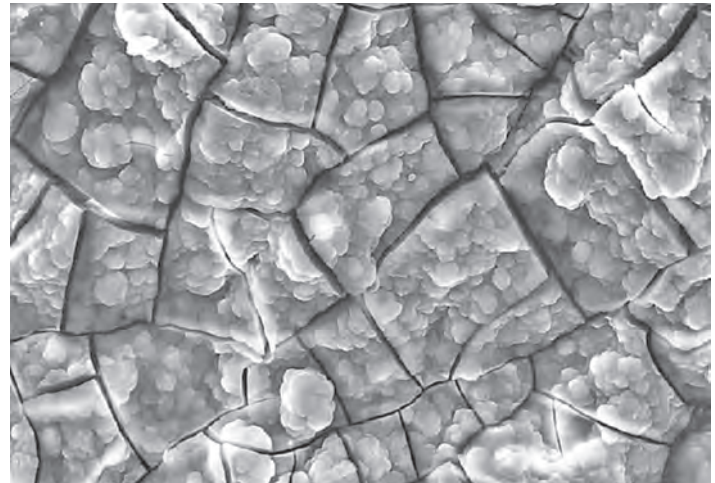


Figure 24.7 Expansive ASR gel (EM) (photograph courtesy of LCPC)

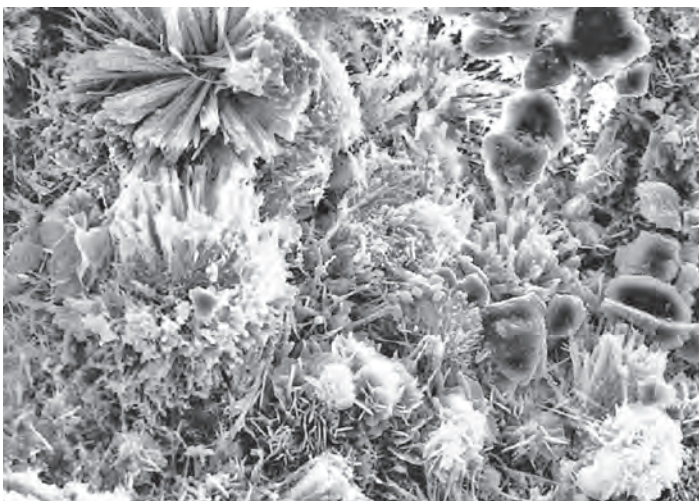


Figure 24.5 Various damaging products of marine attack (EM) (photograph courtesy of LCPC)

Steel

When in contact with atmospheric or chemical agents, in a humid environment, steel can react and be dissolved. In civil engineering structures, this corrosion mechanism mainly results from an *electrochemical process* in solution. The corrosion intensity depends on the metallurgic and mechanical properties of the steel, as well as on the reagent parameters (Engel and Klingele, 1981; Raharinaivo *et al.*, 1998), namely

- metal composition, treatment, shape, stress, surface state
- reagent nature, concentration, pH, oxygen content, impurities, inhibitors, pressure, and, last but not least, temperature, where a 10°C increase can double the corrosion speed.

Steel rusts when exposed to humid air (50%–70%) and to dust or other deposits, especially sulphates. This atmospheric corrosion accelerates with certain polluting gases. In marine

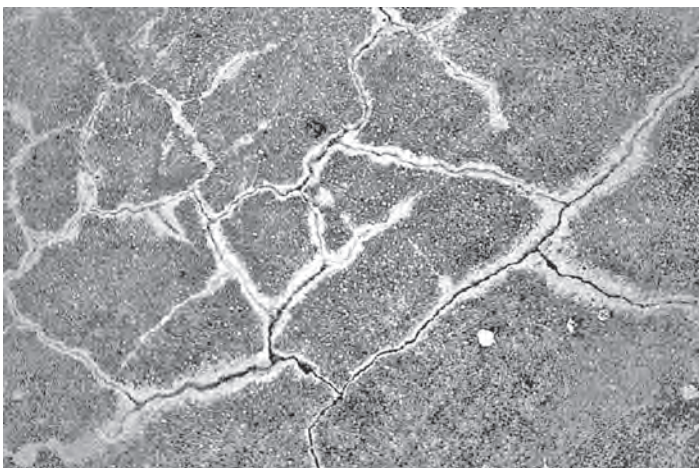


Figure 24.6 Concrete crack pattern formed by alkali-silica reaction (photograph courtesy of LCPC)



Figure 24.8 Expansive ASR crystals (EM) (photograph courtesy of LCPC)

environments, chloride solutions create a thin electrolytic layer, permeable to oxygen, on the steel surface and allow the corrosion process to start.

In reinforced and prestressed concrete, embedded steel corrosion starts when the protective alkaline film is destroyed by carbonation or chloride ions (Figures 24.9 and 24.10). The process advances easily where the concrete cover lacks thickness and density; it also intensifies with available oxygen and humidity, especially when the latter alternates with dryness. The resulting rust reduces the effective steel sections, their ductility and fatigue resistance. Through its expansion, to about six times the reacting steel volume, rust also bursts the surrounding concrete. The inner steel layers may be less affected but the danger resides in their inaccessibility, hence difficult inspection and uncertain assessment, except through methods explained later in this chapter.



Figure 24.9 Corrosion in reinforcement tendon (photograph courtesy of LCPC)

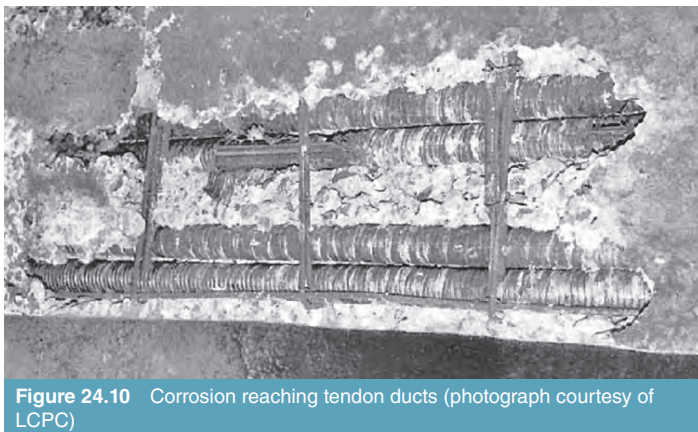


Figure 24.10 Corrosion reaching tendon ducts (photograph courtesy of LCPC)

Prestressing tendons are particularly sensitive to cover and to grout injection quality; they are also vulnerable to other corrosion forms involving cracks, hydrogen embrittlement or fretting fatigue mechanisms, described next.

In many cases, as already mentioned, steel corrosion in concrete takes place without external signs or defects. However, where waterproofing is defective, water leakage may bring rust traces or efflorescence to the surface.

In suspension and cable-stayed bridges, cables may deteriorate and fail, either by section reduction or by cracking of constitutive wires. Each type of damage can result from both electrochemical and mechanical processes, respectively leading to corrosion and *fretting*.

Wires corrode in two ways (Figure 24.11). *Pitting corrosion* occurs mainly at water retention points, such as the lower parts of suspension cables, deviation saddles, anchorage zones and most other connections. *Stress cracking corrosion* involves a reaction between surface oxides under tension and the steel, producing nascent hydrogen, hence metal embrittlement.

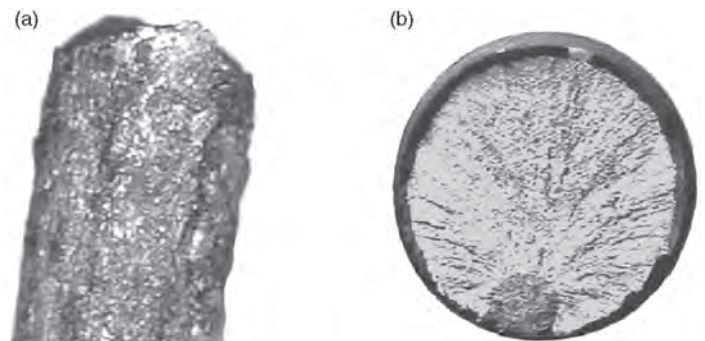


Figure 24.11 Two types of wire corrosion: (a) pitting; (b) stress cracking (photograph courtesy of LCPC)

Small relative tangential displacements, between parallel touching wires of curved cables and tendons, can generate periodic, locally high frictional forces (Figure 24.12). This mechanism, called *fretting*, wears away or cracks the contact surfaces in the long term. The damage worsens with the continued presence of metal particles stemming from the process. Stress variation owing to oscillatory friction can also cause fatigue cracks and lead to failure (Brevet and Siegert, 1996; Siegert, 1997).



Figure 24.12 Fretting on contact surfaces leading to fretting fatigue failure in tendon wires (photograph courtesy of LCPC)

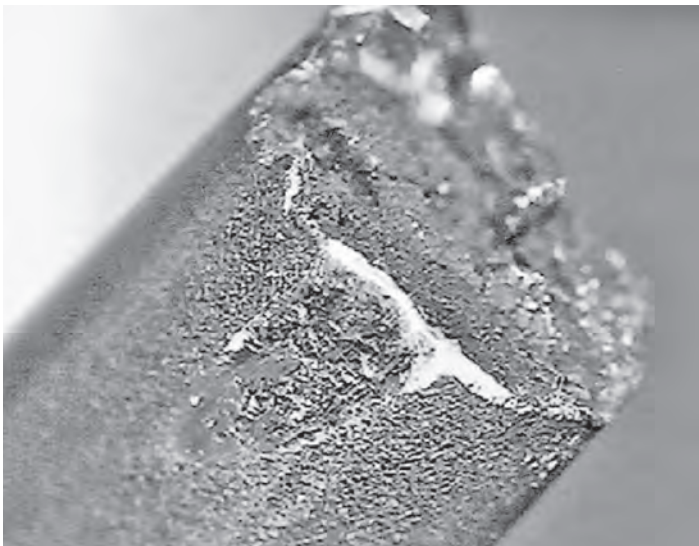


Figure 24.12 Continued

Finally, under excessive and especially concentrated stress, high carbon steel with impurities suffers brittle failure, while mild steel has a better-adapted ductile behaviour (Figure 24.13).

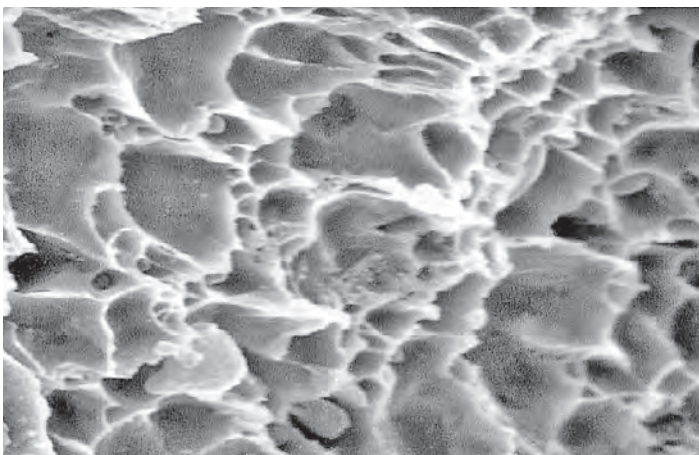
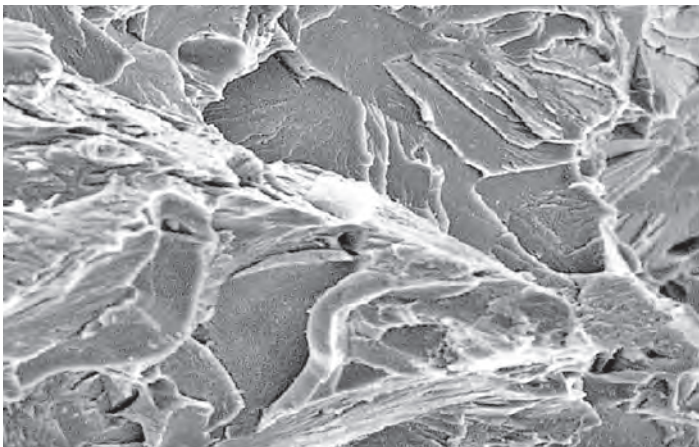


Figure 24.13 Failure of tendon wires: (top) brittle; (bottom) ductile (EM) (photograph courtesy of LCPC)

Degradation through applied forces

From a mechanical viewpoint, a structure is damaged when its static or dynamic equilibrium is disturbed. Failure occurs where the magnitude or repetition of the applied stresses or strains exceeds the allowable limits of the material.

Dead loads and live loads

Surprisingly, careless mistakes are sometimes made in determining dead loads. Underestimating cubage or density and neglecting equipment weight, for example, reduce the safety factor but are rarely a direct source of damage.

Conversely, during the multiphase construction and subsequent life of a statically indeterminate prestressed concrete bridge, the combined time-dependent effects of concrete creep and steel relaxation have frequently caused serious structural cracks under dead load. These effects are now being accounted for in most design codes. However, given the possible differences between the assumed and actual viscoelastic behaviour, it is still advisable to measure the support reactions immediately after construction, or as early as possible, for future reference.

Through their extreme values, as well as their dynamic and fatigue effects, traffic loads are among the major causes of ageing. National loading codes have been periodically revised to cope, on a sounder scientific basis, with steadily increasing vehicle weights and traffic densities. The evaluation of actual live load configurations is necessary for an existing bridge, regardless of the initial design considerations.

In road bridges, dynamic effects (other than fatigue) are incorporated by introducing impact coefficients that depend on structural, surface, vehicle and velocity parameters. These coefficients demonstrate higher near-surface discontinuities and may explain, at least partially, the denser crack patterns often observed at the expansion joints of reinforced concrete bridge slabs. Additionally, vibration and longitudinal braking effects may displace and deteriorate inadequately designed bearing systems.

Most railway bridges constructed in the early 1900s using masonry or metal were oversized and thus able to cope with greater vehicle loads and speeds. However, heavier service increased their exposure to fatigue. Bridges of this type are equally subject to

- rivet loosening, owing to impact effects on discontinued rails
- temperature effects of long rails on fixed supports
- resonance and lateral effects at high speed, as well as horizontal forces caused by speed changes.

Thermal effects

On the materials

By its level and fluctuation, temperature acts on material durability (Divet and Randriambololona, 1998). Through its

diffusion, it creates potential strains with local and global effects on the structure (Behr and Trouillet, 1988; Priestley and Blucke, 1979).

Concrete that is cast in very cold weather may deteriorate by the expansion of freezing water used in the mix. This should be set apart from the swelling and surface scaling effects of frost–thaw cycles on hardened concrete. Concrete that is cast in hot weather ($>35^{\circ}\text{C}$) hardens quickly and has reduced plasticity. Accelerated cement hydration also creates higher core–surface temperature gradients, inducing tensile surface stresses on cooling. Extensive cracking may follow.

Another similar effect is caused by a completely different mechanism: high temperature and low atmospheric humidity favour free water loss, creating considerable hygroscopic gradients close to concrete surfaces. The resulting proportional potential strain, better known as drying shrinkage, induces very high tensile surface stresses, exceeding the concrete tensile strength and often stresses from live loads.

Where specific secondary reinforcement is insufficient, the superimposed effects of hydration and hygroscopic mechanisms cause extensive surface cracks, leading to concrete deterioration and steel exposure to corrosion.

In steel, while low temperature increases the yield point and strength, it also decreases ductility in certain types, where brittle failure may occur with practically no plastic strain (Persy and Raharinaivo, 1987). High temperatures may also catalyse certain chemical reactions, causing material degradation.

On the structure

In concrete bridges, heat from the sun diffuses throughout the deck, creating a potential non-linear strain field, ε , partially restricted by plane section conservation to an actual linear distribution ε_m . Over the section height, h , as illustrated in Figure 24.14, two longitudinal mechanical effects result:

- a non-linear self-balanced stress profile with maximum tension at mid-fibres
- a linear average strain distribution.

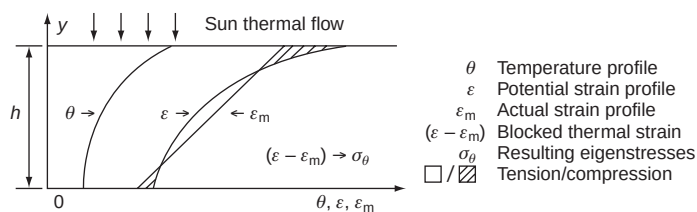


Figure 24.14 Temperature-induced strains and stresses over the height, h , of a concrete bridge deck

The former amplifies the local surface tension, owing to hydration heat and drying shrinkage; the latter affects continuous spans, inducing flexural stresses that, if not accounted for,

may cause serious cracks in the lower fibres at the intermediate supports (Figure 24.15).

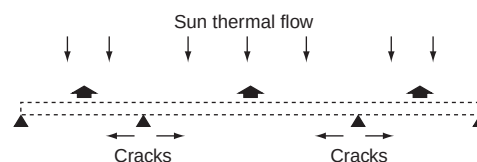


Figure 24.15 Temperature-induced tensile stresses in lower fibres, at intermediate supports

In composite bridges, a deterioration of the connectors has often been observed, owing to the differential coefficient of thermal expansion between steel and concrete.

Miscellaneous destructive actions

The undermining of pier footings by water has been one of the principal causes of bridge failure. With the continual progress in deep foundation technology, this hazard has now been practically eliminated in recent structures.

In seismic regions, a soil acceleration exceeding $0.3g$ may seriously damage concrete bridges with heavy rigid elements, excessive longitudinal steel bars or insufficient transverse reinforcement. Conversely, more satisfactory behaviour has been noticed in square, multispan reinforced and prestressed concrete slab bridges, as well as in prestressed beams on neoprene supports.

Ship and vehicle impact is less frequent but more serious on bridge decks than on piers, especially on light and slender structures (Calgaro, 1991).

Fire deteriorates concrete at 200°C , prestressing tendons at 175°C and high-strength steel bars at 350°C – 450°C (Persy and Deloye, 1986).

Design errors

Design is mainly for material durability and structural adequacy. The degradation of bridges can result from mistakes made at different design stages, such as

- inadequate theoretical models for the prediction of applied forces and the assumption of local and mechanical behaviour, especially fatigue
- insufficient anti-corrosion precautions
- incorrect or inappropriate choices of the shape and arrangement of structural elements with regard to other durability aspects and access for maintenance.

Some specific design errors, linked with the chosen constituent materials and structural system, are considered next (Calgaro and Lacroix, 1997).

Reinforced and prestressed concrete bridge decks

In all concrete decks, extreme care must be taken to check for the shear capacity at the supports and for adequate top bar anchorage in cantilevers; otherwise, there will be a risk of severe sudden failure. Poorly designed waterproofing or drainage and a lack of compact concrete cover may seriously shorten the service life of a bridge.

- In ordinary reinforced concrete bridge decks, careless errors in structural design have a wide range of consequences. Generally, damage may arise from neglected thermal stresses, unbalanced outward thrust of steel bars, secondary reinforcement inadequacy, missing transverse ties of main steel layers, short anchor or splicing bar lengths and bearings too close to deck boundaries. Skew slab bridges may crack in their acute angle area if the required reinforcement density and direction are not observed. In continuous spans, wide laterally cantilevered slabs also crack at intermediate supports if longitudinal distribution reinforcement is inadequate.
- In post-tensioned bridges, insufficient anti-corrosion precautions constitute basic durability design errors, namely, the *absence* of the following: tendon anchorage seals; watertight non-corrosive metal ducts; efficient deck waterproofing and drainage, including at expansion joints and sufficient concrete cover, preventing the degradation chain of reinforcement corrosion–concrete spalling–tendon exposure. Deck anchorages and a larger number of construction joints add to corrosion risks.

In structural design, the following errors have led to flexure cracks in box girders built by cantilever-balanced segments: discarded thermal gradient effects, underestimated prestress frictional losses or stress redistribution owing to creep and shrinkage and simultaneous stopping of several slab-anchored continuity tendons. Segment joints may also open when tendon couplings are improperly distributed or lack efficient de-bonding devices and complementary reinforcement.

Near the supports, shear cracks result from underdesigned active stirrups or an overestimated ability of longitudinal prestressing to reduce shear. Flexural and shearing inadequacies often combine to cause an intermediate form of cracks, as illustrated in Figure 24.16.

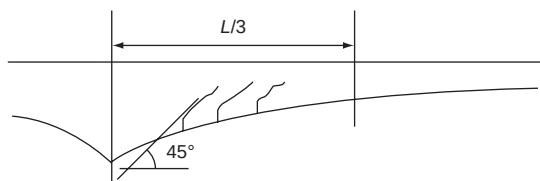


Figure 24.16 Shear cracks combined with flexural deficit in a box girder bridge

Locally, underdesigned reinforcement around tendon anchorages causes diffusion or transfer cracks (Figure 24.17). In front of web-embedded anchorages, diffusion cracks retrace the tendons on the surface; although mechanically tolerated, they favour the infiltration of water, which expands on freezing

and worsens the case. At the bottom slab anchor blocks, diffusion cracks appear on both sides in a fishbone arrangement and may extend to the webs with 45°-inclination; transfer cracks develop transversally, right behind the blocks.

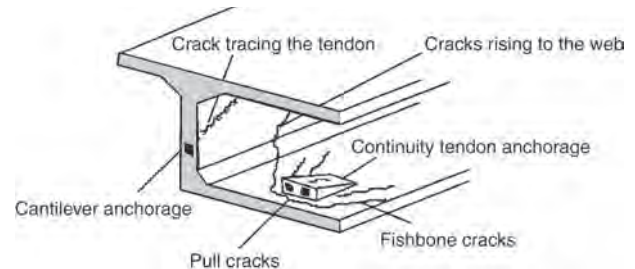


Figure 24.17 Principal crack types in anchorage zones and neighbouring span sections

The consequent discontinuities, mainly flexure cracks and opening joints, often develop a high fatigue risk for the tendons, depending on the number, intensity and variability of load cycles. Bond failure and irreversible deflections may follow, seriously affecting service life. Experience has shown, however, that fatigue damage hardly ever occurs where structural design succeeds in avoiding cracks in concrete sections.

Steel and composite bridge decks

On *steel bridges*, corrosion is by far the main cause of deterioration. This electrochemical process, described previously, results from poor waterproofing or drainage and from other specific errors in the design. Mistakes include inappropriate arrangement and connections of structural elements, favouring water stagnation and condensation. For example, channel sections are often severely corroded when open upwards but usually spared, even without maintenance, when inverted. Widely spaced rivets allow an initial and growing space for humidity and corrosion in between the connected plates. In a wet atmosphere, these plates may also be laminated by corrosion when metals of different electric potential are used at riveted points.

In steel road bridges, fatigue damage is often caused by live loads of intermediate magnitude and frequency. Orthotropic plate bridges are particularly vulnerable when the girder–stiffener welded junctions and the flexible overlying floor plate are not properly designed for fatigue. Examples are found in narrow, initially temporary orthotropic bridges with underdesigned floor plate thickness, thin surfacing and low-quality welding.

In older railway bridges, which were not really designed for fatigue, the steady increase in traffic has led to serious deterioration, especially in short members, such as stringers and floor beams, which directly receive axle loads and are often calculated with unrealistic boundary conditions.

Generally, fatigue damage shows in riveted and welded connections owing to underdesign, stress concentration of different origins and poor workmanship.

Errors in structural design include

- non-concurrent truss bars developing substantial secondary moments at the joints and causing rivet or plate failure
- a sudden change in section profile, where, together with welding shrinkage, the cumulative stress concentration often causes cracks
- longitudinal rivet pull-out at the end angle connections of truss stringer webs, designed as simply supported on the floor beam webs, but constructed as continuous spans.

While the design and execution of repairs are usually simpler in metal than in concrete, several mistakes may have serious consequences, for example

- strengthening by welding without checking the state of the metal base
- failure to apply welding specifications on crack repair
- replacing truss bars without computing the dead load redistribution on adjacent bars
- strengthening parts of an indeterminate structure without evaluating the new stiffnesses and their effects on moment and stress redistribution
- strengthening a riveted assembly by welds, which, besides their risky execution, respond only to live loads and after rivet failure.

In *composite bridges*, durability and structural problems often arise from the inadequate design of slab–girder connections with respect to delayed strains and local forces. Hair cracks first develop in the slab, around the connectors, owing to restrained early-age concrete creep and shrinkage. The gradual growth of these cracks leads to local concrete plasticisation, then to a relative slip between slab and girders, disturbing the required monolithic behaviour of the deck section.

These transverse discontinuities, worsened by negative moments at intermediate supports or diaphragms, allow water infiltration and accelerate material degradation, affecting service life.

Construction defects

Inadequate work drawings, non-observance of the rules of the art, lack of quality control and poor organisation are among the main causes of construction defects (Calgaro and Lacroix, 1997). The probable causes of some of the most serious and frequent defects are given next.

Reinforced and prestressed concrete bridge decks

Commonly, the quality and use of materials on the site are the most important factors.

In *reinforced concrete*, construction defects include irregular mixing and transportation from the plant, segregation owing to

excessive vibration or pouring from too high a point, unprepared formwork inner faces, poorly executed construction joints and abrupt thermal treatment. Inadequate curing accelerates and deepens drying shrinkage cracks, sometimes beyond the concrete cover, affecting reinforcement durability.

Precast concrete elements with inaccurate dimensions may cause not only matching problems but also defective force transmission and stress distribution.

As to reinforcement, a lack of quality control may lead to a bad choice of steel type and bending radii. Incompatible welding modifies the elastic modulus. Failure to maintain the reinforcement in the designed positions, before and during casting, often reduces concrete cover, causing steel corrosion and concrete spalling; it can also decrease the initially required structural capacity.

In *prestressed concrete decks*, specific construction defects are encountered. Those heavily affecting durability are a lack of waterproofing, incomplete grouting of tendon ducts in humid or aggressive environments and poor sealing of end, deck and transverse anchorages. In areas congested with ducts, especially flange soffits, concrete must be cast carefully or there is a risk of honeycombs, shrinkage cracks and spalling, with all the vulnerability they bring to the constituents.

At the structural level, misplaced tendon ducts are a major defect, giving rise to lateral parasitic forces and outward thrusts that spall off concrete and even laminate the slab. A significant increase in frictional forces and wobble may also result, reducing the effective prestress, and hence the structural capacity. In segmental post-tensioned bridges, especially of variable depth, the lack of interface glue and shear keys allows relative movements of the top and bottom slabs of successive units, increasing the risk of water seepage and corrosion. Poor matching of these segments also generates longitudinal cracks, starting from the contact points. In cantilever construction, very serious accidents may occur when the consecutive construction phases are not in accordance with the specifications.

Steel and composite bridge decks

The different parts of these bridges are usually prepared in factories, where quality control is relatively easy to carry out. Assembly and protection are thus mainly concerned with possible construction defects on-site. Metal bridges can be assembled in different ways: launching on a slipway, using pontoons or cranes, cantilevering, and so on. If not carefully tested for each project, these procedures may all be risky and jeopardise the reliability of the structure. Poorly executed welds can be another assembly defect; in cold weather, they are also liable to quenching. Welding quality is usually tested using non-destructive methods, such as X- or γ -rays or ultrasound, if access is provided in the design. Defective protection against corrosion and difficult access for maintenance both lead to the extensive damage described earlier. Defects mostly concern joints, bar arrangements and other details favouring the condensation or stagnation of water and the accumulation of various humid deposits.

Composite bridge decks may become vulnerable, owing to uncontrolled concrete cracks resulting from certain construction defects:

- non-observance of the specified concrete mix and casting phases intended to minimise shrinkage
- excessive steel–concrete thermal gradient while concrete is setting
- too early formwork removal and insufficient curing.

Structural boundary equipment

Bearings and expansion joints grow old, wear out or deteriorate. In certain cases, they may reveal or induce abnormal structural behaviour by restricting or extending the designed boundary movement.

Bearings allow rotation with or without sliding. They are made of metal, concrete or laminated rubber. Their main defects are corrosion in metal, cracking and spalling in concrete and, in laminated rubber, sheet corrosion and relative slip with rubber cracks. Experience has shown that, in most cases, the corrosion of bearings is due more to their environment than to their material.

When incorrectly designed or executed, expansion joints and their supports deteriorate under repetitive dynamic loads. A few millimetres' difference between the joint and road surfaces strongly amplifies the effect. Dynamic degradation includes failure of metallic elements and their welds, anchorage disorganisation and failure or dislodging under vehicle braking or snowplough action. Frost–thaw cycles and de-icing salts also degrade concrete anchorages. In curved or skew bridges, the use of toothed joints is often incompatible with their additional transverse movements.

Evaluation and testing methods

For the evaluation of bridges, the classical scientific method still holds: gather facts and data, form a hypothesis, start investigating, check results or observations using other methods and establish a theory or draw conclusions.

The evaluation of bridges thus begins with a thorough *desk study* of all drawings, records on the constituents, site reports, environmental conditions and other available information.

On-site, these documents are coupled with a detailed *visual inspection* by trained eyes in search of apparent defects, mentioned in the section on main causes of degradation.

An intelligent *guess* is then put forward as to the probable causes of the observed or suspected defects.

Guided by this hypothesis, the *investigation* process starts. With reasonable selectivity and optimum cost, it should enable evaluation of

- the state and quality of the *constituent materials* with respect to established standards

- the actual *structural response* as compared with the theoretical behaviour of a sound bridge.

While each of these two objectives involves its own methodology, the frequent interaction between material and structural defects often necessitates the association of both.

Methods for the appraisal of the state of the materials involve laboratory analyses and experiments on samples, as well as in situ tests on incorporated constituents.

A variety of techniques are used to characterise structural behaviour: topographic and geometric variations, applied forces at key positions, the local mechanical state and the response to loading and to environmental fluctuations.

A substantial part of the task involves spotting cracks and monitoring their number, size, geometry and direction, and the associated leakage of water and other substances; hence, the importance of beginning with a careful visual survey.

Visual inspection

Targets and access

The first basic inspection step on-site is a visual survey, throughout the bridge, by experienced specialists. The outcome determines the subsequent investigation phases and considerably influences decisions and actions.

On the whole, inspection codes define three visual survey levels:

- *routine*, in search of any recent major flaws
- *thorough*, covering the whole structure at intervals varying with national codes
- *detailed*, at intervals depending on the state of the bridge, with complete appraisal of the extent and severity of deteriorated areas, including an accurate determination of crack widths, which should remain within ≤ 0.1 mm for metal or prestressed concrete, 0.3 mm for reinforced concrete and 0.4–0.5 mm for masonry.

This detailed survey is stepped up with new or further signs of degradation.

Survey targets

Many of the potential visual targets of the campaign have already been discussed and are illustrated in the section on main causes of degradation.

A brief, non-exhaustive reminder of observable flaws may be outlined for a few categories.

- *Reinforced and prestressed concrete*. Cracks or clear-cut discontinuities varying in position, direction, width and geometry; structural cracks (shear, flexure, torsion); cracks tracing prestress tendons or reinforcement bar grids; crack patterns; scaling; spalling; segregation cavities (honeycombs); swelling; bursting;

quick or slow disintegration (sulphate reaction, carbonation); leakage or traces of gel, rust or other soluble reaction compounds; efflorescence; stalactites; exposed and possibly corroded reinforcement (search for concrete cover thickness and density).

- *Steel*. Structural cracks (shear, flexure, torsion, etc.); deviation signs of elastic instability (buckling, warping); rivet or weld failure; defective waterproofing or anti-corrosion protective layers; pitting corrosion at water retention points; stress cracking corrosion with surface oxides under tension; metal dust from fretting at contact surfaces of parallel wires; fretting fatigue wire failure (through oscillatory friction); brittle failure in high carbon steel under excessive concentrated stress.
- *Composite bridges*. Hair or wider cracks around slab–girder connectors; relative slab–girder slip; transverse discontinuities with possible traces of water infiltration.
- *Boundary equipment*: ageing or dysfunction of bearings and expansion joints; deformed parapets.

As in the medical field, inspectors always bear in mind that similar visible defects may stem from *completely different origins* and should be unmistakably distinguished and clearly identified, as in the following examples.

- Scaling of concrete surfaces may result from either frost–thaw cycles or extreme compression owing to environmental or mechanical effects, respectively.
- Shear cracks *at the supports* have much quicker and severer consequences than similar equally inclined shrinkage cracks at a statically neutral position in span.

Current means of access

Currently, most equipment employed ranges from simple binoculars and focused light sources, used from the ground at preliminary stages, to heavy mobile platforms, allowing closer observations but often disturbing traffic during their transportation and site operation (Figure 24.18).

The persisting trend towards ever-larger structures further complicates the use of conventional access equipment, with increasing concerns about personnel security and substantial rises in expenditure.

Prospects

An idea thus came up to replace the current cumbersome equipment with an airborne, remote-controlled, highly advanced colour video camera, supplying images of metrological value to deliver, after treatment, reliable qualitative and quantitative information on the observed degradation. Drone technology transfer contributed to certain development phases of the apparatus and associated software.

The camera carrier must be an unmanned aerial vehicle (UAV), smoothly performing horizontal, vertical and stationary flights that the mission necessarily demands – namely, a first fast reconnaissance round, followed by other rounds for

closer scrutiny of the selected targets (Derkx and Sorin, 2005).

Consequently, the drone option is for a lightweight UAV mini-helicopter, <2 m in dimension, 9 kg in weight (Figure 24.19), equipped mainly with a Global Positioning System (GPS) antenna for automatic navigation and a high-frequency (HF) module for signal control and video image transmission. While approaching the target (Figure 24.20), the maximum horizontal and vertical speeds are 10 m/s and 2 m/s, respectively.

A powerful automatic pilot includes on board a flight control computer and a complete set of navigation instruments. Data on flight parameters, acquired through various sensors, are processed and sent to the servo actuators controlling speed, position and stabilisation.

The colour camera itself is an ultracompact lightweight tri charge-coupled device (CCD), with a high-zoom lens and wide surveillance range, mounted on a gyro-stabilised turret (Figure 24.21). A computer-aided remote control system smoothly operates the turret pan–tilt rotations, keeping the camera pointed towards the target, with optimum zoom and auto-focus. Images are directly recorded on board and transmitted by the HF module to the ground station screen.

The airborne navigation and vision instruments weigh 5 kg, bringing the total flying load up to 14 kg.

The mission is thus entirely automatic, with complete pre-configuration from the ground station, where a multi-window control panel, coupled with a laptop, shows the task progress and parameter systems in real time.

The whole equipment and procedure have been tested on a reinforced concrete bridge in an urban area. Figure 24.22 shows three cracks, detected from a distance of 9 m: the upper left and right cracks were 0.25 mm and 0.50 mm wide, respectively. The lower crack was 1 mm. Further efforts now aim to attain a detection accuracy of 0.1 mm.

While already maintaining a reasonable visual inspection quality, this new approach significantly decreases human risk, cuts down overall expenses, eliminates surface traffic disturbance and may even help to extend the scope of the mission itself.

To push back the performance limits, mainly facing the detection of ≤ 0.1 mm wide cracks, research moves towards further objectives:

- sharper image contrast and three-dimensional or stereo vision
- strain monitoring to spot active cracks, using shearography
- image processing for a still higher quality, by eliminating speed-generated blur and unfocused lens and other parasitic effects
- very precise definition of the image position, by adding *differential* GPS and fixed markers, for a clearer future reference and a more reliable reconstitution of successive images.

However, in spite of the reduced UAV size and speed, the mission requires complete mastery by highly qualified and trained personnel and remains subject to strict security precautions and air traffic regulations, especially in urban areas.



Figure 24.18 Conventional platform for close visual inspection of a bridge



Figure 24.19 Drone helicopter approaching the target (Derkx and Sorin; LCPC)

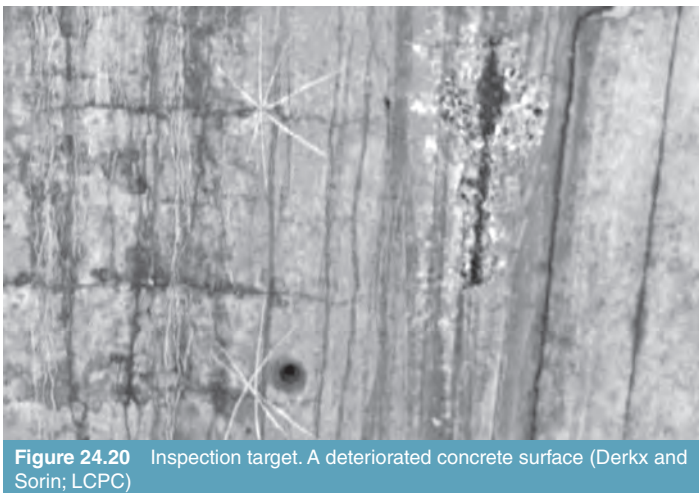


Figure 24.20 Inspection target. A deteriorated concrete surface (Derkx and Sorin; LCPC)

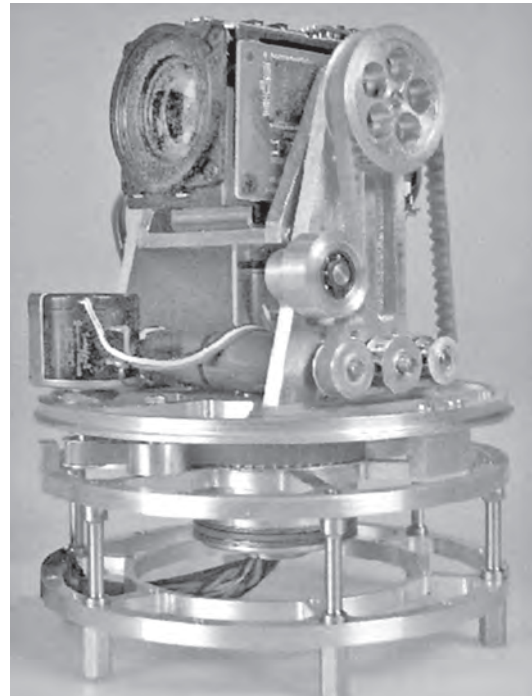


Figure 24.21 Airborne video camera (photo courtesy H Dlahousse, LCPC)

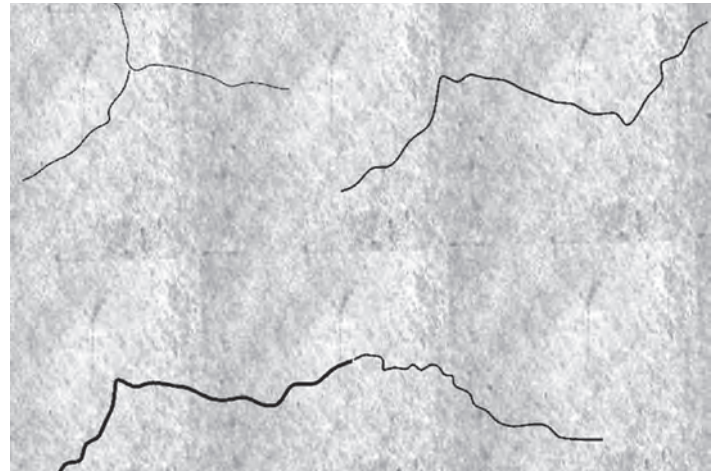


Figure 24.22 Detected cracks after camera data processing on the same deteriorated concrete surface as in Figure 24.20 (Derkx and Sorin (2005); LCPC)

least structurally vital points of the bridge. Hence, though insufficiently representative, they can serve, after size effect correction, as calibration references, completing documentary information and non-destructive test data.

Physical and mechanical tests

For *concrete*, physical tests on cored samples include density, porosity, water content and sound velocity measurements. The data they supply, compared with established standards and reference points, help to delimit the extent of variation or degradation and, qualitatively, estimate their intensities. For

Tests on the constituent materials: investigation of samples

Extracted through partially destructive procedures, samples are usually limited in size and number and taken from the

instance, γ -densimetry is used in the laboratory on concrete samples to determine the percentage of water loss variation with depth and thus enable a correlation with drying shrinkage eigenstresses. The density profile of a concrete slab can also help to estimate the depth of its degradation after exposure to fire. Porosity measurements usually reflect the quality of concrete cover protecting the reinforcement. Finally, before samples undergo destructive tests, it would be useful to measure their longitudinal and diametrical sound velocity (Figure 24.23), a technique discussed later.



Figure 24.23 Sonic test on concrete sample (photograph courtesy of LRPC Lyon)

For *steel*, samples are taken from already broken wires of suspension or stay cables and of post-tensioning tendons. The fracture surfaces can be examined to determine the cause of failure, such as pitting or stress cracking corrosion and fretting fatigue.

Concrete or steel samples are, respectively, subjected to compressive or tensile destructive tests to determine the corresponding ultimate strength of the material and, in the process, its stress–strain relationship. After ductile or brittle failure of steel samples, as already shown in Figure 24.13, microscopic examination of the fracture surfaces may partially reflect their potential mechanical properties. Although these mechanical tests are simple and conventional, concrete samples very rarely have the quality and dimensions of standard cylinders; results may therefore be seriously misleading unless all defects and size effects are carefully accounted for.

Petrography and metallography

Petrographic examination, using electronic and possibly optical polarising microscopy, can give the mineralogical constitution of concrete ingredients and their degree of deterioration. As illustrated earlier in this chapter, these observations identify ASR and expansive ettringite destructive products.

Metallography provides complete data about the nature, structure, grain form and size (texture), impurities and formative

stages of the metal, and hence its properties (Horstmann and Henry, 1979). It can be associated with chemical analyses.

Chemical and physicochemical tests

For these powerful methods and the data they supply, even small samples can be sufficiently representative. However, this double advantage is partially tempered by high costs, necessitating, as already stated, a judicious selection of available methods, guided by the hypothesis formed on the probable causes of observed or suspected flows.

Regarding *metals*, chemical analyses can identify the elementary constituents. They are completed by metallography. Electron microprobes are used to determine the nature and distribution of metallic or non-metallic inclusions.

Concerning the *concrete* material, current methods include

- elementary chemical analyses for oxides using a plasma torch
- X-ray diffractometry on concrete powder samples to identify their various minerals, superficial deposits and crystallised deterioration products
- differential thermal analysis and thermogravimetric analysis, where the physicochemical transformations and weight loss, respectively, of the sample are monitored, while different components are successively modified or destroyed, each at a given temperature
- further scrutiny under an electron microscope to examine destructive reaction products.

In reinforced concrete and post-tensioned bridges, following field tests on undisturbed material, samples for chloride detection are taken from grout in ducts and concrete at joints and anchorages. Accessible trapped water is also sampled for chlorides, other corrosive ions and pH. The same is done for corrosion products observed on tendons or reinforcement.

Together, these methods supply most of the required information on concrete composition: the actual cement content and nature are compared with the initial construction data to detect any deterioration. The initial dry mix, grading and chemically linked water can also be determined to help understand the degradation mechanism.

Tests on the constituent materials: investigation of in situ incorporated materials

In spite of the very useful data acquired through extracted samples, non-destructive tests (NDTs), and even intrusive methods, remain necessary on the same materials incorporated in the structure. Laboratory and field investigations are complementary: the former sets a reference, the latter facilitates a reasonable extrapolation to reality.

Non-destructive tests on concrete

Such tests as the hammer strike and ultrasonic pulse velocity methods are used to appraise the variation of material properties. Tests involving impact echo and radar were mainly developed to explore the inner geometry of the investigated volume. Radar can be an intermediate technique for exploring concrete and certain inclusions associated with embedded steel.

The hammer test

A small metal projectile is flung by a spring, through a tube, against an anvil in contact with the concrete surface. On rebounding, the projectile contracts the spring through a distance proportional to the superficial concrete hardness, roughly reflecting its compressive strength. While unquestionably simple, this old method gives only qualitative results and relative values. It may be of interest for concrete areas with significant clear-cut differences in quality.

Sonic tests

Theoretically, in a homogeneous medium, the propagation velocity of ultrasonic waves is a simple function of the elastic modulus, Poisson's ratio and density. In concrete and other heterogeneous materials, this relationship does not really hold but the distribution of measured relative velocities can reflect the quality and continuity of the assessed medium (Cote, 1996; Prost, 1974).

The ultrasonic pulse velocity (UPV) method is among the best-known methods of this category. Using a specialised piece of equipment, the velocity is obtained by measuring the wave propagation time between an emitting source E and a receiver R . These two devices, generally piezoelectric, are placed either on opposite faces or on the same surface of the structural element. The former arrangement, shown in Figure 24.24(a), can better assess inner layers but may be liable to errors. The latter is more often used, especially when only one face of a tabular member is easily accessible. As illustrated in Figure 24.24(b), the emitting source is fixed at the starting point, while the receiver is moved along a straight line, preferably at equal intervals. The testing length may be 1–2 m and the pace 10–20 cm. The propagation time is automatically measured using an electronic counter with microsecond accuracy. In a homogeneous continuum, the time–space diagram is invariably a straight line, which, given the incremental approach, does not need to pass through the origin (Figure 24.25(a)).

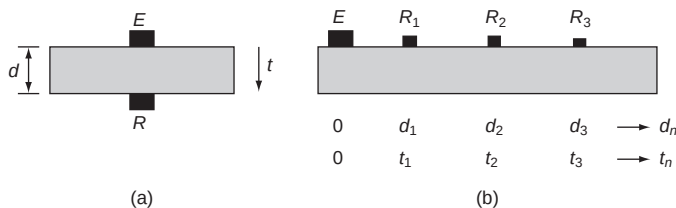


Figure 24.24 Measuring sound velocity (distance d /time t): (a) through the thickness of a concrete flat element; (b) on the surface

Measurements usually continue along several other parallel lines, forming a 'data grid', to map the assessed surface. Ultrasonic pulse velocity is thus used in concrete and masonry bridges to:

- spot discontinuities and local defects (Figures 24.25(b) and 24.25(c))
- appraise the quality *variation* of the material over an area, by mapping the measured apparent velocity ranges (Figure 24.26).

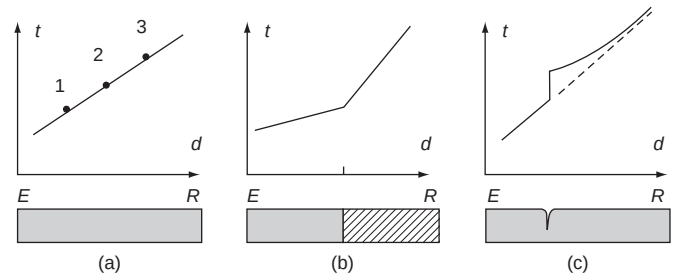


Figure 24.25 Different time–space configurations using the ultrasonic pulse velocity: (a) linear time–space relationship, representing a homogeneous continuum; (b) curve discontinuity at a construction joint; (c) crack effect

It should be noted, however, that there is no close relationship between measured velocity and concrete compressive strength. In certain cases, this strength may be estimated, but only after calibration on extracted samples and fitting in correlation models.

Seismic tomography

The UPV method has successfully been extended to enable, from surface observations, a more direct mapping of the velocity range contours, through a plane section of the investigated object (Cote and Abraham, 1995). The method is based on the tomography principle, already practised by other means in the medical field. In civil engineering applications, seismic waves are generated in the structural element and propagate from a

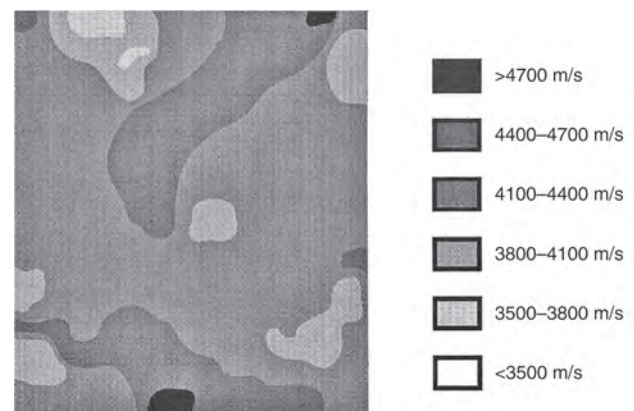


Figure 24.26 Sound velocity ranges reflecting material quality variation

group of sources to a group of fixed receivers, placed on the surface with given coordinates (Figure 24.27).

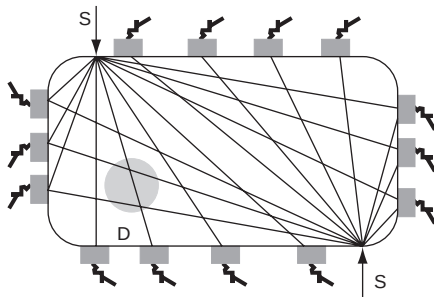


Figure 24.27 The principle of seismic tomography (only two source positions, S, are shown out of several needed): the ray coverage quality depends on the number and direction of rays intercepted by disc D, for all its positions in the investigated area; the size of D is closely related to the wavelength used

The wave propagation times, along their unknown respective source–receiver paths, or rays, are recorded. From the acquired time–distance data, a computer-aided solution of the inverse problem yields the velocity range contours of the assessed section. As already explained, the resulting map may be interpreted to appraise the relative variation of concrete or masonry properties. If requested, a three-dimensional description of this variation can also be obtained either by repeating the procedure at successive parallel sections of the investigated volume or through a direct three-dimensional imaging technique. Although relatively expensive, seismic tomography is fast and easy to apply. It may be best suited for checking the effectiveness of grouting, evaluated through comparison of the velocity maps before and after the strengthening operation. An example is given in Figure 24.28. It is also worth remembering that such a comparative approach minimises inherent measurement errors.

Impact echo (IE)

Generated by mechanical impact, stress waves propagate within the concrete structure and are reflected to the surface

by internal discontinuities and external boundaries. Near the source, a transducer records the reflection time signals, which, in turn, are converted to frequency data. Analyses of the detected frequency peaks on the amplitude spectra, coupled with velocity calibration, locate and characterise existing concrete discontinuities. These can be voids, plate delaminations or lack of grout in post-tensioning ducts. Cornell University was the first to develop the IE method and has so far obtained the best results by adapting the impact force to the properties of the structure.

Ground-penetrating radar (GPR)

Material discontinuities may also be detected by radar, a technique analogous to the sonic IE technique in its general approach (Cariou *et al.*, 1997). Through a concrete structure, electromagnetic waves can be emitted and their echoes received from interfaces, using a specific antenna and keeping the signals in the time domain. Reflection time signals, at successive antenna surface positions, form a ‘radar profile’, where echoes owing to material discontinuities can be spotted. If the wave velocities in the assessed medium are given or assumed, the measured time can be converted to depth, determining the echo position. In the same material, both GPR and IE methods have the same order of wavelength and solving power.

Ground-penetrating radar is mainly used to determine thicknesses and locate plate delaminations or relatively large voids in beams containing tendon unlined or plastic ducts. Metal ducts, opaque to electromagnetic waves, can hence be indirectly located. The inclination of fractures may be estimated by establishing successive parallel radar profiles at a convenient spacing.

In the presence of reinforcement, wave diffraction by steel grids considerably complicates interpretation, unless the bars form a mesh larger than the wavelength and have a well-known geometry, with a sufficiently simple arrangement facilitating the correct orientation of the antenna set (Figure 24.29).

On masonry structures, applications are quite possible, but only with great care when crossing relatively conductive materials, such as clay.

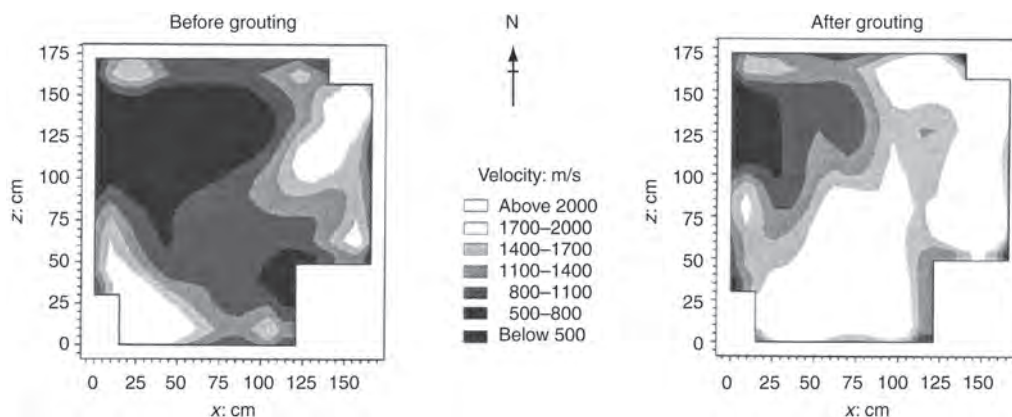


Figure 24.28 Grouting effectiveness in strengthening a masonry pier, shown by tomography

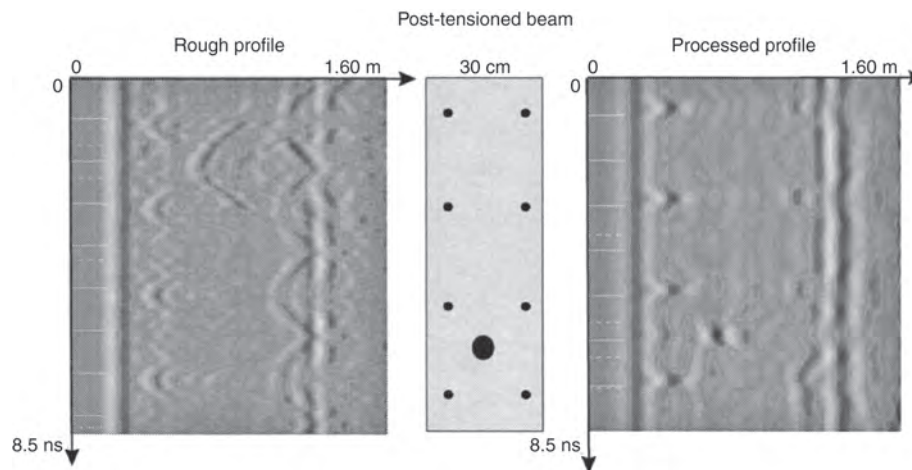


Figure 24.29 Radar profiles, before and after processing, obtained on a concrete beam with a single prestressing tendon between two layers of reinforcement bars

Site tests on reinforced concrete materials

As already stated, reinforcement rusts when in contact with sufficiently high contents of aggressive reagents, coming from the environment, such as chloride ions and carbon dioxide (concrete carbonation). To check for reinforcement corrosion, and possibly predict its extension rate, specific NDTs are conducted on both the steel and its protecting concrete cover. These tests are also supported by intrusive investigation and sampling methods.

Locating reinforcement

Specific magnetic devices or cover meters have been commercially developed to enable non-destructive location and identification of embedded steel awaiting investigation. Guided by construction drawings if available, the cover meter is run over the concrete surface to trace the actual reinforcement bar positions. To supply correct values, readings of concrete cover and bar diameters are first calibrated by direct scale measurements through a small borehole at a reference point.

Other more advanced metal detectors, developed for pre-stressed concrete, will be described next.

Detecting reinforcement corrosion

To locate corroding reinforcement, the most common procedure involves measuring the steel *half-cell potential* E_c . This is the potential difference between a point on the reinforcement, reached through a small borehole, and a mobile reference cell run over the concrete surface (ASTM, 1999; Brevet, 1983). A millivoltmeter is connected to each of these two poles (Figure 24.30). The reference cell is often either a copper–copper sulphate or a silver–silver chloride electrode. The optimum spacing of measurement points may range between 30 mm and 1 m, depending on the accuracy/cost ratio.

The measurement positions and the corresponding values of E_c are mapped and the *equipotential* contours plotted (Figure 24.30). Data can be acquired at other reinforcement levels to obtain section mapping.

To estimate the reinforcement *corrosion risk*, ASTM C 876-91 recommendations divide the obtained E_c values into three classes. For a reference cell of saturated copper–copper sulphate, the correlation is

- $E_c \sim -200 \text{ mV}_{\text{CSE}}$: corrosion unlikely (passivation)
- $-350 < E_c < -200 \text{ mV}_{\text{CSE}}$: corrosion possibility
- $E_c < -350 \text{ mV}_{\text{CSE}}$: high corrosion probability.

Hence, the corrosion risk increases with more negative potential differences.

It should be noted, however, that the half-cell potential data do not necessarily convey a certainty but only a probability of the state of the reinforcement and that, for reliable

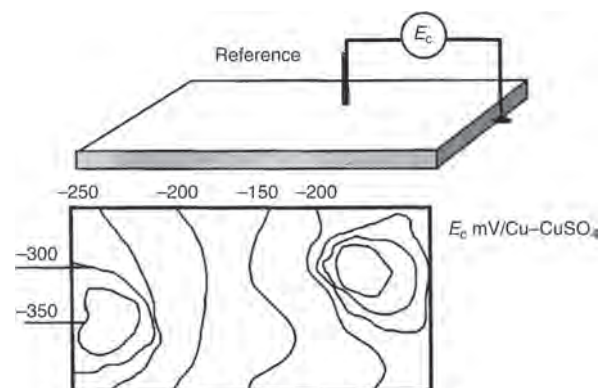


Figure 24.30 Principle and application example of the half-cell potential measurement E_c , reflecting steel corrosion probability in concrete

measurements, certain conditions must be met: electrically continuous bars, sufficiently humid concrete cover and an uncoated concrete surface. Obviously, the method does not apply to post-tensioned structures with metal ducts. However, progress has been made to extend this method to subaquatic structures.

Tests on concrete cover

Analysis of the concrete cover leads to the evaluation of the steel corrosion risk or the identification of the reagents that might have already started the mechanism. Tests thus concern surface permeability, electrical resistance, carbonation extent and chloride content.

- **Surface permeability**, a property linked with durability, is measured on-site by placing a bell-shaped container firmly against the concrete surface and pumping the air out to produce an internal vacuum (Figure 24.31). The time needed to restore the initial atmospheric pressure reflects the permeability. This NDT helps to appraise the resistance of concrete superficial layers to the infiltration of external aggressive reagents. Permeabilities can either be compared at different points of the structure or monitored versus time at a given position.
- **Electrical resistance** of the concrete cover is inversely proportional to the moisture and salt contents, hence to the corrosion risk. It is measured by an NDT for locating highly suspected areas. Four metal electrodes are placed on the concrete surface and a current is induced between the two remote ones (Figure 24.32). The potential difference V , measured between the two neighbouring electrodes, is an inverse function of the corrosion risk.
- **Carbonation extent** is determined by sampling cylindrical concrete cores, splitting each through its diametrical plane and applying on phenolphthalein (a coloured indicator) to the two freshly exposed surfaces. If a pink colour appears, no carbonation is assumed to exist and the pH exceeds 9.
- **Chloride content profile** is traced by drilling at different depths through the concrete cover and analysing the extracted powder in the laboratory.

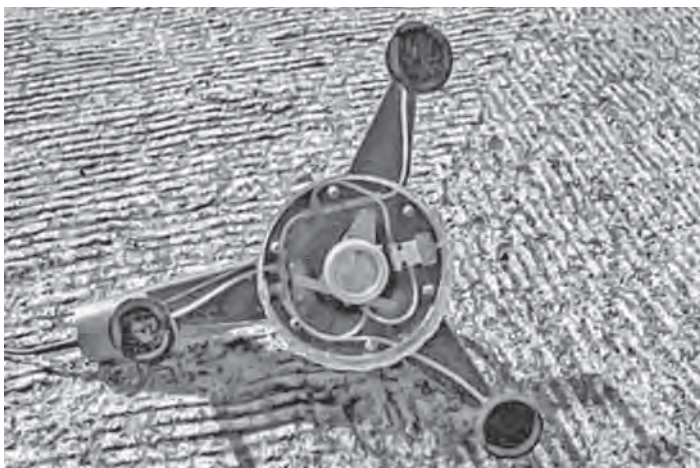


Figure 24.31 Top view of transparent vacuum bell for measuring the permeability of concrete surfaces (photo courtesy LREP)

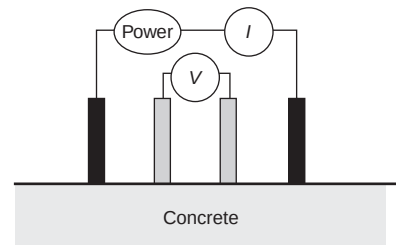


Figure 24.32 Measuring concrete electric resistance V/I , where V is the potential difference between the intermediate electrodes and I the current induced by the power source

Predicting corrosion extension: polarisation resistance method

Several electrochemical methods that utilise polarisation resistance techniques are used on-site to measure the *corrosion rate* (Feliu *et al.*, 1989). In a polarisation test, a direct current is passed between the steel and a metal counter-electrode, often placed on the adjacent concrete surface, above the tested reinforcement (Figure 24.33). Polarisation spreads through the concrete, roughly as a truncated cone, from the counter-electrode down to the reinforcement plane. If S is the polarisation-affected lateral area of the bar and $i = I/S$ is the current density, the resulting steel potential change E can be measured against a reference electrode placed close to the reinforcement. The DC polarisation resistance R_p is the slope of the $E-I$ curve (I is measured in amps, E in volts, S in cm^2 and R_p in Ωcm^2).

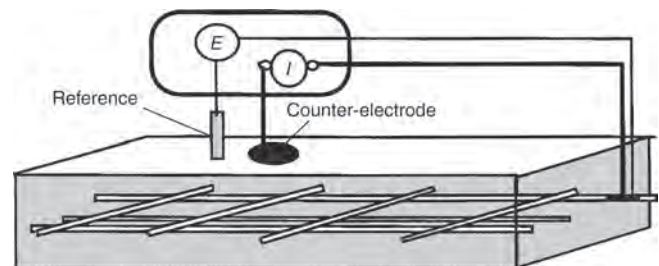


Figure 24.33 Polarisation test

For polarised reinforcement with neither cathodic protection nor stray current effect, the current density reflecting the corrosion rate I_{corr} is inversely proportional to R_p , hence

$$R_p = B/I_{\text{corr}} \quad (24.1)$$

where constant $B = 0.026 \text{ V}$ (if R_p is measured in Ωcm^2 and I_{corr} in A cm^{-2}). As the polarisation resistance is usually measured at only a few points of the bridge, these must be carefully chosen in the light of other investigations if possible. The lateral areas of the polarised bar segments should be well determined. Acquired data must be corrected for the instantaneous effects of variable climatic factors, temperature and humidity. Several measurements at consecutive time intervals are therefore needed to obtain a reliable mean value of the corrosion rate.

Whatever the type of the equipment used, a corrosion rate I_{corr} greater than $10^{-6} \text{ A cm}^{-2}$ is considered to be significantly high.

Built-in sensors

To monitor steel corrosion, sensors may be permanently placed in the concrete, at critical locations where corrosion is likely to occur. They are mainly of two types.

- A *macrocell sensor* consists of two embedded plain carbon steel bars; one near the concrete surface, the other at a deeper level (Figure 24.34(a)). As the shallow bar usually corrodes first, its half-cell potential will subsequently differ from that of the deeper one, which is normally placed in sounder concrete. Hence, measuring the potential difference between the two bars detects corrosion initiation and sometimes even determines the corrosion current, to estimate the rusting rate.
- A *polarisation resistance sensor* comprises three closely positioned elements. One is a steel bar placed near the reinforcement. The second is a steel piece used as a counter-electrode to measure the polarisation resistance of the first steel bar. The third element is a reference electrode (Figure 24.34(b)). Measuring the polarisation resistance thus determines the steel corrosion rate at the sensor location.

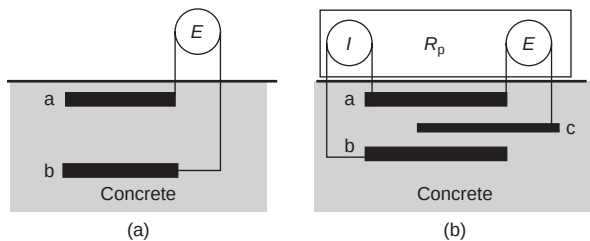


Figure 24.34 Sensors embedded in concrete: (a) macrocell sensor for detecting steel corrosion initiation: a and b are steel bars; (b) polarisation resistance sensor for determining steel corrosion rate: a and b are steel bars, c is a reference electrode

Site tests on prestressed concrete constituents

Post-tensioned bridges can be severely damaged without showing significant external signs. Surface visual inspection should hence be more detailed and accurate than for other types of concrete construction. From a material durability point of view, corrosion and its consequences on service life are the major concern. But while reinforcement corrosion is relatively simple to detect by the aforementioned methods, deeply embedded tendons and their ducts often require direct visual examination through intrusive or partially destructive procedures. To make this internal inspection as selective and efficient as possible, a detailed desk study and NDTs should be carried out first.

A desk study of available drawings and other records facilitates the location of the parts usually most vulnerable to chloride attack: anchorages in their different positions and configurations; joints between segments; construction joints, if

poorly executed and high or low points along the duct profile, where the former may be too close to the deck top and the latter a potential water-collecting trough.

To better locate and identify tendon ducts and suspected defects, the specific NDTs in use mainly include radiography, radioscopy, perhaps the magnetic flux method and other lighter but less efficient techniques, such as steel detectors, impact echo and ground-penetrating radar. These last two methods have already been considered, in the section on tests on the constituent materials.

Based on these criteria, a very limited number of points are selected for visual examination and other associated tests, thus completing the field investigation.

In the following subsections, more details will be given on the equipment and procedures related to these methods.

Radiography and radioscopy

If a beam of γ - or X-rays strikes one side of the assessed concrete member and passes through its thickness, the emerging radiation energy will show up, with differential attenuation, on a photographic plate placed against the opposite side. Steel, denser than concrete, thus leaves a lighter trace, while missing grout or other voids give darker images. This is the basic principle of conventional *radiography* (Figure 24.35).

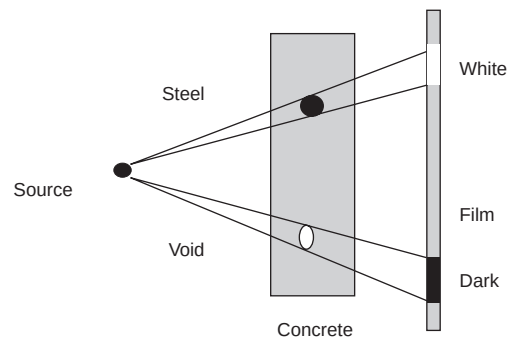


Figure 24.35 The radiography principle: the denser the element, the lighter the image

Already applied in steel construction for checking the quality of welds, the technique has now been successfully extended to post-tensioned structures (Guinez *et al.*, 1999).

Gamma radiography uses a radioactive source, iridium (^{192}Ir) or cobalt (^{60}Co). For a concrete thickness up to 0.5 m, it detects

- incorrect positions of tendons, ducts and reinforcement bars
- broken or slack wires or strands
- voids in ducts, owing to poor grouting (favouring tendon corrosion)
- defective ducts
- concrete bonding quality
- concrete discontinuities, such as honeycombs, certain cracks, defective construction joints or density variation.

However, radiography cannot directly detect steel corrosion but only its advanced consequences: seriously reduced sections or fractured tendon elements.

High-energy X-rays, generated by a linear accelerator, enable the investigation of thicker concrete members and with greater safety – there are no contamination or standby radiation hazards and the exposure time is shorter. Moreover, the penetration is extended to a thickness of 1.2 m and the image quality is good.

The exposure time varies directly with the square of the source–film distance, inversely with the source activity and exponentially with the investigated concrete thickness. Given these considerations, the source–film distance ranges in practice from 0.7 to 1.2 m. Through a concrete thickness of 0.3 m, for example, the exposure time is 46 min for iridium, 2 min for cobalt and 20 s for a linear accelerator.

Radiographs (Figure 24.36) must be interpreted by qualified staff.

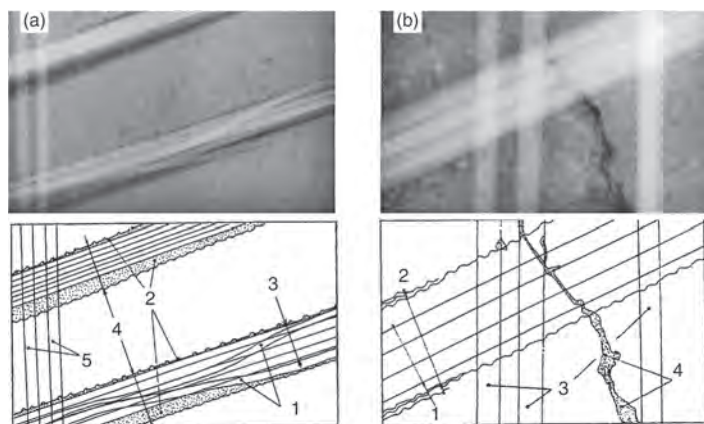


Figure 24.36 Two radiographs and their interpretation: (a) 1, broken or slack wires; 2, complete absence of grout; 3, parallel tendon wires; 4, tendon ducts; 5, reinforcement bars; (b) 1, sleeving ducts; 2, tendon strands; 3, reinforcement bars; 4, construction joints (photograph courtesy of LRPC Blois).

However, this method has several imperatives and limitations.

- The use of radiation sources is subject to *strict safety rules*, not only for the operating staff but also for all people present in the area. During the operation, entrance is strictly forbidden within a 20–100 m radius of the radiation source, depending on the type of the equipment and conditions of use.
- Occasionally, difficult access to both sides of the structural member may hinder the optimum source and film positioning and orientation.
- Certain configurations sometimes complicate the interpretation of radiographs, for example, congested or horizontally parallel tendons.
- Radiography is relatively expensive, like many other specialised activities.

Further extension of the radiography principle has been achieved, allowing:

- a continuous survey throughout the tendon lengths, with variable three-directional incident angles for easier interpretation
- image recording on videotape in real time.

This advanced version is *radioscopy*, using X-rays generated by a linear accelerator placed on a mobile platform. It enables the inspection of a greater concrete thickness with a shorter exposure time. Radioscopy, however, does not give the location accuracy of radiography. Conventional radiography is hence also integrated in the inspection system for more precise radiographs at points of particular interest prior to eventual drilling and regrouting.

Radiographic methods have greatly facilitated the location of poor grouting quality in many bridges liable to tendon corrosion.

Despite the stringent safety precautions they do require and the reluctance they may arouse, radiography and radioscopy remain at present among the most powerful non-destructive investigation methods for post-tensioned bridges.

Magnetic stray field method

Recent significant developments in procedure, metrology and signal analysis now enable a more reliable use of the magnetic stray field method for the detection of tendon flaws on-site (Sawade *et al.*, 1997).

The investigated post-tensioned concrete member is wholly subjected to a magnetic field H_0 . Leakage, owing to tendon breakage or section reduction, disturbs the flux distribution locally and creates a corresponding magnetic stray field H_s , detected, without contact, using a mobile Hall-effect probe, advancing on a parallel longitudinal rail (Figure 24.37).

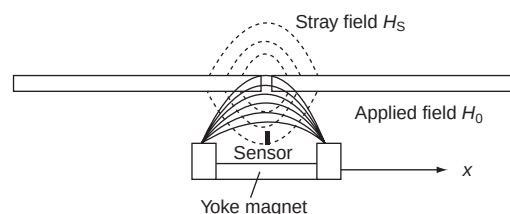


Figure 24.37 Principle of magnetic stray field method

However, stray field signals are affected not only by tendon defects but also by the stirrups (and perhaps their tying wires) close to the probe. To suppress the parasitic signals emanating from these mild steel elements, the new approach recommends *several* active field measurements $H(x)$, taken at *successively increasing magnitudes of the applied field* H_0 . As H_0 rapidly decreases with distance, readings at its low values yield signals from the mild steel reinforcement near the surface. Conversely, at large values of H_0 , causing field saturation, the magnetisation increase is greater in the deeper steel layers than in those close to the surface. The signal increase at large values of H_0 is hence mainly due to the deeply placed tendons.

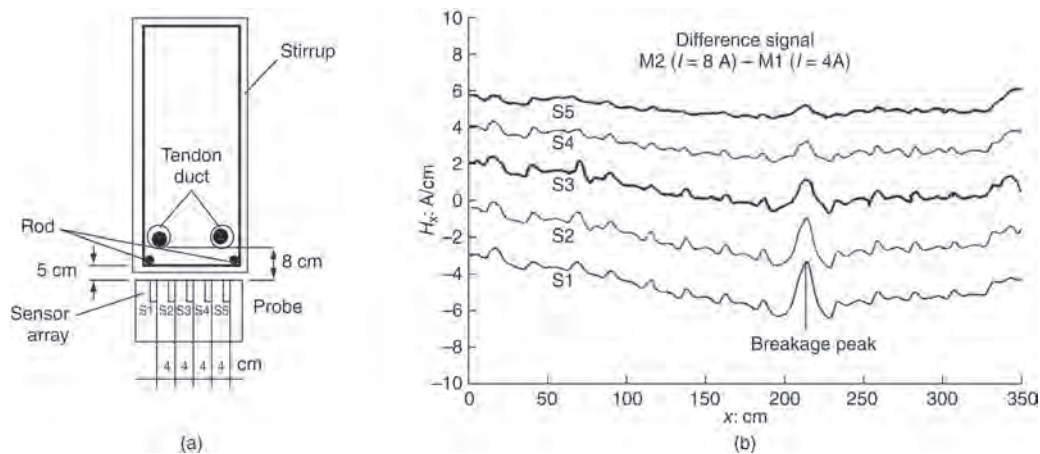


Figure 24.38 (a) Section of instrumented post-tensioned member; (b) typical stray field signal of a single broken tendon

Qualitatively, the detection of a tendon defect is based on a comparison of the shape of the signals at different values of H_0 . Through quantitative analysis, the respective differences of these signals, obtained at different values of H_0 , first suppress the parasitic effects of stirrups near the surface and then amplify the impact of existing tendon flaws on the processed stray field profile, as illustrated in Figure 24.38.

Figure 24.38(a) shows a section of an instrumented post-tensioned member, investigated by the magnetic stray field method in 350 cm segments. Several measurements are taken per segment, under increasing exciting field. Figure 24.38(b) shows the *difference signal* profiles for only two measurements, performed at respective exciting currents of 4 and 8 A.

On these difference profiles, where the portion caused by the stirrups has been clearly suppressed, the tendon near the magnetometer S1 produces a sharp breakage peak. The lateral decrease of the peak towards S5 suggests that the second tendon is intact. These results were confirmed later, after carefully demolishing the concrete.

Steel detectors

Further to the conventional cover meter described earlier, a more advanced surface-operated version consists of a data acquisition scanner and an image monitor. Tendons and reinforcement can be detected, with a selective display at the requested depth, up to ≈ 18 cm. A cursor indicates bar size and position.

Intrusive methods

Internal visual inspection takes place at very few points, carefully selected in the light of the desk study and NDT results, mainly situated where voids in ducts or other defects have been detected or are strongly suspected. Cautious drilling, 25 mm in diameter, is usually acceptable for access to ducts; it may later be slightly overcored if required. A flexible optical fibre videoscope is then inserted through the borehole to examine

tendon, duct and grout, in search for steel corrosion, pitting, wire fractures, or defective grouting (Figure 24.39). Inspection extends as far as the void dimensions allow the probe to reach. Although equally used for ducts, rigid endoscopes may be better recommended for external round-the-corner points, as in support-bearing systems.

Void volume and leakage risk may be estimated to facilitate regrouting. Void volumes are deduced by applying gas pressure from a given container and using the equations of thermodynamics. To check for leakage, the gas pressure drop, if any, is monitored over time.

Tension or slackness of tendon elements can be very locally and qualitatively confirmed by inserting a screwdriver between two adjacent wires and observing whether it can be turned by hand.

Internal samples are taken through access ports, but only after completing all other in situ tests on undisturbed material. Sampling concerns the following, if present:

- grout, for chlorides
- trapped water, for chlorides, corrosive ions and pH level
- steel corrosion products
- fractured wires, for diagnosis and vulnerability estimation to stress corrosion.

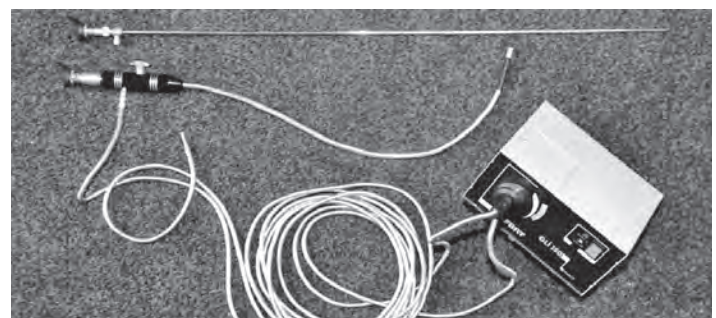


Figure 24.39 Flexible videoscope and rigid endoscope for internal inspection (photograph courtesy of LCPC)

External concrete samples can also be taken for chloride content at joints and anchorages.

After inspection, all boreholes must be thoroughly resealed according to the rules of the art.

Site tests on steel construction materials

In conventional steel bridges, investigation is mainly carried out on extracted samples. Field tests on the material are relatively rare but do exist. Conversely, in suspension and cable-stayed bridges, laboratory tests are hardly able to appraise the material condition of main structural members and field investigation is imperative.

Checking steel elements and welded assemblies

Material hardness tests are still conducted, especially on old structures. When properly conducted on well-prepared corrosion-free surfaces, they detect metal quality variations. With the same precautions, the thickness of structural elements can be measured using ultrasonic methods.

In welded assemblies, several techniques are used to detect cracks or other discontinuities: dye penetrant inspection, magnetic particle tests, radiography and back-reflective ultrasonic methods. The first is the simplest for preliminary diagnosis. Also known as the back-percolation test, it involves applying a special liquid over the cleaned assessed surface, letting it penetrate discontinuities and removing all remaining excess. An absorptive substance is then spread to draw back the infiltrated liquid. The visible absorption traces thus reveal and locate any defects. The liquid must have a very low surface tension and a marked colour. The magnetic particle test is roughly analogous to dye penetrant inspection; magnetic particles are spread and their rearrangement pattern in a magnetic field reflects material discontinuities. The selective use of radiography and reflective ultrasonics for detecting defects is similar to that already presented for concrete.

Checking for corrosion damage in cables

In suspension or cable-stayed bridges, cables comprise concentric wire layers, either coated or threaded into ducts. While external wires can be easily inspected, inner ones may rust and break invisibly, causing cable failure. For a more extensive

evaluation of corrosion and its effects, two methods are usually applied: electromagnetism and acoustic emission.

Electromagnetic survey for single-strand cables. To estimate the corrosion extent and effect on section loss, two half cylinders are placed around the cable and assembled to form a continuous coil (Figure 24.40) fed by a 10kHz alternating current. The resulting variable magnetic field creates eddy currents in the cable, with their corresponding self-induction opposed to that of the coil (Gourmelon and Robert, 1985). Corrosion develops a larger contact electric resistance between the wires and consequently allows a smaller eddy current intensity, thus increasing the self-induction and hence the measured coil impedance. This quantity increases and stabilises at a maximum when all wire layers have rusted.

In practice, the activated coil is moved along the whole cable and consecutive impedance variations are measured. For a reasonable evaluation of corrosion, prior laboratory calibration tests are required on typical cables of different degradation depths and intensities. For each configuration, a relationship can then be established between the oxidation ratio and the percentage impedance variation, taking a perfectly sound cable as a base value.

It should also be noted that the electromagnetic method cannot distinguish between pitting and stress cracking corrosion.

Attempts have been made to use electromagnetism to detect local defects, especially wire breaks, in single-strand cables. One procedure involves locally magnetising the cable using a 100Hz alternating current through a surrounding solenoid. The receivers are two identical crossed coils, connected in the opposite sense, thus detecting flux variation caused by local defects while hardly responding to uniform oxidation. A peak on the recorded diagram may indicate a discontinuity, other geometric irregularities or a very localised oxidation. A clear signal is hence necessary but not sufficient to suspect a wire break, while a flat diagram confirms the total absence of local defects in the cable.

Acoustic emission survey. In a corroded tensioned cable of a bridge, wires can break spontaneously. For a clearer assessment, it is often necessary to monitor the number and successive locations of these events over a period, depending on the situation.

When a wire breaks, the released energy creates a transient

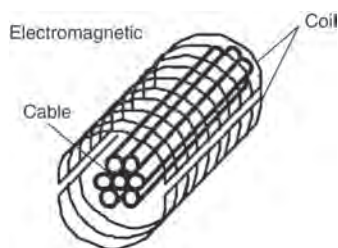


Figure 24.40 The electromagnetic survey principle: a two-element coil carrying an alternating current is placed around the cable; the measured self-induction, L , depends on the cable corrosion

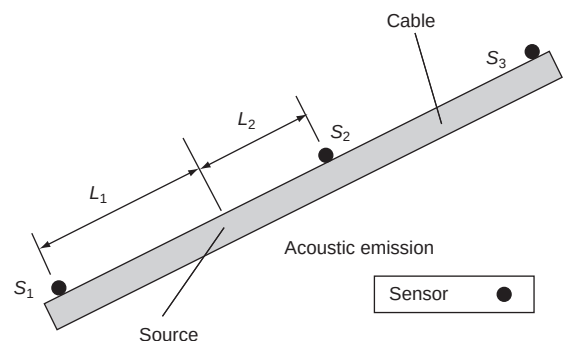


Figure 24.41 Acoustic emission technique

elastic wave that propagates through the cable with identifiable amplitude and a practically constant velocity (≈ 4700 m/s). Several sensors, S_i ($i=1, 2, \dots$), distributed along the cable, can determine the respective arrival times $t(S_i)$ of the signal reaching each of them from the same breaking point, called the *source*. The reference time 'zero' is the instant when the wave reaches the closest sensor to the source (Robert *et al.*, 1990).

In Figure 24.41, the given positions S_i and arrival times $t(S_i)$ yield the following: the distance S_2/S_3 and the arrival time difference $[t(S_3) - t(S_2)]$ determine v , the propagation velocity. The first two arrivals are at S_1 and S_2 . The source is situated at distances L_1 and L_2 , respectively, from S_1 and S_2 . The difference $L_1 - L_2$ is given by $v[t(S_1) - t(S_2)]$, and $L_1 + L_2$ is the distance S_1/S_2 . The values of L_1 and L_2 thus obtained locate the source or breaking point.

Sensors are usually accelerometers, spaced at ~ 15 m in cables. They can also be fixed on the concrete surface of post-tensioned bridges, with ~ 5 m spacing to monitor the tendons.

A specific data acquisition, processing and remote transmission system locates and reports cable wire breaks in real time.

Figure 24.42 gives examples of results obtained on two suspension bridges, A1 and A2. In A1, a 66 mm diameter suspension cable broke suddenly. The remaining cables were monitored by acoustic emission during a 16 day strengthening period. Acoustic emissions were detected. After the cables were replaced, a detailed examination confirmed recent wire breaks. In bridge A2, only a part of one cable was monitored for 6 months. One acoustic emission per day was detected. After the cables were removed and replaced, visual inspection confirmed the acoustic emission results.

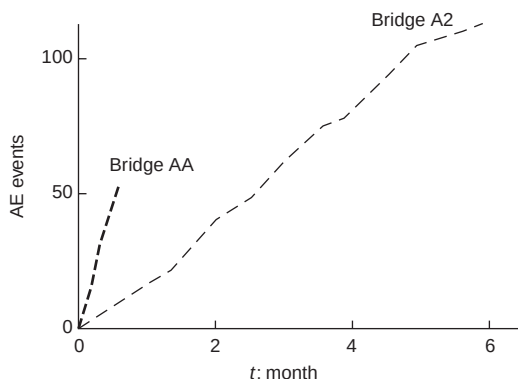


Figure 24.42 Examples of acoustic emission results obtained for two suspension bridges, probably corresponding to wire breaks, as determined after further examinations

Structural evaluation tests

For a given type of construction, structural assessment methods are usually chosen for their capacity to evaluate the required parameters and for their suitability with regard to the mechanical system, constituent materials and the observed or assumed type of damage or deficiency.

The development of these methods, towards better performance and new concepts, did not always follow the chronological order of the structural designer's usual checking procedure. Stress, for example, the first parameter that conventional calculations usually yield, was one of the last made accessible to direct measurement on real structures.

In this section, field investigation methods will concern the general deformation of the bridge and the movement of its bearing points, the response of a sound or locally damaged bridge to loading, the forces resisting the loads, the local behaviour of certain areas or their discontinuities and finally the acting stresses.

A selective use of these techniques facilitates structural assessment and, possibly, optimum strengthening.

General deformation under dead load

Topographic monitoring of the structure's geometry helps to evaluate its general condition. It may disclose support settlement or movement, a modified cyclic response to temperature reflecting gradual damage, irreversible deformation of a concrete deck owing to underestimated creep and other random effects.

Displacements are monitored by periodic precision levelling at several targets on the bridge, preferably referred to a network of benchmarks.

Rotations are measured using inclinometers and distances using Invar tapes. Parallel temperature monitoring is necessary for thermal correction of all measurements.

In addition, a careful visual examination of the parapets or barrier walls can often detect permanent abnormal deformation of the deck.

Span deformation curve under test loading

Load-induced deflections, rotations and curvature are measured along the bridge spans and compared with their theoretical values, in order to analyse the actual general behaviour and detect eventual discontinuities. Customarily, *deflectometry* now stands for deflection metrology while *inclinometry* includes both rotation and curvature measurements.

Deflectometry

Traditionally, deflections under static test loading are among the first data obtained for the approval of a bridge. They set a reference for future assessment.

Before, during and after the loading tests, a detailed inspection must be carried out on the whole bridge by qualified staff. Access to all parts should be provided.

Loads are first applied to the supports, next to the spans, one by one, and subsequently to different span combinations, in accordance with the specifications.

In the case of damage affecting the structural capacity, static test loads are cautiously applied and guided by prior calculations or direct stress measurements. If the load-deflection

relationship deviates from its linear elastic path, the test is immediately stopped and the acquired data are analysed for a decision on possible new service load limit.

Deflections are usually measured at midspans and the supports. Other points may be included, depending on the instrument accuracy, often of the order of millimetres.

There are various existing deflection-measuring techniques and devices, mainly

- topographic levelling methods, demanding highly qualified staff
- mechanical deflectometry (Figure 24.43), necessitating a fixed anchorage point below the bridge, which is not always possible
- electrical displacement gauges, which need rigid independent supports
- laser deflectometry (Figure 24.44), which is more recent, is of delicate implementation, supplies continuous data and is used for high bridges across rivers, railways or comparable obstacles.

The choice depends on the bridge dimensions, deflection range, access conditions and loading procedure. For data confirmation, two techniques are often used in parallel.

Under dynamic loading, at midspan or pier heads, the induced vertical and horizontal deflection components are

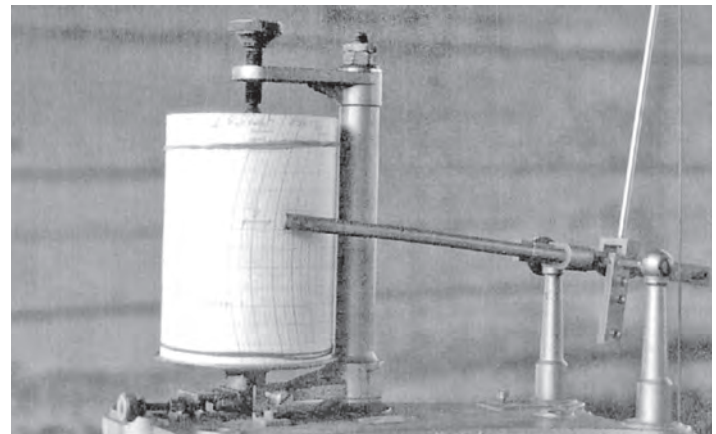


Figure 24.43 Mechanical deflectometer (photograph courtesy of LRPC Nancy)

estimated either directly using seismographs or indirectly using accelerometers, through double integration of the acquired data. However, while this method has some success in monitoring offshore structural elements, the deduced dynamic properties of a concrete bridge are not yet sufficiently sensitive to early damage. Data analysis seems to be influenced more by surface layer degradation than by real structural defects.

Inclinometry

Under loading, through rotation or curvature measurements, inclinometry has proved quite suitable for completing the bridge deformation curve and, in particular, for detecting and monitoring discontinuities, such as flexure cracks or opening segment joints.

Rotation measurements along the span offer a complementary description of the structure's response to loading. They improve the deck deformation data and may extend them to the supports and other neighbouring members. Rotations are mainly measured by inclinometers, now commercially developed with a 10^{-6} to 10^{-8} rad sensitivity, easily fixed on the deck, without external links or reference points.

For a vertical crack, two coupled inclinometers, as shown in the left panel of Figure 24.45, are placed on either side of the crack. Differential readings, $\Delta\theta$, can then monitor the angular opening variation.

Figure 24.46 illustrates the influence of an increasing bending moment M on the behaviour of a detected rotation discontinuity $\Delta\theta$, monitored by differential readings of two coupled inclinometers, A and B, placed on either side of an existing flexure



Figure 24.44 Laser deflectometer: a stationary laser beam emission source (left) fixes the position of the target, a beam receiver cell mounted on a trolley (right), which, being brought to a standstill, has a sliding vertical track linked to a point on the bridge; after an initial reading, the relative sliding vertical distance is the deflection of that point on the bridge (photographs courtesy of LRPC Nancy)

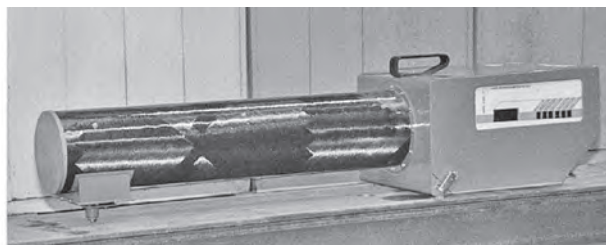


Figure 24.45 Left: coupled inclinometers; right: curvature meter (photographs courtesy of LREP)

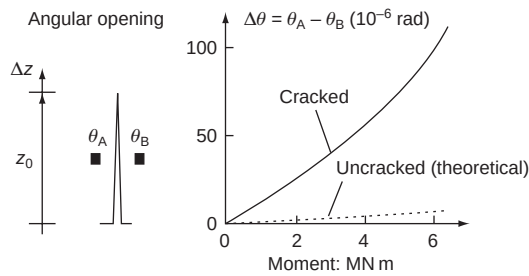


Figure 24.46 Influence of the bending moment on the angular opening of a bridge girder crack, measured by differential inclinometry

crack. The linear first part of the curve $M=f(\Delta\theta)$ represents the angular opening variation of the crack without any further growth, that is, within its initial tip height limit z_0 before the current loading. The non-linear second part marks the crack growth, $z_0 + \Delta z$, under a greater moment M .

Curvature variation measurements are a further step, completing the collection of span deformation data under a test load and reflecting the response to flexure.

In sound sections, curvature variation is conventionally obtained from the strain profile measured by extensometry, using strain gauges or displacement sensors. For most bridges with many cracks and inaccessible residual sections, a more global metrological approach is imperative, as provided by inclinometry, where the developed specific instruments are now accurate, robust and mobile. They supply principal angular deformation data and filter out local disturbances, facilitating the analysis of damage effects on the general flexural behaviour.

Theoretically, the following expressions for curvature $\theta'(x)$ apply to a sound beam section x , of height h and flexural stiffness EI , under a bending moment $M(x)$ and a strain difference $\Delta\varepsilon$ between extreme fibres:

$$\theta'(x) = d\theta/dx = \Delta\varepsilon/h = M(x)/EI \quad (24.2)$$

where the term $d\theta/dx$ represents inclinometry and the term $\Delta\varepsilon/h$ represents extensometry.

For damaged sections, the extensometry expression is difficult to apply, owing to stress concentration and complex strain redistribution. The inclinometry expression remains reasonably valid.

The load-induced curvature diagram $\theta'(x)$ is determined by differentiating closely spaced rotation readings φ_i throughout the span, using either coupled inclinometers or a mobile curvature meter. The latter, shown in Figure 24.45, is basically an instrument supplying automatic differential rotation readings between two points with a fixed spacing. Both types are usually placed on the uncracked, upper or lower, extreme fibre.

The *spacing* of inclinometric measurements is governed by several factors and requirements.

To trace the curvature variation diagram $\theta'(x)$ along a bridge, the required rotation measurement's spacing, s_B , depends on the beam and eventual crack heights, as well as on the accuracy of the instruments and the expected results. In the author's

experience of inclinometry and bridge metrology in general, spacing varies from one-half to one-tenth the deck height, respectively, in sound and cracked configurations.

In *apparently sound* beams, the optimum spacing s_B , between two consecutive rotation readings φ_1 and φ_2 , can be determined as

$$\text{load-induced local curvature } \theta' = (\varphi_2 - \varphi_1)/s_B \quad (24.3)$$

Assuming a negligible error in setting the base length s_B and an intrinsic instrumental uncertainty $\delta\varphi$ at each rotation measurement point, the maximum possible error in the differential rotation reading is $2\delta\varphi$ and the corresponding curvature uncertainty will be

$$\delta\theta' = 2\delta\varphi/s_B \quad (24.4)$$

Along the same instrumented extreme fibre, situated at a normal distance c from the neutral axis, the load-induced stress σ is

$$\sigma = Mc/I = EI\theta'/cI = cE\theta', \quad \text{hence} \quad \delta\theta' = \delta\sigma/cE \quad (24.5)$$

Thus, an uncertainty $\delta\sigma$ in the extreme fibre stress corresponds to an uncertainty $\delta\theta'$ in the measured curvature.

If the load-induced stress is required with an accuracy of $\delta\sigma = \delta\sigma_r$, then, from these equations, the *optimum spacing* s_B of rotation measurements on sound beams, is given by

$$s_B = c(2E\delta\varphi/\delta\sigma_r) \quad (24.6)$$

For example, an apparently sound concrete T-section of height h , with $c=0.4h$, $E=50\,000$ MPa, $\delta\sigma_r=0.25$ MPa and $\delta\varphi=10^{-6}$ rad) requires an inclinometer spacing of approximately $0.16h$.

In *damaged* sections, with sharp curvature variation, the maximum admissible inclinometer spacing should not exceed one-tenth of the beam height, with one obligatory differential reading immediately over the crack. Furthermore, the influence length of a single crack may be as much as of the order of the beam height h , on each side, hence a total of $l_c=2h$. With a number of cracks over a length q , the total range of sharp curvature variation is about $L_c=q+2h$.

The same reasoning applies to the spacing of mobile curvature meter readings.

The moment–curvature method

The aforementioned theoretical suitability and development of inclinometry led to a field investigation method for detecting transverse discontinuities and determining the actual flexural response of sound or damaged bridges (Abdunur, 1997).

The bridge is equipped, throughout its spans, with inclinometry instruments supplying curvature variation data. Under convenient test load configurations, the measured curvature variation diagrams θ' are plotted for the whole length of the bridge.

If these diagrams are regular and reasonably follow the theoretical diagrams, the flexural adequacy of the structure is probably maintained.

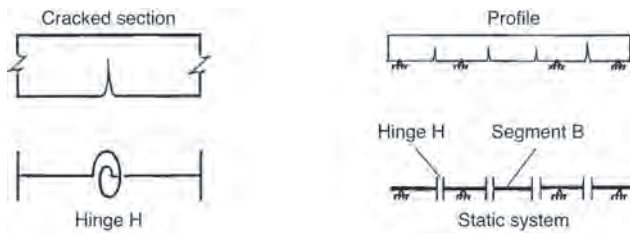


Figure 24.47 Modelling assumptions of cracked sections of a bridge

If, on the contrary, a sharp curvature redistribution appears at certain points, transverse cracks or other discontinuities should be suspected and the structural system may be represented as illustrated in Figure 24.47.

The cracked sections and their disturbed vicinities are assimilated to a series of elastic or plastic hinges H , alternating with sound beam segments B and jointly setting up a new system in equilibrium.

The main difficulty is the realistic determination of the relative residual flexural stiffness $[EI]_H$ of the hinge, where several variables and assumptions are involved. The proposed evaluation of $[EI]_H$, at a point x along the bridge, is hence experimental, based on the relationship between the applied bending moment $M(x)$, the resulting measured curvature $\theta'(x)$ and the flexural stiffness $EI(x)$. It proceeds as follows and as shown in Figure 24.48.

Under a test moment, as already stated, the resulting curvature diagram $\theta'(x)$ is known throughout the spans; in particular

- $\theta'_H(x)$, over the hinge H comprising the crack and its short influence zones on either side
- $\theta'_B(x)$, along the sound beam segments B_1 and B_2 , especially their parts near the crack.

All sound beam segments B are assumed to conserve their given initial stiffnesses $[EI]_0$.

The bridge length is divided into modules (Figure 24.48(a)), each covering the hinge zone H of stiffness $[EI]_H(x)$ and the near parts of both adjacent sound segments B_1 and B_2 of given initial stiffness $[EI]_0$.

For each module, B_1 , H and B_2 , the beam equation is $M = EI$. The curvature θ' is applied at successive sections, more closely spaced through the hinge zone H . At this zone, the moment M_H remains almost constant but the curvature θ' varies considerably and the local stiffness EI varies inversely to it (Figure 24.48(b)). The moment M_H is applied by the sound beam segments B_1 and B_2 . Hence, at any section x within the H length

$$[EI]_H(x) \theta'_H(x) = M_H = [EI]_0 \theta'_B \quad (24.7)$$

$$[EI]_H(x) = [EI]_0 \theta'_B / \theta'_H(x) \quad (24.8)$$

In rare cases, the acquired monitoring data of the curvature θ' are disturbed by extensive *local* parasitic effects, owing to the material of certain structures. The data grow less consistent and less representative of the assessed medium. Consequently, Figure 24.48(c), a specific analytical guidance curve ($\theta' = \exp^{-x^2}$) was proposed and successfully tested to eliminate the dispersal and illustrate a *representative* curvature redistribution profile. This profile can thus be *correctly* compared with the theoretical one, stemming from a corresponding finite-element analysis. Proper conclusions may then be drawn.

This proves useful on difficult sites, where experimental data cannot go beyond a few key sections.

Most often, the *relative* residual rigidity $(EI)_H/(EI)_0$ is preferred for estimating the fractional remaining capacity of the bridge. Its *inverse*, $(EI)_0/(EI)_H$, is usually chosen for graphical representation, as in the actual example shown in Figure 24.49.

The respective residual rigidities of the damaged sections H and sound segments B can now be introduced into a reliable structural analysis programme to define the new mechanical system of the bridge and enable

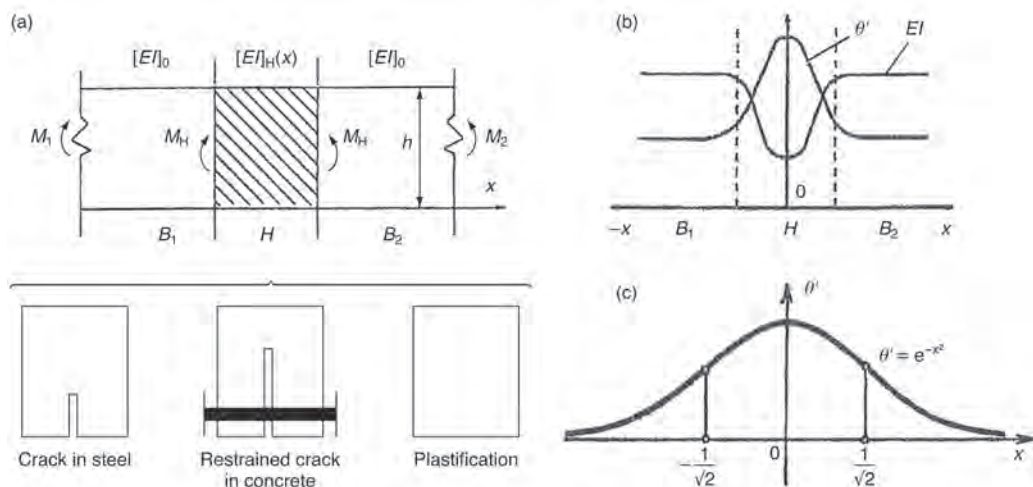


Figure 24.48 (a) Moments M and stiffness values EI in cracked sections H and sound beam segments B ; (b) redistribution of curvature θ' and stiffness EI ; (c) analytical guidance curve ($\theta' = \exp^{-x^2}$) to a representative curvature redistribution profile, when investigation data are disturbed by extensive local effects

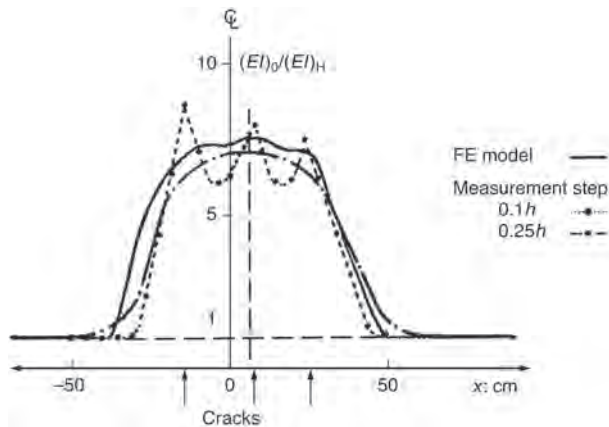


Figure 24.49 Theoretical and experimental redistribution curves of the inverse relative flexural stiffness over a beam segment with three cracks

- prediction of the real flexural response of the damaged structure to any given loading
- evaluation of the residual load-bearing capacity and, possibly, the optimum needs for strengthening.

The whole procedure can be repeated at a later stage to verify the effectiveness of eventual repairs or simply to monitor a time-dependent mechanical change. The moment–curvature method applies to steel or prestressed concrete bridges.

Towards new concepts in bridge acceptance procedures, in the light of the metrological possibilities already discussed, inclinometers can now fully accompany deflectometry in approval procedures for newly constructed or strengthened bridges.

Measurement of forces acting on a bridge

Support reactions and suspension or stay cable forces provide the external equilibrium of the bridge. Embedded tendons maintain the concrete stress profile within the allowable limits. The force, acting through each of these structural elements, has its own specific metrology.

Support reactions

In statically indeterminate bridges, support reactions are periodically measured to evaluate the developing forces during construction and, mainly, their *redistribution* throughout service life.

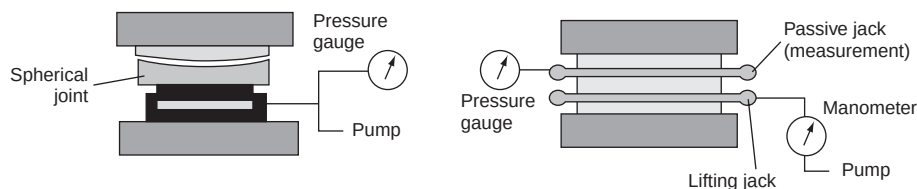


Figure 24.51 (left) Flat piston jack and (right) double flat jack for measuring support reactions: to put these devices in position, a 15 cm minimum clearance should be available between the deck and the bearing shelf; the bearing strength of the involved concrete surfaces must also be checked – it is also currently possible to install bearing systems directly equipped with permanent force-measuring devices

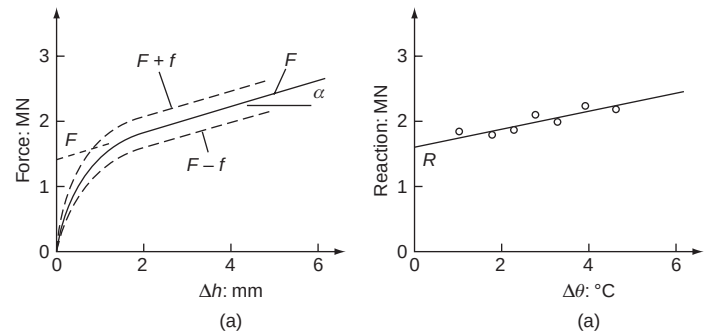


Figure 24.50 (a) Force–displacement curve giving a preliminary value of the support reaction; (b) final value after correction for temperature gradient

The measurement procedure consists of inserting a set of jacks around the existing bearing and using them to lift the deck. The force–displacement curve is plotted for the whole lifting–lowering cycle. The average curve thus eliminates frictional effects, f (Figure 24.50(a)). The first part of the graph marks the release of the original bearing. The second part, usually a straight line, represents the deck bending deflection; the slope gives the flexural stiffness. The reaction is deduced by negative extrapolation of the straight segment to zero (Chabert and Ambrosino, 1983). However, this value closely follows the cyclic variations of thermal gradients. In fact, the current metrology was the first to reveal the intensity of these effects on continuous bridge spans, sometimes attaining the equivalent of full live load configurations. Consecutive parallel measurements of both the reaction and thermal gradient are hence obligatory for at least 24 h to establish a relationship, and finally obtain the corrected reaction value at zero thermal gradient, as shown in Figure 24.50(b).

As only the *total* reaction can be measured, high-precision instruments are imperative to reasonably detect differential values, of greater monitoring interest. A 1% error already limits reaction measurements to the abutments, where loads are much reduced and variations more detectable. To this end, high-precision instruments were developed. Figure 24.51 shows two versions of the equipment now in use: the flat piston jack and the double flat jack.

In continuous spans, temperature-corrected reactions respond to dead load developments, differential settlement, stress profile modification in the deck (e.g. post-tensioning or differential creep) and specific or random consequences of various defects.

Support reaction metrology can thus now be used to detect

abnormal dead load distribution and explain certain observed defects. More basically, it can also verify or accordingly modify structural design assumptions.

Forces in external cables

The vibration method. During the construction of suspension or stay bridges and externally post-tensioned concrete structures, the individual cable forces are adjusted to achieve the required equality of stresses for the whole system under dead load. Over time, this uniformity may be disturbed for various reasons. Periodic checking is hence necessary for appropriate readjustment. Theoretically, tensile cable forces could be measured using the types of jack that initially applied them, but the operation and the equipment each have their drawbacks.

The vibration method offers a worthwhile alternative; it is simple, quick and has a wide application field (Robert *et al.*, 1991). The cable is modelled as a vibrating string of length l and mass μ per unit length, where the frequency f is related to the applied tension T by a well-known equation:

$$f_n = \frac{n}{2l} \sqrt{\frac{T}{\mu}} \quad (24.9)$$

where $n = 1, 2, 3, \dots$ is the rank of the harmonic.

The measured fundamental frequency and higher harmonics hence lead directly to the required force. The model is valid only when the cable flexural rigidity can be neglected, an assumption that is often acceptable, given the cable length and high tension. In practice, a 0.5% error is estimated if a linear relationship is confirmed between the order n of the successive harmonics and the corresponding measured frequencies. This linearity test is obligatory for each cable configuration to check the adequacy of the vibrating string model. If another 0.5% error is assumed for measurement instruments, the uncertainty in frequency would sum to 1%.

The equipment comprises an accelerometer, with its associated electronics, and a data analyser, immediately giving the successive frequencies. The accelerometer is mounted on the cable. Transverse oscillations are induced by either an abrupt change in tension or a supply tip hammer stroke, both at midlength, to favour the fundamental frequency. To reduce the error in the effective cable length l , the fixed points may also be equipped with accelerometers to check that no oscillations are detected on these vibration nodes. Measurements take a few minutes.

The overall accuracy is ~5% of the tensile force, given the uncertainties in the measured frequency, length and mass. However, if the cable force variation is monitored, only the frequency error remains, which is of the order of 1%, as already estimated.

Forces in embedded tendons

The crossbow method. The actual prestressing force, determined through the measured concrete stresses in a section, can also be evaluated by direct action on the tendons, using the crossbow method. The principle is based on the simple fact that

the effort necessary to deflect a tight rope is proportional to its axial tensile force. Figure 24.52 outlines the procedure, which is implemented after carefully clearing the adjacent concrete, duct and grout. The resulting disturbed tendon length, $2(l+x)$, is approximately 60 cm, which may hopefully decrease to 40 cm in versions to come. A controlled perpendicular force P , coupled with a displacement sensor, then successively deflects prestressing wires through a distance f , limited to 4 mm. Theoretically, the tensile prestressing force F in each wire may be deduced as

$$P = 2(F+k)(f/l) + K(f/l)^3 \quad (24.10)$$

where k and K are given constants.

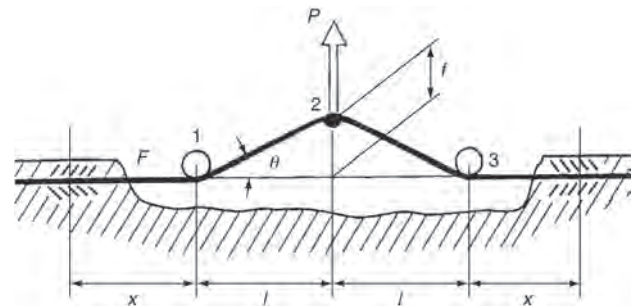


Figure 24.52 Principle of measuring the force in a prestressing wire

In practice, the parasitic effects of friction, flexural stiffness, overstretching and random bond failure necessitate prior calibration tests on simulation models in the laboratory, using the same type of wire and the exact disturbed length $2(l+x)$. A family of $P = g(f)$ reference curves is traced for different values of F . The curves established in situ could then be interpreted by direct comparison, leading to the actual prestressing force F in each wire. The data should undergo statistical treatment before acceptance as reliable experimental results.

The incremental drilling method. Another way of measuring the tendon force is through minute incremental drilling, in the wire, of a hole, 1.5 mm in both diameter and maximum depth. The strain release, measured by strain gauges on either side of the hole, leads to the total stress, including the residual part owing to the wire-drawing process. This part can be estimated either by identical parallel tests on unstressed samples or by referring to the manufacturer's records or other sources. It thus gives access to the applied stress, hence the force.

Local behaviour of sections and discontinuities

Crack geometric survey under dead load. If correctly interpreted, cracks not only indicate the positions and directions of excessive stresses, but can also be external signs of the actual structural behaviour. In concrete structures, for instance, visual inspection by experienced staff should already be used to distinguish between active structural cracks, caused by intense forces or weak construction joints, and superficial cracks, caused by insufficient curing.

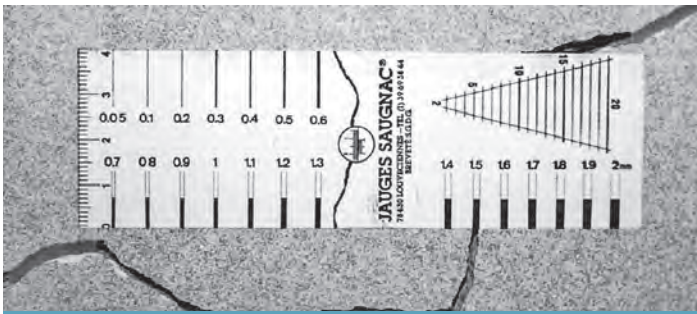


Figure 24.53 Specific scale for estimating crack widths (photograph courtesy of LRPC Bordeaux)

At successive inspections, possible developing cracks are traced on both the structural drawings and the bridge itself, each group with different colours and consecutive dates. Crack openings are marked down to one-tenth of a millimetre (Figure 24.53).

To check whether or not a crack goes through the structural element, coloured water or coring are used, the latter being more reliable but complicated in the presence of curved discontinuities.

Recording detailed data on the structural drawings facilitates interpretation and diagnosis, but does not dispense analysts from making visual inspections on-site.

Local response to external loads

The previous investigation methods, describing the general structural behaviour, should be locally completed under external loading by a detailed analysis at certain positions of particular interest. The local response to loading is generally characterised by multidirectional strains on continuous material surfaces and by relative displacements across cracks, opening joints or other discontinuities.

Strain

Under loading, strain is measured at several points of the structure not only to check deformability assumptions but also to calculate the corresponding stress variations. In linearly elastic media, a well-known strain–stress relationship is used, involving the Young modulus, E , and Poisson's ratio, ν . In the frequent cases of simple uniaxial loading, one strain direction is usually sufficient. Two orthogonal strain measurements give the respective stress variations along these same axes. Three directional strain measurements represent the general case of plane stress variation, determining, in addition, the principal stresses and their directions, as well as the tangential stresses.

While the deduced stress variations are usually reliable for steel, they may be less so for concrete, owing to changing elastic properties and surface quality. The absence of a time parameter in the elasticity equation, as well as frequent drift in strain gauges, does not allow the interpretation of long-term creep effects on stress. By definition, relaxation cannot

be monitored at all. Strain measurements are hence limited to load-induced instantaneous variations.

For most cases, the required strain measurement sensitivity is approximately 50×10^{-6} for steel and 2.5×10^{-6} for concrete, corresponding to 10 and 0.1 MPa stress variation accuracy, respectively.

Relative movement across discontinuities

Measuring the load-induced variation of a crack or joint opening is often useful for appraising overtension in tendons or reinforcement bars crossing the discontinuity (Figure 24.54). However, overtension can be reasonably estimated only by associating constitutive laws on steel–concrete bonding. For reinforced concrete, these laws are relatively well-known. For post-tensioned concrete, assumptions depend on grouting and duct quality. Hence, for a reliable estimation of overtension, prior non-destructive checking is necessary. The sensitivity required for measuring relative displacements is 10^{-3} mm for post-tensioned concrete and 10^{-2} mm for other types.

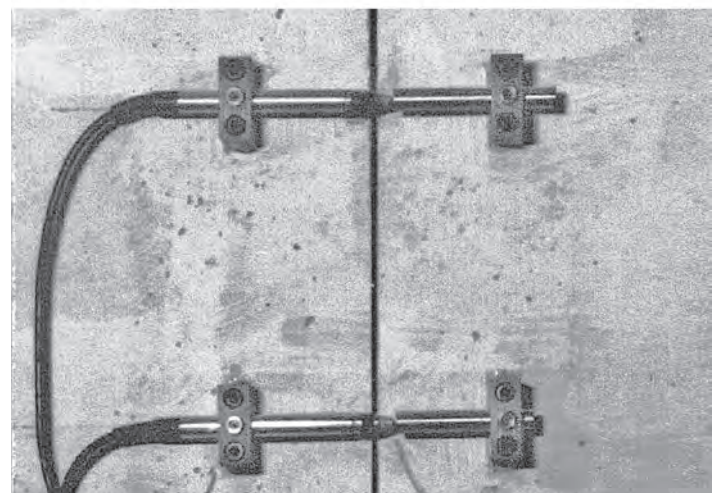
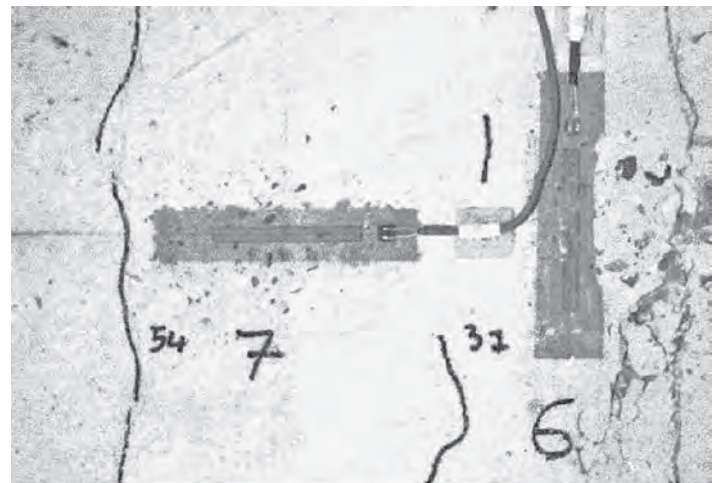


Figure 24.54 (top) Orthogonal strain gauges and marked surface cracks; (bottom) displacement sensors across a joint (photograph courtesy of LCPC)

Instruments

The main types of strain or displacement measuring device are as follows.

- Electric resistance strain gauges, with 1×10^{-6} overall sensitivity, are used on steel and concrete surfaces, and sometimes embedded in concrete. Drift is the main drawback.
- Vibrating wire strain gauges, with 1×10^{-6} sensitivity, which have no significant drift, are based on the oscillation frequency variation of a tight wire as a function of length. They were initially used embedded in concrete, but are now also used on the surface. They are robust but need specific equipment for automatic measurement.
- Mechanical displacement gauges, with 10^{-3} mm sensitivity, mainly on a 100 mm base, are used to measure steel strain and concrete crack-opening variation.
- Electric displacement transducers, mainly linear variable differential transformers (LVDTs), with 10^{-3} mm sensitivity, have no particular drawbacks.

Coupled strain–displacement measurements

In cases where cracks or segmental joints do open under service loads, tendons are first checked both for corrosion, using NDTs, and for fatigue, using existing numerical methods and other references. The discontinuity can sometimes be fully instrumented on both the concrete and the steel surfaces to estimate the local flexural capacity (Chatelain *et al.*, 1990; Godart, 1987). At several levels of the concrete section height, displacement sensors are placed across the discontinuity and respectively aligned with long base strain gauges glued on one side at a minimal yet sufficient distance from the edge to avoid local parasitic effects (Figure 24.55). Under a gradually applied decompression moment M , when the crack is still closed, both the sensors and the strain gauges usually vary linearly and proportionally (Figure 24.56). When the crack opens, at a

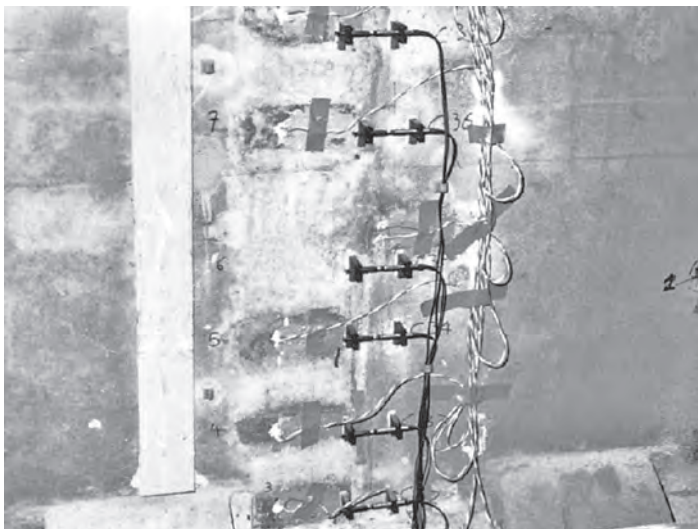


Figure 24.55 Strain gauges and displacement sensors, coupled all along the joint height

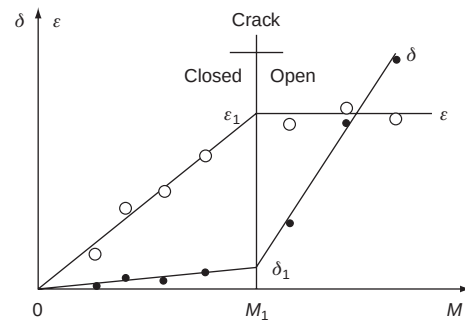


Figure 24.56 Joint openings, δ , and edge strains, ϵ , under a bending moment, M

moment M_1 , the sensors show a much higher extension rate, while strain gauges no longer respond. Moment M_1 is hence an estimation of the residual flexural capacity. To determine the steel overtension, small strain gauges are sometimes glued on the exposed tendons crossing the discontinuity.

Some guidelines for local field measurements

The success or failure of past field investigations has led to certain guidelines.

- *Define the framework of the formulated problem.* Through a detailed discussion between the design office and the experiment task group, both should
 - recognise the difficulties of field metrology and work out a realistic programme
 - know the experimental stakes and provide optimum instrumentation.
- *Determine the order of magnitude of the measured quantities.* Prior calculation of magnitudes, with high and low assumptions, may facilitate the choice of the best-suited technology, reconcile accuracy with economy and fix the operating limits during tests.
- *Use all necessary means.* While optimum cost remains a part of a carefully prepared investigation, a blind economy reflex has often proved too expensive in the end.

Hence, it is necessary to

- provide an adequate number of measurement points
- use parallel available methods for confirmation
- acquire complementary data for correction, especially regarding thermal cyclic variations.

Direct stress measurement and monitoring in concrete bridges

In the assessment of concrete or masonry bridges, the actual stress profile is a major parameter for estimating material and structural capacities. Under dead load, stress in concrete is a combination of an internal component, inherent to the material,

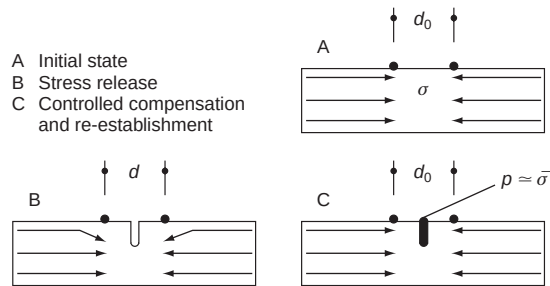


Figure 24.57 (a) Stages of direct stress evaluation

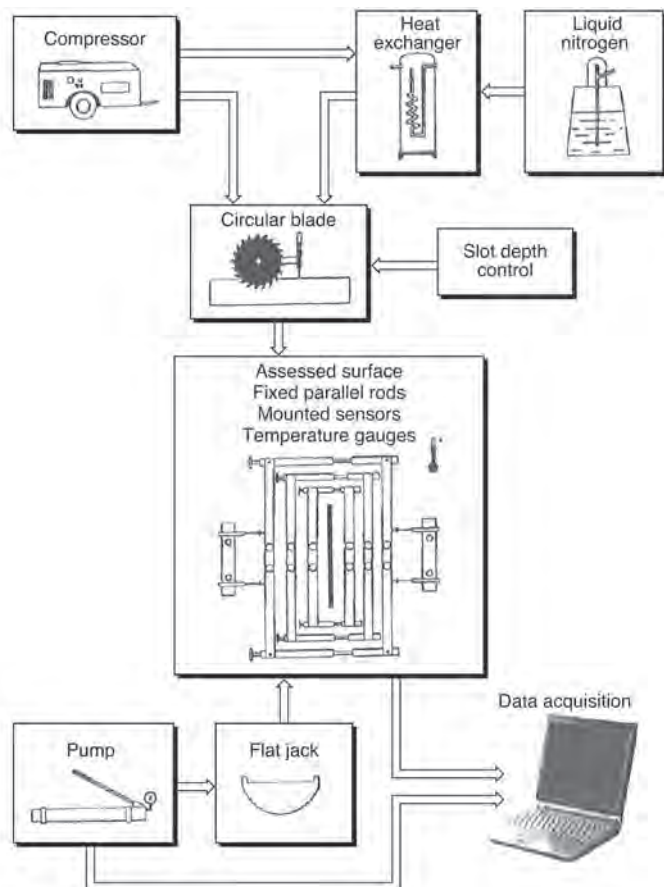


Figure 24.57 (b) general set-up, including specific prototype instruments, for the direct evaluation of stress and its components; this applies to concrete, masonry, timber and mechanically comparable materials

For such cases, stress cannot be evaluated through conventional strain measurements. In the absence of cracks or clear discontinuities, and with no prior knowledge of the actual absolute stresses, test loading may become arbitrary and destructive, undermining its own purpose.

In a fundamentally different approach for treating these cases, the partial stress release method was developed to measure the total stress *directly* and deduce its components (Abdunur, 1993, 1995). Indirect methods have also been developed (Ryall, 1996).

The release method

As shown in Figure 24.57(a), the release method is a local and partial release of stress, followed by controlled pressure compensation. Figures 24.57(a) and 24.57(b), respectively, present the principle and the instrumentation. In practice, a reference displacement field is first set up on the assessed surface. A slot 4 mm wide is then cut in a plane normal to the required stress direction (Figure 24.58). Finally, a special very thin flat jack

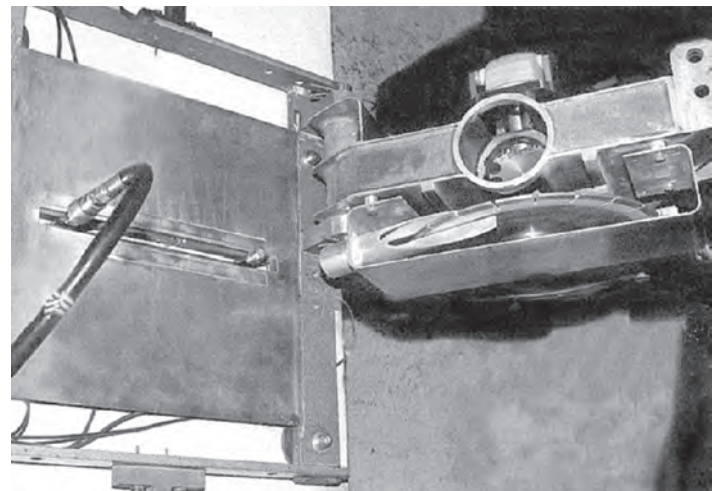


Figure 24.58 Cutting and pressurising a slot (photograph courtesy of LCPC)

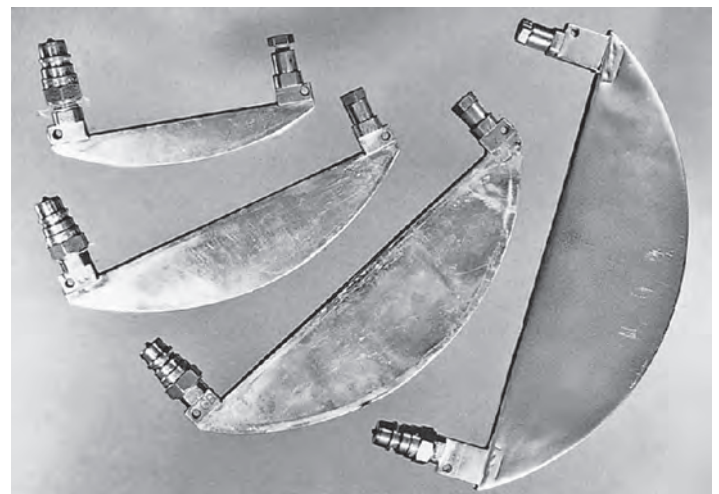


Figure 24.59 Set of flat jacks (photograph courtesy of LCPC)

and an external one, caused by thermal effects and applied forces, including prestressing.

If the total stress in a structure can be directly measured, it may immediately be compared with the allowable working values to estimate the material's safety margin during service life, eventual strengthening or partial reconstruction.

If the applied (or external) stress component can be isolated, it may lead to the real mechanical reserves in complex structural configurations, such as ageing or damaged bridges with uncertain elastic properties and various degrees of static indeterminacy.

(Figure 24.59) is inserted in the slot and used to restore the initial displacement field. The value of the cancelling hydrostatic pressure gives the average total compressive stress normal to the slot. In the same way, with the same accuracy, *tensile* stresses are determined by negative extrapolation, as discussed later. The operating depth range is 80 mm.

Theoretical and technological studies have enabled miniaturisation and accuracy to be reconciled. The maximum error margin is 0.3 MPa. The measurement is direct in the sense that the same physical quantity is involved (pressure for stress) and that none of the elastic properties of the material is needed. These are, in fact, determined in the process. Unlike conventional methods, no test loads are used. Post-measurement remedial techniques can restore the initial mechanical and aesthetic state of the investigated medium. The release method can thus be classified as globally and locally non-destructive.

Average compressive and tensile stresses

Compression is measured by the cancelling pressure, as illustrated by the *lower* straight line in Figure 24.60. This is the jack pressure restoring the displacements to zero. In rare cases of damaged media, the compression line may be slightly curved and a special procedure leads to the correct stress.

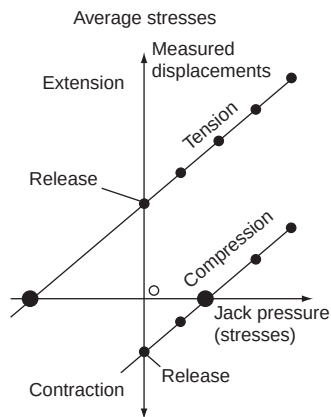


Figure 24.60 Access to average stresses by partial release and pressure compensation

Tension can be estimated through measurements of supplementary forced extensions resulting from additional applied pressure increments, then by negative extrapolation down to zero displacement. This is illustrated by the *upper* line in Figure 24.60. For concrete, the relationship is usually linear, confirming its elastic-brittle tensile behaviour.

The slope of each line reflects the medium's deformability or the *inverse* of the actual in situ elastic modulus.

Separation of stress components

As already mentioned, after evaluation of the average total stress estimating the material's safety margin, it is at least as

important to correctly extract the *applied* (or external) stress reflecting the structural capacity. This requires

- precise determination of the total stress profile as a function of depth
- detailed knowledge of the internal stresses associated with the material.

The applied stress can be obtained by subtraction.

To obtain the *stress profile*, the operations already described and shown in Figure 24.61 are repeated at increasing slot depths, z_i . Figure 24.57 summarises the model-aided analysis of the acquired data. The required total stress profile, defined by the column matrix $[\sigma_j]$, can be determined by solving the linear system

$$[C_{ij}][\sigma_j] = [\delta_i] \quad (24.11)$$

where δ_i are the successive surface displacements measured for a slot attaining depth z_i and C_{ij} is the compliance matrix for a reference medium, adjusted to each case using the *actual* medium's response to the flat jack pressures. The actual in situ modulus is thus determined in the process.

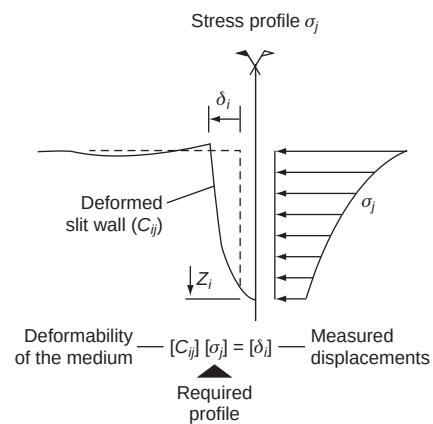


Figure 24.61 Access to the total stress profile through a proposed model

Internal stresses in concrete are mainly due to *drying shrinkage*. Using the release method itself, shrinkage was found to induce high superficial tension, depending on the loading and exposure histories (Abdunur *et al.*, 1989). It falls rapidly with depth and changes to a moderate compression throughout the core. Like all other internal effects, shrinkage stresses are balanced over the section. In the presence of applied external forces, the superposition principle has proved to be valid. Hence, given (a) their specific distribution, (b) superposition validity and (c) auto-equilibrium, the shrinkage eigenstresses can be identified and isolated from the measured total stress profile. A striking similarity in shape is often observed between the obtained shrinkage stress profile and the percentage water loss distribution measured or determined by γ -densimetry (Figure 24.62 (a) and (b)) (Abdunur, 1993).

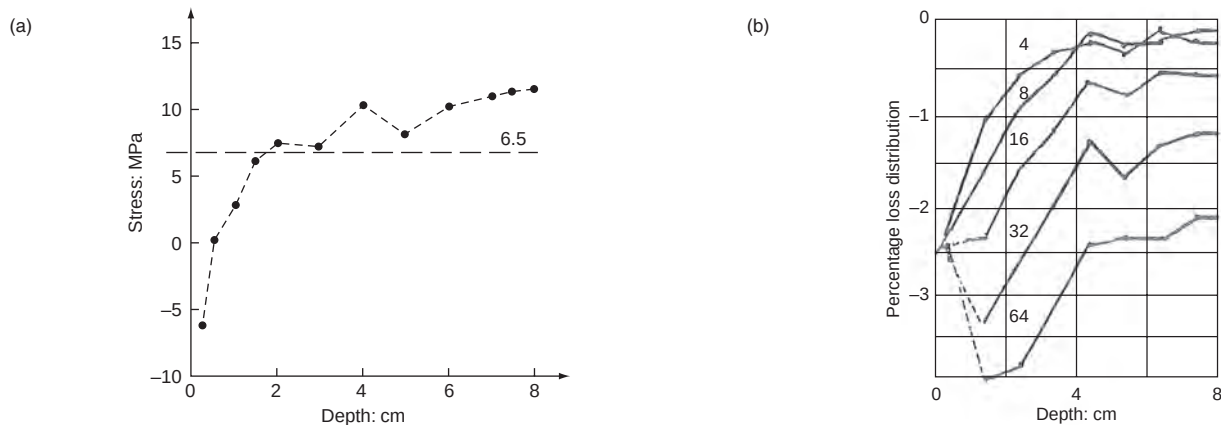


Figure 24.62 (a) Total stress profile combining shrinkage stresses in equilibrium and a 6.5 MPa uniform prestress (b) Percentage water loss distribution measured by γ -densimetry, at different ages of concrete, across the same concrete thickness as in figure (a)

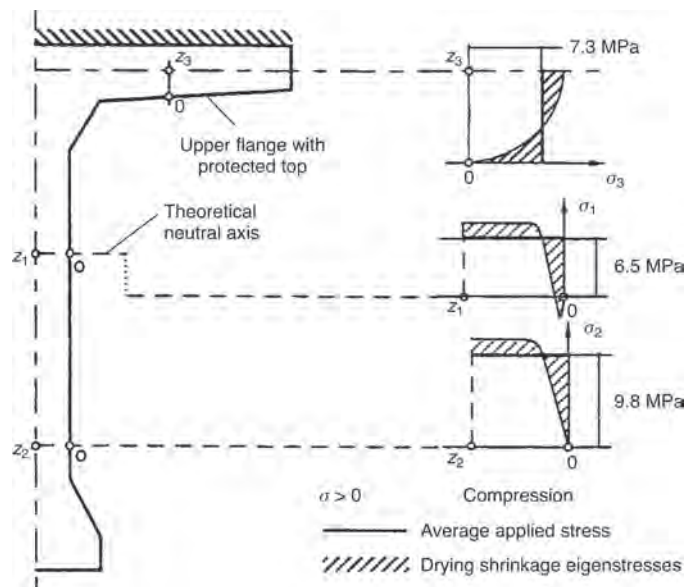


Figure 24.63 Stress distribution across beam wall thicknesses

Example. For a post-tensioned slab under dead load, Figure 24.62 gives a typical total stress profile across the thickness, traced from the stress release data after processing using the numerical model. The profile combines prestress and shrinkage effects. As shown, these two components are separated using the three aforementioned properties of shrinkage eigenstresses.

Stress redistribution and prestressing force. Further to this example, the stress profile across the thickness is confirmed at three fibre levels of ageing post-tensioned T-beam sections (Figure 24.63): one in the upper flange and two in the web, at the theoretical neutral axis and nearest to the lower flange containing the tendons. The three average compression values, represented at their respective fibre levels (Figure 24.64), show a highly curved stress profile over the section height, fundamentally different from the conventionally plane distribution assumed or admitted by many codes of practice. This is probably the mechanical effect of differential non-linear creep and shrinkage, loading the bulky elements and relieving the thin ones (Abdunur, 1996).

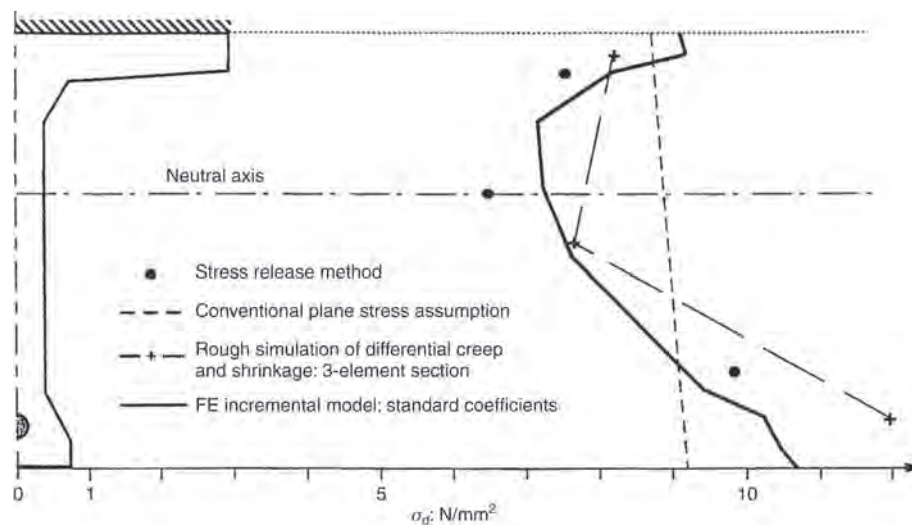


Figure 24.64 Measured and calculated dead load stresses in ageing post-tensioned concrete beams

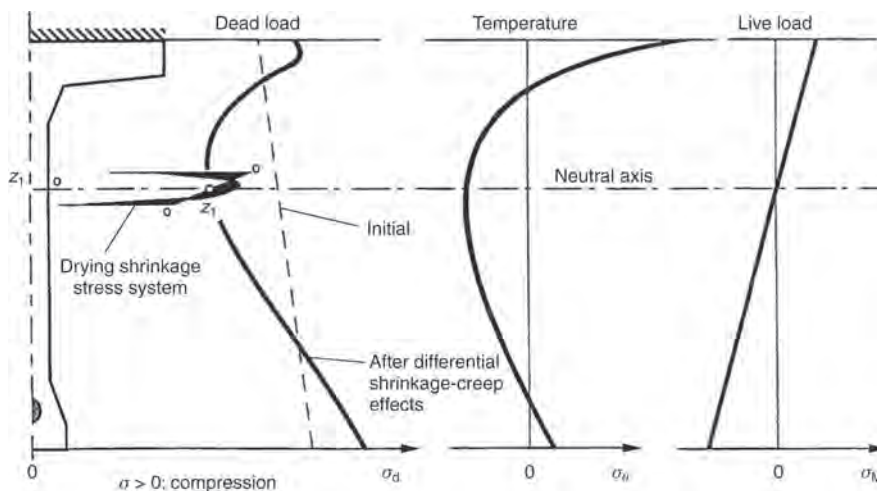


Figure 24.65 Main components of stress in ageing post-tensioned concrete bridges

Recapitulation of acting stresses

The dead load stresses, their double redistribution owing to concrete differential creep and shrinkage, the thermal gradient effects and live load stresses are summarised in Figure 24.65.

Contribution of the release method to evaluating time-dependent prestress loss in concrete bridges

In a detailed assessment campaign, direct stress measurements were made on 12 selected post-tensioned concrete bridges, 22

to 45 years old, with initially laboratory-estimated relaxation ρ , as well as friction and wobble coefficients, μ and φ .

These bridges were strictly identical in all other respects, particularly in their dimensions, intact constituent materials, design methods, construction procedures, inspection and maintenance policies, all in addition to a main factor in common: their environment.

Using the release method, concrete stresses were directly evaluated to deduce the actual prestressing force. Using the crossbow method, steel stresses were measured in a few nearby undisturbed sections. The stress values obtained by both methods were consistent (Figure 24.66).

In a very homogeneous group of N bridges, solely differing in age, the N respective stresses, all measured at a given date, may be assumed equivalent to measurements taken on one single typical bridge, at the same N consecutive dates of its service lifetime. In this case, the 12 corresponding stress values can thus form a synthesis curve representing an experimental, time-dependent loss of prestress.

In Figure 24.66, the directly measured concrete stresses are converted to forces over the sections and expressed as a percentage of the initial prestress force F_0 at the anchorage, found in the records of every bridge. These acquired experimental data, curve (a), lie between two theoretical envelop curves (b), stemming from two constitutive models, briefly described next. Curves (b) guide the extrapolation of the measurement points towards the past and the future. In return, a further scrutiny of reliable experimental data may help to verify, or accordingly modify, the assumptions adopted in the theoretical models.

Theoretical models of prestress loss

After a preliminary analysis leading to significant recommendations, two main frameworks of prestress loss estimation, the *IPI* and the *BPEL* models, were proposed at successive periods of bridge construction development.

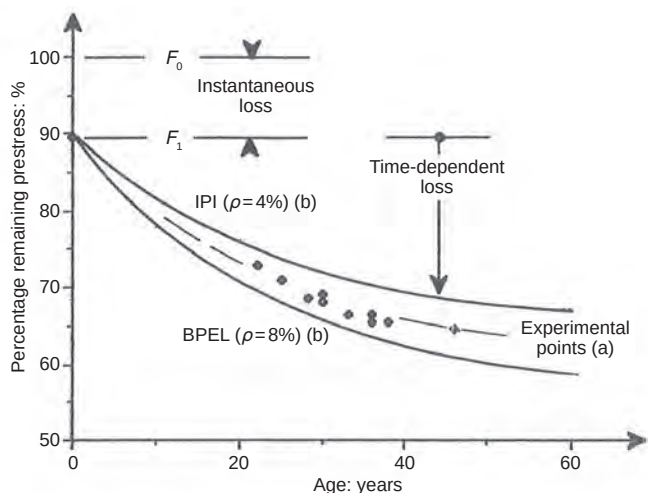


Figure 24.66 Evaluation of time-dependent prestress loss in concrete bridges (experimental and theoretical data, showing the contribution of the stress release and crossbow methods): curve (a) is the directly measured residual prestress at midspan, on 12 identical bridges, differing only in age, with initially laboratory-estimated steel relaxation ρ and friction coefficient μ ; curves (b) are the corresponding theoretical envelop curves of prestress loss, obtained from two predictive models, with different basic assumptions and values of ρ , which guide, against time, the extrapolation of measured prestress loss curve (a) and also help to interpolate the relaxation ρ for the steel actually used on-site

In the IPI model, based on the concept of allowable stresses, a group of constitutive equations accounts for the following sources of loss:

- concrete shrinkage, depending on the mix and thickness, atmospheric humidity and time-dependent strain coefficient
- concrete creep, as a function of the initial elastic strain, atmospheric humidity, time-dependent strain coefficient and other constitutive factors
- steel relaxation, depending on the initial steel stress σ_i , strength R_g and percentage stress loss ρ at 1000 and 3000h, in a standard relaxation test.

The main assumption in the IPI model is a straightforward cumulative effect of these three factors, instead of their interaction.

The BPEL model, which is more elaborate, is based on the serviceability and ultimate limit states, more explicitly formulated further, in a separate approach. Compared with the IPI model, there is hardly any difference regarding concrete shrinkage. As for concrete creep, in a new BPEL model version, the effect of atmospheric humidity is more accurately formulated in the basic law. Conversely, an additional but different law was introduced for creep recovery when the acting forces decrease or disappear.

Concerning steel relaxation, the BPEL model kept the same IPI parameters, but in a slightly different formula. As illustrated in Figure 24.66, it is less optimistic in defining the behaviour of the materials used in the bridges under consideration.

The basic assumption of the BPEL model is the partial interaction between concrete creep and steel relaxation. This is more realistic than taking a simply cumulative effect.

A practical consequence of the monitoring campaign

Having estimated the *remaining* structural reserves of a bridge, using these procedures, the next step is to update its *required* total capacity, pending its needs for strengthening. To this end, load models, such as Eurocode 1 (BSI, 2003a), are proposed. The difference between the required and remaining reserves leads to its needs for strengthening. In the present case of a post-tensioned concrete bridge, additional *external* prestressing must be provided.

At a later stage, the concrete stresses can again be measured to check the effectiveness of the operation. If the prestress still needs readjustment, a corrective action on the external cables should always be possible, to maintain the optimum reserves.

Stress monitoring and readjustment: an example of the smart structure's approach

After the instantaneous evaluation of stress, a further and equally important step would be to monitor its cyclic and irreversible variations (Abdunur, 1992). In terms of the release method, stress monitoring may be defined as the time-dependent pressure of the flat jack, constantly ensuring the same local normal strain development that would occur if the medium were still free of any inclusion. The procedure is demonstrated in Figure 24.67. Once the initial stress is determined, the sensors over the flat jack will form the active system, measuring the time-dependent response of the surface displacement field in the artificially reconstituted continuum. Along the longitudinal axis of the active system, a second group of identical sensors, forming the reference system, is placed slightly beyond the range of the jack to follow up all displacement field variations in a close but undisturbed section. A specific regulator then automatically

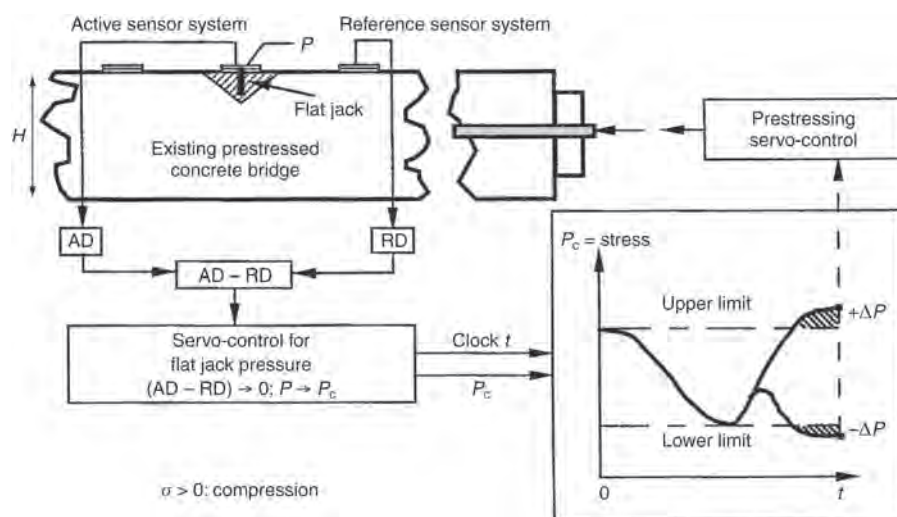


Figure 24.67 Processes of stress monitoring and eventual readjustment

adjusts the flat jack pressure so that the displacement variations measured in the active system always equal those detected in the reference system. The time-dependent pressure, p_c , thus obtained, gives the corresponding normal stress variation in the equipped section. Monitoring can be remote-controlled and last for the lifetime of the structure.

In certain configurations, such as prestressed concrete bridges, direct stress monitoring may lead to corrective measures. If the flat jack pressure p_c clearly indicates a lasting stress outside the allowable limits, as indicated in the two shaded areas of the graph in Figure 24.67, a remote-controlled selective action on external tendons may restore it to its optimum level. As already hinted through its title, this example points to the much broader field of *smart* structures and their active control, outlined next.

Stress is thus gradually acquiring metrological independence from strain, jointly improving the means of evaluating structures and exploring constitutive laws.

Commentary

This section presented, separately, investigation methods regarding materials and those concerning structure, where the distinction is not really consistent, since the final evaluation of the state of a bridge is necessarily global. This highlights, however, two parallel lines of approach that cannot always coincide: material properties, mainly studied in the laboratory; and structural analysis, mostly practised in the design office. This separation also reflects the current modelling limits in coupling material constitutive laws with structural analysis, to predict and describe their interaction on a more realistic basis.

Conversely, the methods presented must be selected on the basis of experience-based judgement. Many of them have to be used on distressed bridges because defects have been detected too late. Their cost is very high compared with those of a 'normal' management policy. Hence, it is never too early for a first detailed inspection, for other periodic ones to follow and for appropriate action as soon as flaws are discovered.

In spite of their present performance, the available investigation methods need continuous development, insofar as research and technology transfer can offer, to better face the steadily broadening spectrum and growing complexity of cases in store.

Finally, as summarised in the next section, and thanks to the efforts exerted in a wider scientific and technical framework, a permanent monitoring and active control system may now advantageously replace or complete the present conventional approach of periodic field investigations and maintenance, thus avoiding repeated equipment transportation and delicate laboratory tests.

Smart materials and structures

The permanent stress monitoring and readjustment process, described earlier in this chapter and illustrated in Figure 24.67, offers a specific example of a much wider scope, the *smart*

systems concept, a recently advanced technology, which is steadily gaining ground in many fields, including structures.

Smart bridges are *permanently monitored* and, in real time, actively reactive to a detected abnormal or unfavourable event.

The event may be a deviation from the established reference values of the state parameters defining the material and structural safety, as well as the required behaviour of the bridge under the applied forces and environmental conditions. Among many other possibilities, the deviation may suggest a degradation of the intrinsic material properties or a state beyond the allowable limits of span deflections, support reactions and tendon forces, as well as stress, strain or temperature profiles.

This elaborate many-sided process stimulated research and innovation at different levels. For its implementation in the field of structures, the process mainly requires

- evolving, high performance adaptive materials, with both sensing and actuating constitutive capacities, enhancing an appropriate response to mechanical and environmental factors
- at the structural level, an integrated, computer-aided, remote-controlled apparatus to permanently accomplish the following tasks:
 - monitoring of basic state indicators, through a network of incorporated sensors
 - remote data acquisition and processing, detecting abnormal deviation from reference values
 - analysis and interpretation, assessment and conclusion, generally aided by artificial intelligence but constantly subject to human final decision
 - action through remote control, with suitable incorporated actuators, maintaining or restoring the reference values of the state parameters involved in the material, structure, support contact points and environment
 - verification of the effectiveness of the operation through the same data processing.

Although ideal for installation in new constructions, smart systems must equally benefit existing bridges, which form almost all the world's stock and greatly need improved monitoring and optimum solutions to various problems arising throughout their remaining service life.

Two parallel smart bridge technologies are thus emerging, with common and complementary tools and procedures. Integrated instrumentation is usually embedded during construction and may be surface bonded or inserted for existing structures, whether still intact, damaged or repaired; hence, the additional term *smart maintenance and repair*.

Smart technology transfer from the aerospace industry has been particularly valuable to civil applications. With structural dimensions one to two orders smaller in magnitude, and subject to strict performance-durability imperatives under severe environments, the aerospace industry has offered high-quality adaptive materials for optical fibre sensing systems and developed structural active control, as well as artificial intelligence.

Smart materials

A daily comfort many now enjoy is permanently stabilised light intensity passing through windowpanes, windscreens or even sunglasses, in spite of the fluctuating power of the incoming rays. Modern loudspeakers continuously maintain the quality and optimum volume of the outgoing sound, whatever the disparity of the source. These advanced filters, with sensors and actuators perfectly assimilated into the matrix, may be qualified as smart. They respond to inconvenient intensity levels by modulated absorption.

More actively, countermeasures by external actuators can now enhance the soundproof characteristics of panels by real-time emission of identical counter-waves, neutralising the incident source. The same basic idea of prompt active damping is successfully applied in vital sectors of industry, such as earthquake-resistant construction and accelerated mass production of delicate high-precision instruments.

Throughout history, employed materials moved on from strictly natural elements, used as such, to highly designed alloys and compounds, evolving from passive to active, then to actively reactive behaviour, approaching certain functions of a biological model. These new, ever-improving composites permanently monitor their loading and environment and immediately react to unfavourable changes by modifying their shape and position.

In structures made entirely from composites, the homogeneous incorporation of optical fibre sensors confers additional smart functions regarding the material itself and an optimum control of its properties.

In the meantime, conventional materials have made their own progress, both intrinsically and in association with new designed alloys. Concrete, for instance, passed from primitively plain, to simply reinforced, to a high-performance prestressed material, with tendons and optical fibre sensors running parallel throughout the span, to detect and restore the preset reference axial force, supplying greater mechanical reserves against the various potential cracking hazards.

Smart monitoring tools, completing conventional equipment

Optical fibre sensors, such as fibre Bragg grating (FBG) systems, are increasingly used to monitor strain, force and temperature variations in structures. Under intense laser beams, the input signal splits into transmitted and reflected parts. Monitoring the Bragg wavelength shift of the *reflected* signal leads to the required mechanical and thermal variations, given the material refractive index and the grating pitch. According to certain studies (Casas, 2004), FBG systems proved accurate, resistant to aggressive media and adjustable to different stages of the structure's service life.

Inductive or capacitive displacement sensors are well-adapted conventional instruments to smart technology.

Conversely, while electric resistance strain gauges remain essential for studying instantaneous mechanical behaviour,

they often suffer significant drift that limits their suitability for reliable long-term monitoring of concrete structures on-site. The inclusion effect may add a local error when these gauges are embedded in concrete. Their optimum length and installation should reconcile a sufficiently representative response with minimal interface field disturbance.

To monitor embedded reinforcement, gauges installed in a mini-duct, machined inside the bars, are already applied in research and technically justified in specific cases but, economically, they may be hard to defend for regular, large-scale use on-site.

Nevertheless, conventional equipment remains indispensable and goes through specific technologies, to monitor different forces. Here are some reminders.

- The *total* support reaction is measured, at a contact bearing point, by inserting a flat piston or double flat jack, then plotting the force–displacement cycles. High precision is essential to detect *differential* values, of greater monitoring interest.
- In suspension, stay or externally post-tensioned bridges, the vibration method has proved suitable for following up individual cable forces, if the flexural rigidity can be neglected.
- Force gauges can monitor prestress at the extremities of internal unbonded tendons.

Examples of actuators for smart structural response

Continuous concrete bridge spans are often subject to a time-dependent, thermal or viscoelastic moment redistribution, inducing serious disturbance in the stress profile. To counteract this detrimental mechanism, direct stress monitoring, as explained in the previous example, first supplies the necessary data and then *verifies* or *guides* the required response. Two smart options, with different actuators, have been considered and successfully applied, one acting on the *relative* support levels, the other acting directly on the stress profile.

- Support bearings, equipped with appropriate sensors and actuators are closely monitored and can, instantly and accordingly, vary their relative levels of bearing contact with the superstructure. The process continues smoothly until the support reaction forces coincide again with their initial reference values, hence restoring the original optimum bending moments and corresponding flexural stress profiles.
- In prestressed concrete girders, another option is made feasible by using actuators exercising a coordinated action, on external cables or internal unbonded tendons, restoring the stress profile to its optimum level, guided and corrected by direct stress monitoring.

This stabilising action enhances structural integrity, and hence service life, in frequent cases of statically indeterminate structures subject to asymptotic or periodic variations caused by the material or the environment.

The smart systems concept is thus evolving at an accelerated

rhythm, on the broad spectrum of science and technology. In years to come, smart systems are more likely to become the rule, rather than a persisting exception.

Residual strength evaluation

Besides the standard assessment procedures periodically applied to the whole stock, certain bridges come under further scrutiny when classed as either *structurally deficient* or *functionally obsolete*. The former is a danger and requires *repair* to eliminate detected flaws and restore the initial design level of service at its construction date. The latter is a growing nuisance and suggests optimum *strengthening* to cope with updated traffic systems involving additional lanes, greater standard loads and exceptional convoys. However, it surely happens that the same structure may simultaneously need both operations.

Before any operational move, the evaluation of the actual load-bearing capacity of a bridge is a particularly essential task to start with, requiring a thorough assessment of its structural safety. The process mainly involves appraising the physical and mechanical states with the highest possible accuracy.

With the great diversity of the bridge stock and with the often-changing design and construction rules, it is difficult to set up, in advance, specific assessment procedures covering the very wide range of individual cases, sharply contrasting in age, span, constituent materials, exposure to traffic and environment. The process takes additional dimensions when the structure faces existing or potential degradation.

The intricate notion of structural safety involves a reliable, accurate acquisition of deterministic and random data.

In this context, the following sections will present concepts and guidelines of a general methodology, where an experienced engineer should adjust each assessment phase to the given bridge, in the light of related available data.

Stages of structural appraisal

Generally, the safety evaluation of a bridge has two successive phases: preliminary diagnosis and recalculation, with an intermediary field investigation programme.

Preliminary diagnosis

This is a brief assessment based on existing documents and first inspection results on the main structural members of the examined bridge. Conscious of the various possible deterioration mechanisms, recapitulated in the first section of this chapter, the engineer puts forward a hypothesis about the probable causes and consequences of the observed or suspected damage, ordering urgent action if need be. Provisional safety precautions may include traffic limitation or closure, preventative emergency scaffolding and a selective barrier system to concerned vehicles. Exceptionally, cautious loading tests may

already be necessary to estimate the residual range of elastic reversible behaviour in a potentially damaged structure.

Investigation programme on-site

Based on the first-phase conclusions, and using investigation methods such as those already described, this programme should yield completely updated information on the

- exact dimensions and drawings of the bridge
- actual values of the material properties

with the maximum possible accuracy.

The acquired data should lead to a clearer comprehension and definition of the actual structural system.

The complete investigation results and comments should be carefully assembled in a report to serve throughout the next main phase.

Recalculation

This is a detailed analysis based on the results of the aforementioned investigation programme. It has a double objective:

- determination of the *actual state of stress* of the structure, while strictly considering the influence of any observed or suspected flaws
- evaluation of the *safety margin* with respect to failure or to certain irreversible limit states.

The first objective amounts to a *reference state*, characterising

- the behaviour of the bridge under normal service conditions
- the degree of eventual fatigue damage, whether in totally metal structures or in tendons and rebars.

Stresses are determined, under dead weight and service loads, in accordance with the conditions summarised in the section on recalculation of a distressed bridge.

Recalculation starts with several preliminary steps.

- Pass from a brief to a thorough examination of the bridge design, construction and maintenance records.
- Search for the maximum information available on
 - current loading systems and their past history, including authorised exceptional convoys and other unauthorised, rarely recorded, aggressive loads
 - differential settlement at the reactions
 - stress redistribution in continuous decks, owing to differential settlement or creep, often roughly compensated by imposed counterstrains through relative level readjustment of the supports
 - damage at and around impact points, owing to different external bumps.

- Draw up a clear and brief table summarising the calculation assumptions and verification rules in force at the construction date, as well as those to be adopted for recalculation.
- Use all current improved knowledge available on material constitutive laws, for example, creep and shrinkage, thermal gradients and fatigue.

The main importance of the recalculation procedure is that it justifies the many-sided preparation given in the following section. The outcome of the process should lead to a more accurate diagnosis and a reliable quantitative evaluation of structural safety.

Different solutions may then be considered from technical and economic viewpoints to facilitate a decision on the future of the bridge: strengthening, repair, reconstruction or even conservation at the existing state, with particular restrictions, maintenance and monitoring.

Preparing for recalculation

The preparatory steps include a theoretical approach through structural analysis, backed by an experimental study providing the actual values of certain involved parameters.

Structural analysis

The actual structural response is analysed, using the conventional strength of materials or numerical methods, usually assuming a linear elastic behaviour. However, non-linear models do help explain local and even general problems, such as differential concrete creep and shrinkage. Other specific models, dealing, for example, with damage analysis and fracture mechanics, are required for cases of fatigue cracking in steel or composite bridges. To be reasonably representative, the calculation must take into account the actual accurate geometry of the structure, the consecutive construction phases and the eventual degradation of the resisting constituent materials, whether passive or active.

Actual geometry of the structure

Sonic tests determine the thickness of slabs or other elements and, indirectly, the weight of equipment. For old bridges with non-existent records, a complete geometric survey is required, using advanced numerical interpretation of photographic plates. Within the concrete, the tendon and reinforcement positions affect the applied internal forces, hence the section resistance. If a difference between theoretical and actual positions is suspected, especially in the tendons, a check by radiography or radioscopy will be necessary.

Construction phases and unexpected events

The analysis of an existing bridge should necessarily consider *all* the construction phases that may have modified the structural system or permanently acting forces, with respect to the

initial design. Unfortunately, available records do not always contain the entire information and rarely mention unplanned operations carried out on the resident engineer's initiative, without notifying the design office.

Such operations often involve imposing deflections and strains to establish or restore the required geometric compatibility between adjacent structural members. Uncontrolled parasitic stresses are thus induced, locally disturbing the structural reserves.

Another example is the use of less conforming prestress ancillary elements, such as insufficient duct diameters. Forcing tendons into narrower ducts amplifies friction losses and remains seriously misleading if not reported or else load limiting if not compensated.

Material degradation

Recalculation should account for possible weakening of the constituent materials. Concrete may degrade locally through various physical or chemical attacks while steel sections may suffer a substantial reduction through corrosion.

In concrete bridges, cracks affecting section thicknesses also modify the distribution of forces in statically indeterminate configurations. In reinforced concrete, it is very difficult to evaluate the state of the steel stress. Conversely, in seriously damaged steel decks, section loss can be estimated by local surface tests for corrosion.

Prestress force evaluation

One of the main uncertainties in evaluating tendon forces is the location of possible wire failure points. These can be partially detected, using radiography, when the tendons are in relatively small bundles placed in thin-walled concrete sections that are reasonably accessible.

In grouted tendons, even if the failure point is located, the question remains as to how well and how far from the breaking point the two new wire tips would redevelop their anchorage, hence prestressing capacities.

Thus far, beyond the failure zone, there are no reliable perfectly non-destructive methods to measure the prestress force accurately through embedded tendons on-site, without advances in instrumentation. Intrusive techniques, presented earlier, can be used instead: the crossbow method and incremental drilling.

If doubts are raised about the sound continuity or friction coefficients of tendons throughout the span, the recalculation process may resort to force reduction factors already established, for each prestressing system, through parametric experimental laboratory simulations and site tests, subsequently readjusted by statistical analysis, as already mentioned for calibration in the crossbow method.

Hence, for recalculation, the residual average prestress force is estimated on the basis of the verified tendon profiles, assuming realistic friction coefficients usually associated with the post-tensioning system adopted.

In a more pragmatic approach, bypassing tendon-associated data requirements, the stress release method can more directly lead to the prestress force actually transmitted to concrete key sections. Other valuable information may be obtained in the process, as already explained.

Structural recalculation procedure

Only road bridges will be discussed on this subject. Railway bridges are usually recalculated by a specialised authority, given their specific problems of structural and operational safety.

Context and concepts

Beyond a certain limit of load intensity, the structure's response becomes irreversible and the structure gradually loses its designed performance. The consequences may be visible cracks, plasticised areas or initially masked fatigue defects.

Degradation has two consecutive stages, respectively corresponding to two relatively conventional concepts, the *serviceability limit state* (SLS) and *ultimate limit state* (ULS), represented in Figure 24.68.

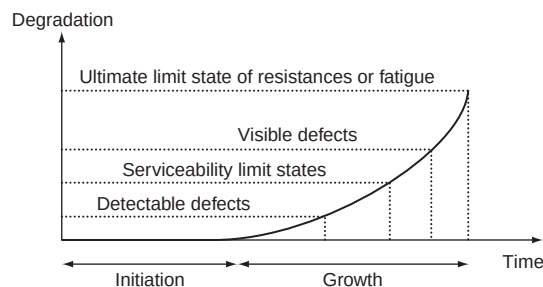


Figure 24.68 Degradation stages of a bridge with time

Visible defects usually indicate that certain serviceability limit states have crossed the reversibility threshold. Beyond the SLS, the bridge increasingly modifies its structural system, hence mechanical behaviour.

In a steel bridge, local *fatigue* defects reflect a steady development towards an ultimate limit state, putting the structure out of service. Repeated loads may then cause failure without even attaining the maximum allowable static load limit.

At this stage, the recalculation objectives are

- to establish the aforementioned *reference state*, characterising the present probable behaviour of the bridge
- to provide a framework defining repair or strengthening principles.

This reference state also facilitates eventual further investigation.

One of the basic assessment criteria is *structural reliability*, covering both serviceability and structural safety, respectively related to the various SLS verifications and the ULS imperatives. This also extends robustness and durability parameters.

Calculation codes and standards

Recalculation is generally based on codified or standardised texts related to test or traffic loads and to construction verification rules. These documents offer a logical framework to select the safety and reliability levels for a bridge awaiting repair or strengthening. Existing codes should be applied after cautious interpretation and judgement, given the particular features of most cases encountered.

On the whole, consecutive code amendments are constantly moving towards greater safety. Thus, maintaining a structure within the rules in force at the strengthening date proves more expensive than a mere defect correction in accordance with the codes that existed during its construction.

Current codes reflect reality more directly, contain greater safety margins and provide a more reliable analysis than the old ones.

Checking a new construction project, for the ULS of resistance, does not really situate it with respect to a physical collapse process, but only secures a safety margin beyond the SLSs.

Concrete bridges

Computations should refer to SLSs, whether for recalculation or for checking a repair or strengthening project.

In prestressed concrete bridges, for example, unacceptable cracks occur long before general failure. The SLS computation bases essentially correspond to the decompression limit states. At these discontinuities, the ultimate limit state does not govern, except for fatigue in well-bonded tendons crossing active cracks.

Verifications are made for longitudinal and transverse bending, shear and torsion, prestress diffusion and eventual tendon outward thrusts. When these all act simultaneously, only a study of the physical equilibrium of the different section parts can give a satisfactory result (Figure 24.69).

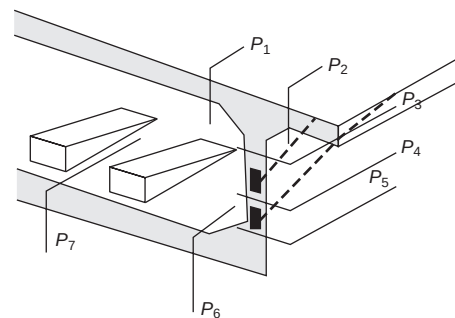


Figure 24.69 Sections P_1 to P_7 , for stress calculation, owing to different concomitant forces, in a concrete bridge

However, the risk of sudden failure exists in regions of prevailing shear and corrosion-stricken tendons. In this respect, special attention should be given to tendons at post-tensioned beam extremities when there are doubts about the protecting grout quality.

In reinforced concrete bridges, the current codes place a considerable importance on ULSs, hence the tangible effect after any reduction in the safety partial coefficients. In fact, the usually higher values of these coefficients are for crack limitation; if they are reduced, specific precautions should be taken in this respect, independently from the failure risk. Thus, here again, it is mainly necessary to refer to SLSs.

Steel and composite bridges

Many steel bridges now take loads of a much higher intensity than those they were designed for. This previously mentioned paradox stems from the cautious old codes assuming a wider variety of load models and integrating the random quality of metallurgic products, hence imposing very low allowable stresses. However, while cables, girders or other main structural elements cope reasonably well with the current traffic, secondary members, designed for much lighter local loads, should theoretically prove less effective. The observed resistance of many old stringers and floor beams to new axle loads is probably due to conservative old codes neglecting the favourable indeterminacy effects and, occasionally, to secondary states of equilibrium achieved through friction with the slab.

Hence, many observations show the difficulty in evaluating the load-bearing capacity of an old bridge from the original calculations.

The probable structural state is thus determined as in the case of concrete bridges, but the repair or strengthening project refers to the *ultimate limit state of resistance*. Under service loads, while the elastic normal stress distribution in beam cross-sections is complicated, owing to local and tangential components, evaluation of the ultimate bending moment is relatively reliable, since most behaviour particularities disappear with a totally plasticised steel section. This is why the ULSs are less conventional for steel and composite bridge sections than for their concrete counterparts and any reduction in the partial safety factors marks its full effect.

Concerning fatigue, the calculation of an existing structure's safety margin depends on assumptions strongly influencing the

accuracy of Wöhler curves, the choice of the detail category and other data. Several studies attempting to determine the reliability index with regard to fatigue have not yet obtained conclusive results.

Particular effects for long thin-walled structural members; the law of sectorial surfaces

Basic assumptions

Consider a bridge of length l , a characteristic width d and a section wall thickness t . If these dimensions are such that $t/d \leq 0.1$ and $d/l \leq 0.1$, the bridge is conventionally classed as a long, thin-walled structure.

Under pure axial compression or flexure, such bridges still behave according to the conventional rules of linear statics and elastic stability. However, under a transverse eccentric load or action, inducing a *restraint flexural torsion*, the simplifying basic assumption of *plane section conservation* no longer applies on its own. In addition to the conventional effects of pure flexure, the bridge cross-section does *warp*, while staying rigid in its own plane.

Torsion sets off two simultaneous mechanisms:

- the usual rotating *tangential* (or shearing) stresses owing to pure torsion
- an additional *particular* system of normal stresses, *self-balanced* over the section.

Normal stresses thus develop but their integral over the section is zero. The generalised system of forces, generating such effects, is known as longitudinal *bimoment*.

Physically, these normal stresses act locally but heavily at times, requiring, where need be, adequately resisting material. Conversely, in the general structural system, they play a minor role.

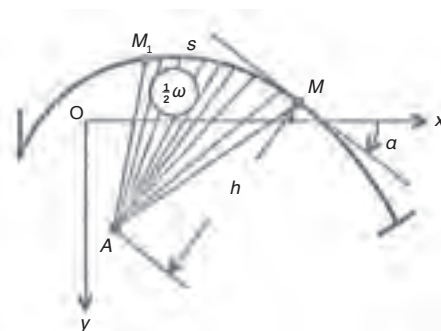
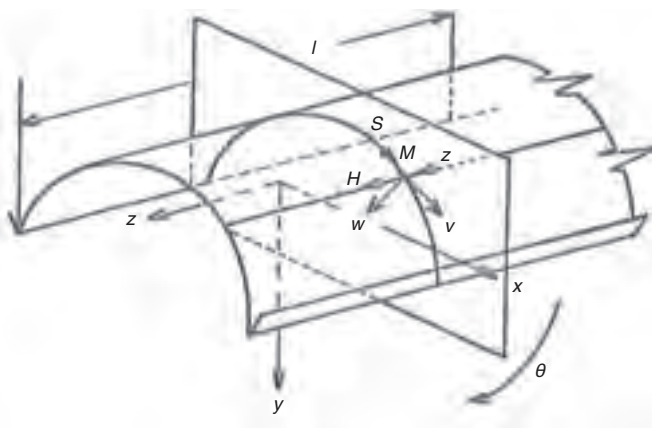


Figure 24.70 (a) Coordinates and sense of displacements of a thin-walled structure; (b) sector surface, ω , swept between the reference axis AM , and a data vector radius AM .

Analogy and conversion between two systems

Thus we face a cautious but necessary correlation between *pure flexure* and *flexural torsion*. The two systems are complementary, but they respectively impose their own assumptions, basic equations and fields of application.

- *Pure flexure* is a *linear* approach in statics, enforcing the *plane section law* with its simplifying assumptions, which have proved fairly realistic in this given context.
- *Flexural torsion* is a *radial* approach, significantly comprehensive, admitting the fact of section warping.

The *migration* from pure flexure to flexural torsion (Figure 24.70) takes place as follows.

- The conventional neutral axis becomes the *torsion centre*, emanating a *reference axis* and *data radii*.
- The simple linear displacement across the section fibres becomes the *swept sector surface*, ω , between the reference axis and a data vector radius.

This method is successfully applied to long *thin-walled* bridges, such as the two-beam bridge shown in Figures 24.71 and 24.72.

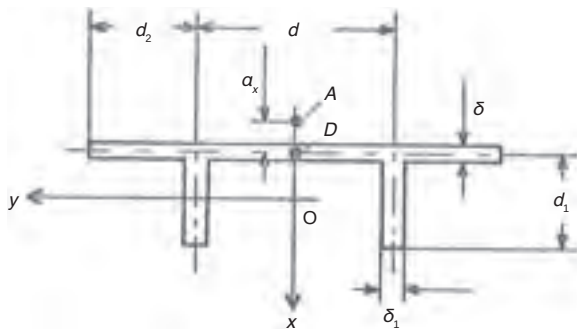


Figure 24.71 Thin-walled, twin-beam bridge, subject to detailed scrutiny while under transverse eccentric loading

Further analysis has shown that both the flexure and the pure torsion theories are simply particular cases of a greater general theory, governed by the *law of sectorial surfaces*, assuming infinite stiffness for the transverse sections in their own plane. This law has proved valid for reduced models and on-site.

Semi-probabilistic verification of structures

In the light of extensive scientific and technical studies in many countries, two concepts have emerged to characterise and estimate the structural aptitude of a bridge: the *safety factor* and the *failure probability*.

The former, a multicomponent factor γ , allows for uncertainties, through a limit states approach, assuming deterministic values for the actions' effects E and the resistance R , and checking for the condition $E < R/\gamma$.

The latter, called the failure probability, P_f , starts where, in a reliability-based calculation process, the effects E attain and cross the resistance level R , each represented by its respective independent probability distribution curve. The failure probability, P_f , varies *inversely* with another basic criterion defining structural safety: the *reliability index* β .

At present, this semi-probabilistic verification forms the basis of most structural calculation rules, establishing safety imperatives through different complementary channels:

- partial safety factors
- a set of margins introduced into the different calculation models and their corresponding equations
- representative values of random quantities, such as loading and resistance, corrected for their statistically admitted dispersal, or based on acceptance and control rules for the employed materials.

Road load models

For road traffic on lanes, cycle tracks and sidewalks, the Euro-code proposes various vertical and horizontal load models,

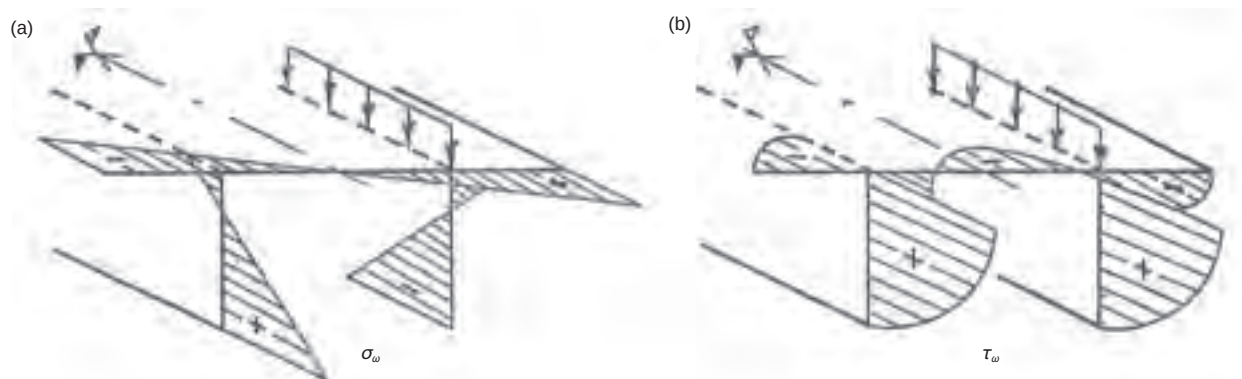


Figure 24.72 Redistribution of (a) normal stresses σ_w and (b) shearing stresses τ_w owing to restraint flexural torsion induced by eccentric loading on a long, thin-walled structure

covering all verification types associated with resistance and durability criteria. It also offers five checking models for the fatigue ultimate limit states.

Vertical load models for structural resistance evaluation

The main model, number 1 of 4, includes concentrated and distributed loads, on the whole road surface, according to rules based on the conventional lane concept. It is intended to cover the real traffic effects, including impact, with their intensity modulated by adjustment coefficients, with 1 for the heaviest traffic.

The other three models complement the main model, dealing, respectively, with

- local verifications for one axle load, including impact
- a series of special vehicles, 600–3600 kN, combined with the main model to simulate exceptional convoys, moving alongside the normal traffic
- dense pedestrian movement, associated exclusively with transient situations.

Several load intensity levels are defined, according either to their accepted overload probability p during a reference period r or to their equivalent return period t , both related as

$$t = -r / \ln(1-p) \approx r/p \quad (24.12)$$

The reference period r , used as a basis for assessing statistically variable actions, is taken here as the conventional service life of the bridge, generally 100 years.

Variable actions, Q , are thus defined as follows:

- *characteristic value*, Q_k , the main representative value, corresponding to a 10% overload probability in 100 years, or to a return period of 1000 years $\approx r/p$
- *frequent value*, $\psi_1 Q_k$, corresponding to a return period of 1 week, expressed as a determined part of the characteristic value, using a factor $\psi_1 \leq 1$
- *quasi-permanent value*, $\psi_2 Q_k$, expressed as a determined part of the characteristic value using a factor $\psi_2 \leq 1$, generally equated to zero for road loads ($\psi_2 = 0$).

From these models, the load groups are formed and assumed as a global action, to facilitate the engineer's task. The main load group, classed no 1 and denoted *gr 1*, includes the main vertical load model and, when applicable, a uniformly distributed load of 2.5 kN/m² on the sidewalks.

Fatigue models

Of the five proposed models, the first two refer to Wöhler curves on fatigue limits at constant amplitudes, hence exclusively concerning steel bridges. The very first is derived from the main characteristic model, through reduction factors applicable

to concentrated and uniformly distributed loads. The second involves a series of five lorries, considered separately.

The third model offers a standard design for fatigue in new structures. It consists of four 120 kN conventional axles. Using specific adjustment factors, this model is supposed to simulate fatigue damage under real traffic, for a conventional number of 2×10^6 applications.

The last two models tackle the fatigue problem by calculating, in different ways, the structure's expected lifetime. The former obtains the necessary traffic data by probabilistic simulation, the latter through direct recording.

Load combinations with respect to ultimate limit states of resistance

After checking for the static equilibrium of a section, member or connection, the limit state of failure or excessive deformation must be verified by the condition

$$E_d \leq R_d \quad (24.13)$$

where E_d is the design value of the actions' effects, such as internal forces and moments, and R_d is the design value of the corresponding resistance.

Generally, and in accordance with the given situation, verifications regarding the *ULS of resistance* apply the following load combinations of actions.

Fundamental

$$\sum_{j \geq 1} \gamma_{Gj} G_{kj} + \gamma_P P_k + \gamma_{Q1} Q_{1k} + \sum_{i \geq 1} \gamma_{Qi} \psi_{0i} Q_{ki} \quad (24.14)$$

Accidental

$$\sum_{j \geq 1} \gamma_{GAj} G_{kj} + \gamma_{PA} P_k + A_d + \psi_{11} Q_{k1} + \sum_{i \geq 1} \psi_{2i} Q_{ki} \quad (24.15)$$

Seismic

$$\sum_{j \geq 1} G_{kj} + P_k + \gamma_1 A_{Ed} + \sum_{i \geq 1} \psi_{2i} Q_{ki} \quad (24.16)$$

where

- G are permanent actions, for example, fixed dead loads and subsequently redistributed forces owing to shrinkage and differential settlement
- P is the relevant representative value of a prestressing action, for example, by tendons or imposed deformations
- Q are variable actions, for example, live traffic or environmental loads, with Q_1 the leading variable action 1 of the combination
- A_d is the design value of an accidental action, for example, an explosion or vehicle impact
- A_{Ed} is the design value of a seismic action, bearing an importance coefficient, γ_1 , which adjusts the characteristic value, A_{Ek} , hence the intensity, to the considered structure: $A_{Ed} = \gamma_1 A_{Ek}$
- γ are partial factors of safety ($\gamma_G, \gamma_P, \gamma_{PA}, \gamma_{Qi}$) for the actions involved, respectively: permanent, prestressing, accidental prestress variation and variable actions i

- ψ are reduction factors, ψ_1, ψ_2, ψ_0 , applied to the *characteristic* values of *variable actions* Q , defining, respectively, their frequent, quasi-permanent and combination values, provided, for ψ_0 , that Q is not the leading action of the combination.

In the case of a linear elastic behaviour, the partial safety factor γ_F , for any action F , may be interpreted as the product of two other factors

$$\gamma_F = \gamma_i \times \gamma_{sd} \quad (24.17)$$

covering the following uncertainties:

- γ_i in the adopted representative value of the considered action
- γ_{sd} in both the structural and action models.

In a parallel approach, the representative resistance value of a material is its characteristic value divided by a partial safety factor γ_m , which, in turn, is resolved into two components:

$$\gamma_m = \gamma_m \times \gamma_{rd} \quad (24.18)$$

where

- γ_m covers a possible deterioration of material properties and eventual local defects
- γ_{rd} accounts for uncertainties in the resistance model plus geometric deviations.

Load combinations with respect to serviceability limit states

Verifications regarding SLS mainly use the following load combinations of actions:

$$\text{Characteristic} \quad \sum_{j \geq 1} G_{kj} + P_k + Q_{k1} + \sum_{i \geq 1} \psi_{0i} Q_{ki} \quad (24.19)$$

$$\text{Frequent} \quad \sum_{j \geq 1} G_{kj} + P_k + \psi_{11} Q_{k1} + \sum_{i \geq 1} \psi_{2i} Q_{ki} \quad (24.20)$$

$$\text{Quasi-permanent} \quad \sum_{j \geq 1} G_{kj} + P_k + \sum_{i \geq 1} \psi_{2i} Q_{ki} \quad (24.21)$$

Numerical interpretation of the partial safety factors

There are various ways to approach partial factors numerically. The European Concrete Committee, for example, proposes the following interpretation.

Permanent actions

Owing to its usually adequate consistency and clear dominance over permanent actions, dead weight is introduced, in a first calculation phase, with its nominal value taken directly from the project drawings and conventionally accepted densities.

In a second phase, support reaction measurements may be needed in the case of inconsistent results.

In the fundamental combination, the partial safety factor γ_G , related to permanent actions, is equal to 1.00 or 1.35, respectively, according to its favourable or unfavourable nature with respect to the basic variable action Q_{1k} . Following this interpretation, the factor 1.35 can be resolved into 1.20×1.125 , where 1.20 is the partial security factor γ_i for the action itself and 1.125 is the partial factor for the model. The same values of γ_G , 1.00 or 1.35, are adopted for other permanent actions. They are generally introduced with upper and lower characteristic values, for example, the prestress force, at a given time t , with extreme limits $P_{k,sup}(t)$ and $P_{k,inf}(t)$. For these actions, the same dead load factor resolution is accepted.

Variable actions

As for permanent actions, the partial safety factor for variable actions is taken as equal to 1.35, given the slight dispersal of their extreme values and owing to the choice of the characteristic value for road loads in the main model, based on the probability of a 10% excess in 100 years, as defined previously.

Safety factors related to the constituent materials

Detailed studies have produced three main safety factors, γ_m , regarding construction materials and products:

- $\gamma_m = 1.10$ for structural steel
- $\gamma_m = 1.15$ for prestressing or reinforcing steel in concrete elements
- $\gamma_m = 1.50$ for concrete.

These consecutive values clearly confirm the impact of homogeneity and manufacturing quality control, which are much superior in steel than in concrete.

For a more explicit interpretation, the following definition may be worth remembering: if X is a continuous random variable, and p a real number between 0 and 1, then the *fractile* of order p will be the value x_p of the variable X such that

$$\text{Prob}(X \leq x_p) = p \quad (24.22)$$

The adopted acceptance limit for resistance was set at an occurrence probability of 5/1000, that is, a 0.005 fractile.

For a reliable purely experimental verification, the required number of tests then increases beyond reason. However, a parallel probabilistic analysis may allow a considerably reduced number of tests, against a moderately higher safety factor.

Thus, for structural steel, the factor $\gamma_m = 1.10$ corresponds practically to the shift from a 0.02 fractile, on which the characteristic resistance definition is based, to the required 0.005 fractile.

For reinforced or prestressed concrete, with passive or active steel, the characteristic resistance is based on the 0.05 fractile of a supposedly very long series of tests. The factor 1.15 can be

interpreted as the product $1.05 \times 1.05 \times 1.05$:

- the first 1.05 is the model factor, γ_{Rd} , mainly accounting for deviations of steel positions with respect to the design, hence influencing the lever arm of the section elastic couple
- the second 1.05 considers steel damage on-site and covers minor errors
- the last 1.05 represents the fractile order transformation from 0.05 to 0.005.

As for concrete itself, the intrinsic factor γ_M is 1.5 to bring the occurrence probability into line with that of steel, at the 0.005 fractile. The adopted factor can be resolved to $1.10 \times 1.10 \times 1.24$. The first 1.10 is the model factor, γ_{Rd} , described already. The second 1.10 accounts for differences, taken as an average, between conventional and effective resistance in the structure. The factor 1.24 results from the 0.05 to 0.005 fractile order transformation of the effective resistance, depending on the testing quality.

Recalculation of a distressed bridge

State evaluation under dead load

The calculations for the structure under permanent loading enable the establishment and validation of the model to be used for both interpreting the observed defects and designing a repair or strengthening project.

Evaluating the structure under dead load involves appraising the state of stress owing to a quasi-permanent combination of

- average or probable dead weight, preferably checked by measurement in certain cases
- eventual average prestressing force, also experimentally checked, as described in the section on evaluation and testing methods
- a representative value of thermal gradients, depending on the return period, chosen in accordance with certain objectives.

Concerning thermal action, it may be remembered that linear temperature gradients are only simplifying assumptions, used in some evaluation methods for new structures, but cannot describe the actual physical mechanism. Established codes (e.g. BS EN 1991-1-5 (BSI, 2003b)) supply non-linear temperature gradients between the extreme fibres of the main types of bridge deck, for a more accurate calculation of the structure's response.

Calculations for a bridge under a quasi-permanent load combination should reflect the actual behaviour; experimental results should be checked.

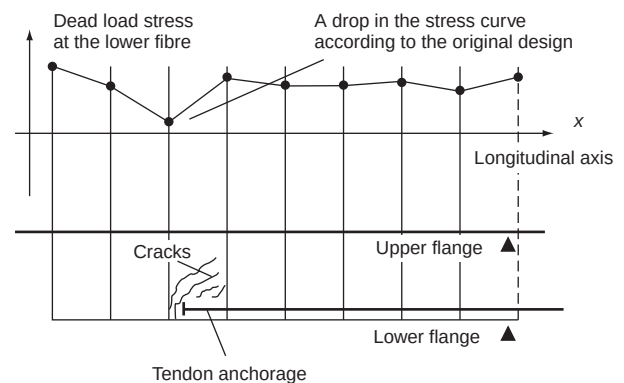


Figure 24.73 Profile of a post-tensioned bridge section: concrete cracks appear where calculations show a sudden drop in compressive stress

- Calculated and measured support reactions must reasonably agree.
- Concrete regions where calculation shows a high tension or a drop in compressive stress should correspond to observed cracks (Figure 24.73).

If this is not the case, the investigation must start all over again. Any repair or strengthening carried out in a doubtful context might fail and cause additional defects.

Calculations should also take into consideration

- the consecutive construction phases, the exact schedule and any identified parasitic forces
- investigation results, for example, geometric survey and strength tests
- the most representative constitutive laws of the involved materials, especially concerning steel relaxation, concrete creep and shrinkage, which are of great importance in statically indeterminate structures.

Stress determination under a quasi-permanent load combination should enable the explanation or interpretation of eventual defects, especially those already visible under dead load.

In post-tensioned bridges, the introduction of a prestress margin may be justified if there are serious doubts about the integrity of tendons or if a recalculation based on a probable prestress value cannot adequately explain the observed structural behaviour. Conversely, experience has shown that, even when an old bridge satisfies the latest codified recommendations, the often-slight section reinforcement generally fails to take up the developing tensile forces and to maintain the structure's safety with regard to brittle failure.

Response to service loads

After calculations have been made for the structure under quasi-permanent combination, the analysis of its behaviour under live loads has two objectives.

- Improve diagnosis and achieve a positive and quantitative interpretation of the observed defects.
- Estimate the load-bearing capacity before designing a repair project.

It is to be remembered that codes defining service loads do not exactly represent actual loads. They are models, accompanied with numerical values, intended to envelop the real load effects, in certain conditions.

For a realistic appraisal of the actual behaviour, both *frequent* and *characteristic* load combinations should be successively considered. These correspond to two distinct loading levels. The former is statistically taken for the traffic flow over a week or so; the latter over a much longer period. The structure's degree of damage is then estimated according to whether the observed defects are better interpreted on one level or the other.

The interpretation of calculation results should be thoroughly analysed. There are two possible cases.

- The calculation *enables* interpretation of the observed defects. The elaboration of the repair project can then start, either to bring the bridge back to its initial service aptitude or, if the operation is too costly, to perform at best selective repair and limit the authorised live loads.
- The calculation model *does not enable* the explanation of the observed defects. Resorting to a more advanced model then becomes imperative, for example, finite-element analysis of the damaged parts. In many cases, these computations should be completed by the field investigation tests described in this chapter. As certain defects may be due to construction or utilisation errors, all possible information, including testimonies, should be collected on these two items, including testimonies.

Conclusion

The elements and guidelines briefly presented in this chapter are necessary but not sufficient to analyse the degradation, target the investigation and accomplish the assessment of existing bridges. These pieces of information supply technical knowledge but cannot directly ensure judgement, only acquired through experience, given the frequently subtle interaction of parameters. For the safety of the public and conservation of the cultural heritage and invested capital, it is essential to attain and maintain such a level of competence. This uneasy yet fruitful task can ensure better assessment and, possibly, optimum strengthening or updating of bridges that internal and external factors might have rendered structurally deficient or functionally obsolete.

While current inspection procedures enable the assessment of bridges at a given time or period of their service life, further possibilities are now available for a thorough monitoring and, possibly, active control of structural parameters, such as stress and curvature redistribution. Incorporated instruments are being developed to measure and transmit the actual evolving parameters, analyse their time-dependent variation

and disregard minor fluctuations but respond to irreversible deviations. The response may be a regulating action on applied forces, such as a carefully controlled readjustment of external tendons or relative support levels. This approach towards integrated, centralised, actively reactive systems, operating in real time, continues to pave the way to generations of smart and sounder structures.

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Chapter 25

Inspection and assessment

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The different types of inspection are described for the maintenance or acceptance of bridges, the former category comprising safety, general, principal and special inspections and inspections for assessment. With reference to rail infrastructure, visual and detailed examinations are equivalent to general and principal inspections. Preparation, records, health and safety, access and good practice are touched on, and the requirements for each type of structure are summarised. Good practice for inspection is discussed under the headings of structural appreciation, structural identity, structural disguise, accuracy, weather and photography. Inspection of different types of structure is described for signs of distress in reinforced concrete structures, prestressed bridges, drainage systems, hollow members, culverts, steel structures, corrugated steel buried structures, masonry structures and retaining walls. Assessment is introduced, and the formulae for identifying whether a structure passes its assessment are provided. The levels of assessment are described, followed by descriptions of preferred methods of analysis for reinforced concrete slabs, reinforced concrete beams and slabs, steel beams and reinforced concrete slabs, inverted T-beam decks, shear key and filler beam decks, single-span brick arches, continuous brick arches, concrete post-tensioned beams or slabs, concrete post-tensioned box girders and steel box girders. Common structural problems are listed for reinforced concrete slabs, reinforced concrete beams and slab, beams, reinforced concrete beams, box culverts, prestressed concrete, halving joints, composite beams, filler beams, riveted beams, cast iron beams, steel beams, steel box girders, jack arches, brick arches and piers. Strength considerations of deteriorated reinforced concrete structures are summarised. A dozen techniques for obtaining additional strength from assessments are then each described in detail. These are realistic models, comprising global models and mathematical models. Global models are described under soil–structure interaction, surfacing, inclined neutral axis, bearing clamping, redundancy, composite action and bridge modelling following pier impact. Mathematical models are discussed under bridge decks and non-linear finite-element models.

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Inspector competency

Before discussing the various types of inspection, it is worth providing an overview of the requirements of the personnel required to undertake these activities, and who then report on associated findings. Inspector competency is not a new concept, but evidence suggests that it is a feature that has until now been addressed in an ad hoc way and, arguably, inconsistently.

What is an inspector in the context of bridges or other structures? An individual with the responsibility for determining whether a structure is safe for use and fit for purpose. This

places a significant amount of responsibility on inspectors and the role that they undertake. As such, the concept of inspector competency is an important one.

Network Rail addressed inspector (or examiner) competency in a more prescriptive manner following the Stewarton Enquiry (RAIB, 2010). More recently, the Department for Transport (DfT) has provided appropriate industry guidance and an associated framework through the Bridge Inspector Certification Scheme (BICS). Assessing the experience and knowledge of inspectors within the industry, a tiered system of certification is now available.

Maintenance inspection

Introduction

Inspections can be carried out for the maintenance or for the acceptance of bridges. The object of maintenance inspections is to detect in good time any defect that might cause an unacceptable safety or serviceability risk or a serious maintenance requirement, in order to safeguard the public, the structure and the environment and to enable appropriate remedial action to be taken. Inspection also provides information that enables the management and maintenance of a stock of structures to be planned on a rational basis in a systematic manner and must be undertaken by suitably experienced and competent personnel under a quality assurance system in accordance with general statutory and other relevant health and safety requirements similar to that referenced in the previous section. The lead inspector or examiner should be an experienced engineer or technician with a background in design, construction or maintenance of highway, railway and other bridges as appropriate. Five types of maintenance inspection are used for highway structures: safety, general, principal, special and inspection for assessment. For rail structures, NR/L3/CIV/006 Structures, tunnels and operational property examinations (Network Rail, 2019) defines seven types of maintenance inspection (or examination): visual examination, detailed examination, underwater examination, enhanced examination, additional examination, visual examination by line of route and inspection for assessment.

Safety inspection

The purpose of a safety inspection is to identify obvious deficiencies that represent, or might lead to, a danger to the public and therefore require immediate or urgent attention, for example, spalling concrete with a risk of injury to the public (Figure 25.1). Safety inspections are carried out at frequencies that reflect the importance of the structure as well as any incidents that might have led to safety concerns being raised, such as bridge strikes or vandalism. Safety inspections, when needed, typically require frequent scheduling, such as monthly, weekly or even daily walkovers. These identify any obvious deficiencies or signs of damage and deterioration that might require urgent attention or might lead to accidents or high maintenance costs, for example, collision damage to the superstructure or bridge supports, damage to parapets, spalling concrete and insecure expansion joint plates. Staff moving around the network should be encouraged to be vigilant at all times and to report anything needing urgent attention.

General inspection or visual examination

The purpose of a general inspection or visual examination is to provide information on the physical condition of all visible



Figure 25.1 Concrete spalling (courtesy of Yellowstone National Park)

elements; the inspection comprises a visual assessment of all parts of the structure that can be inspected without the need for special access equipment or onerous traffic management arrangements. This should include adjacent earthworks and waterways and river banks; the latter for scour or flooding or for conditions such as debris collection or blockages, which could lead to these.

Principal inspection or detailed examination

The purpose of a principal inspection or detailed examination is to provide comprehensive and detailed information on the physical condition of all inspectable parts of a transportation structure. It comprises a close examination, within touching distance of the inspectable parts of a structure, utilising, as necessary, suitable inspection techniques, access and traffic management, and should include adjacent earthworks and waterways. Suitable inspection techniques should include hammer tapping to detect delamination of concrete cover and paint thickness measurement. Alternatives to close examination, such as closed-circuit television, may, in appropriate circumstances, be used for difficult or dangerous locations, for example, obscured areas or confined spaces. For example, unmanned aerial vehicles (UAVs; drones) are being used by some bridge owners to provide visual detail of potential defects that might otherwise be difficult to access.

Testing is not normally a requirement, but for concrete structures limited testing may be used such as half-cell potential, chloride level, cover meter and depth of carbonation, and for steel structures MPI, ultrasonics and radiography to check for weld or fatigue cracks. Principal inspections or detailed examinations are normally carried out at a defined

interval relevant to the type of structure being inspected. Whilst many highway bridge owners still adopt the typical 6 yearly inspection interval, for example, the concept of risk-based inspection provides bridge owners with the option to vary this frequency up or down, depending on the importance of the structure to the network, structural durability and redundancy in elements, structural type and material composition and associated failure mechanisms and accessibility.

Enhanced examination

Enhanced examinations are similar to safety inspections with respect to the rail environment. These are targeted examinations, used where a typical visual or detailed examination regime is insufficient to provide data required to adequately manage the structure safely. An enhanced examination can determine the state or condition of specific structural elements or features between detailed examinations.

Special inspection

Special inspections should be considered for cast iron structures, structures strengthened with bonded plates and structures with weight restrictions, as well as for the following structures: structures having to carry an abnormal heavy load, involving inspection before, during and after the passage, if an assessment has indicated a reduced margin of safety or if similar loads are not known to have been carried; structures in areas of mineral extraction, when subsidence occurs; structures if settlement is greater than allowed for; structures subjected to accidental damage; river bridge foundations after flooding (see the section on common problems in carrying out assessments); permanent access gantries; hoists, winches and associated cables and post-tensioned bridges. Special inspections are also required for more in-depth studies of structural problems.

Additional examination

Additional examinations on rail infrastructure are similar to special inspections. Typically, they are seen as a temporary measure of scheduled inspections. They will generally have a defined scope, such as an intervention in response to a defect or structural element in significantly poor condition. The additional examination is typically used to collect more information on changes in the condition or structural behaviour of elements.

Underwater inspection or examination

An underwater inspection is a specific type of special inspection concerned with parts of bridges below water level, for which a programme of such inspections must be followed. The inspection must record the condition below water level, the existing streambed profile and any evidence of scour. Inspections are usually limited to tactile undertakings, with reduced visibility usually affecting collected data.

Inspection for assessment

The purpose of an inspection for assessment is to provide the information required to undertake a structural assessment. This is necessary to verify the form of construction, the dimensions of the structure and the nature and condition of the structural components, including all the parameters needed to determine the strength of members and elements, including possible deficiencies, for example, cracks, corrosion, settlement, defective materials and drainage. The inspection should cover not only the condition of the individual components but also the condition of the structure as an entity, especially noting any signs of distress and its cause. Reference to available historical and as-built records in advance of an inspection for assessment can support dimensional verification, missing information, existing problems, structural arrangement, and so on. It is recommended that information gaps are identified in advance to make full use of time spent on-site and ensure that enough information is collected to facilitate the assessment stage.

Scheduling of inspections

Inspections should be scheduled to make the most efficient use of resources and to minimise disturbance to the public, for example by taking advantage of traffic management measures planned for other purposes. *Well Managed Highway Infrastructure: A Code of Practice* (UK Roads Liaison Group, 2016) highlights risk-based inspection strategies for highway assets. Various bridge owners have applied the guidance using a methodical approach; for example, CS 450 (HE, 2021a) allows each structure to be assessed with the aim of increasing the principal inspection interval from 6 years to 12 years in 2 year increments. Assessments consider various factors, including structural type and durability, use, consequences of failure, structural condition and available records.

Preparation for inspections

Preparations are vital before inspections (Figure 25.2). Prior to a general, principal or special inspection, the inspector should review the structure records and become familiar with the characteristics of the structure, the condition at the time of the



Figure 25.2 Inspection of bridge soffit over river

previous inspection and any significant maintenance or modifications since. A method statement should be prepared and agreed with the supervising engineer, including what aspects of the structure are to be assessed.

Records

Records are not normally produced for safety inspections, except when urgent action is required, in which case, there will be laid-down procedures, whereby inspection staff have a clearly defined duty to inform the supervising engineer or designated staff at the earliest possible opportunity of any defect that might present an immediate risk to public safety or structural stability.

The records created by a general inspection must include, as a minimum, an indication of the location, severity, extent and type of any defect. Common use of a proforma to record bridge condition indicators (BCIs) provides a recognised system of capturing this information quickly and easily. The intention of any reporting mechanism used to provide a comparative assessment of structures should be to reduce subjectivity; BCI reporting goes some way towards achieving this. The use of a system to provide comparative indexing of stock allows asset owners to benchmark structure conditions and establish a method of determining appropriate intervention and a programme of maintenance and repair.

A principal inspection should include a review of the completeness and accuracy of the inventory records and any deficiencies rectified. The records created should include, as a minimum: the location, severity, extent and type of all defects, including, where appropriate, detailed descriptions, photographs and sketches; headroom measurement; any significant changes, for example, works or deterioration since the last principal inspection; any information relevant to the integrity or stability of the structure; the scope and timing of any remedial or other actions required before the next inspection; the need for a special inspection, additional investigations or monitoring; and a description of any testing, details of information gathered and an interpretation of the findings.

The records created for a special inspection should include the following as a minimum: the background and reasons for the special inspection; a detailed description of the condition of those parts of the structure that have been inspected, including, where appropriate, photographs and sketches; any significant changes, for example, works or deterioration, since the last principal inspection; a description of any testing undertaken, details of the information collected and an interpretation of the findings; any information relevant to the integrity and stability of the structure; the scope and timing of any remedial or other actions required before the next inspection; and a specification of the need for any additional investigations or monitoring.

The records created by an inspection for assessment must provide the information required to carry out a structural assessment.

Acceptance inspection

The purpose of an acceptance inspection is to provide a formal mechanism for exchanging information and documenting and agreeing the current status of, and outstanding work on, a structure prior to changeover of responsibility for operation, maintenance and safety from one party to another. Three types of acceptance inspection are generally used: pre-opening principal inspection, defects liability inspection and transfer inspection. The format, content and timing of an acceptance inspection depend on its specific purpose.

Health and safety for inspection

Health and safety requirements that should be given particular consideration in relation to the inspection of bridges include, but are not restricted to, the following.

- *Working on highways.* Risk assessments must be carried out. Particular care must be taken to ensure the safety of both the inspectors and the public. Following the hierarchy of risk control, such that removal or isolation of a hazard is preferable to control, traffic should be effectively prevented from entering the works area using appropriate barriers and signage. Additionally, all personnel working within the highway boundary should normally wear high-visibility outer clothing except when in a vehicle. Vehicles stopped on the hard shoulder are particularly at risk and should not remain occupied.
- *Working at height.* It should be noted that nearly all bridge inspection activities will require some level of working at height (Figure 25.3). Again, risk assessments must be made. Particular care must be taken to ensure the safety of both the inspectors and the public. The use of appropriate access must be planned prior to carrying out the work and the use of appropriate safety equipment, such as harnesses, must be considered where appropriate.



Figure 25.3 Access using mobile elevated work platform during bridge inspection

- **Confined space inspection.** Risk assessment should be used to identify those structures that constitute confined space hazards. Staff undertaking confined space inspection should be competent to do so and have the training, experience and knowledge of the specific hazards associated with such environments and appropriate controls to manage them. It is important to note that activities undertaken as part of the inspection process might also influence the classification of confined space working. For example, small span bridges or culverts of significant length and carrying water-courses can attract accumulations of silt and organic material. When these accumulations are disturbed, gases such as methane or hydrogen sulphide can be released, thereby increasing the risk to inspecting personnel. Inspectors entering confined spaces must be provided with and use appropriate personal protective and safety monitoring equipment. Entry permits should also be considered, supplementing existing safe systems of work. Health screening and training in diseases such as leptospirosis and other waterborne infections is recommended. Staff should familiarise themselves with equipment prior to carrying out an inspection and undertake periodic checks to ensure that it is functioning correctly.
- **Encountering toxic mould.** Wherever mould growth is encountered in a box girder, it should be treated as toxic, and all inspection work ceased until the level of toxicity has been established as being within safe limits. In the UK, advice on safety should be sought from the appropriate office of the Health and Safety Executive.
- **Working near or on railways.** Inspections near or on railways should be arranged with, and carried out in full accordance with the requirements of, the relevant railway body.
- **Diving operations for underwater inspections.** All diving operations in the UK are covered by health and safety requirements and associated regulations and approved codes of practice. All divers involved in commercial operations are required to hold valid medical certificates, a completed log book and a health and safety approved diving qualification.

For all inspections, an assessment of the risks must be undertaken before work starts. In the UK, the CDM (Construction Design and Management) Regulations 2015 require that a health and safety file is kept of each structure. The purpose of this file is to alert those responsible for operating and maintaining a structure to the key health and safety risks that must be addressed during the operation, maintenance, repair and construction work. There will be considerable overlap between the records required for the health and safety file and the records required by the overseeing organisation for the effective engineering maintenance of the structure.

Environmental aspects for inspection

The environmental impact of inspection work must be considered, such as the effect of the work on nearby people and on sensitive wildlife or habitats. Bats and their roosts, and birds, are legally protected.

Access for inspection

Access for inspection may be by ladder, scaffolds, mobile elevating work platforms (MEWPs), including underbridge units, and abseiling (Figure 25.4). Deciding who needs to have access may influence the method chosen. For example, if a senior engineer or specialist needs to see a particular defect with his or her own eyes, scaffolding or a MEWP may be required, whereas if a photograph will suffice, it may be better to use abseiling or UAV imagery. Specific safety requirements apply in detail to each of the forms of access, and must be followed.



Figure 25.4 Abseiling access

Good practice for inspection

Structural appreciation

Virtually all inspections require the inspector to observe and record the defects present in a structure. However, while diagnosis of the causes of the defects is not necessarily a requirement of the inspection, it is of great value for the inspector to have an appreciation of structural behaviour and of the defects that might occur. Such an appreciation will guide inspectors, alerting them to particular signs to look out for and enabling them to concentrate attention where it is most needed. This will ensure that, when a defect is observed, the necessary data will be collected on-site so that a correct diagnosis can be made. However, it is not unusual for defects to be due to a combination of causes. This can make diagnosis difficult. It is, therefore, important for inspectors not to jump to conclusions: they should be open-minded and always be on the lookout for the unexpected.

Structural identity

A careful check should be made of the identity of the structure to ensure that it is the correct one. A quick look round

should be made to ensure that movement joints and bearings are correctly identified, that the orientation is understood and that any drawings agree with the observed layout.

Structural disguise

Inspectors should be aware that things may not always be as they seem. For example, a retaining wall with a masonry face may be solid masonry or may be masonry cladding to mass or reinforced concrete, steel sheet piling or reinforced earth. The bridge records should make this clear but do not always do so. Some bridges may have had repairs or alterations made that are superficially similar to the remainder of the structure, but conceal a different form of construction. These may not always be recorded. Where identified during the inspection or initial desk study, these should be noted within the inspection record.

Hidden critical elements

Inspections typically focus on visible elements of structures and associated defects without recourse to intrusive investigation. Some defects can be explained through contributions from defective hidden elements, such as waterproofing or foundation movement. However, this is not always obvious. The inspector should always consider those elements that may contribute significantly to structural integrity but are not always visible. For example, bridge deck half joints, elastomeric bearings with reduced clearances between soffit and bearing shelf, buried surfaces of primary deck members, etc. Recommendations for intrusive investigation should be made where uncertainty exists.

Accuracy

The effectiveness of an inspection depends entirely on conveying clear and accurate information to the bridge engineer, who is responsible for deciding on any action needed. Points should be noted as soon as the observation is made, never trusting to memory. After noting details of the defect, the notes should be checked to ensure that the bridge engineer will gain the

correct impression from them. An example of various concrete defects recorded, together with explanatory notes as to what they represented following diagnosis, is given in Figure 25.5.

Weather

The weather at the time of the inspection and the previous few days should always be noted. Rain, wet surfaces or poor light can affect observations and the width of a crack or joint may depend on the temperature. Apparent changes in condition between one inspection and the next can sometimes be due to different weather conditions when the inspections were made. Some defects can also be more apparent after inclement weather; for example, leakage through inadequately waterproofed decks following a period of rainfall or ponding on poorly formed carriageway surfacing that may increase the risk of skidding during freezing conditions.

Photography

Photographs are extremely useful, both of a significant part of the structure, and close-ups of details or defects. For the latter, a familiar object should be included to demonstrate the scale. It is better to take more rather than fewer photographs than may be required. Additionally, photographs showing general views of the structure help in future reviews of inspection records to understand the structural arrangement more holistically. Photographs with references of some type to key elements shown in the drawings are also important, particularly where there are a number of repetitive elements on a structure, such as cross-girders or diaphragms on a multispan structure. Here, it is particularly important to ensure that it is clear which element is being photographed.

Where access is difficult, it can be useful for future reference to record, by photographs or video, the access or traffic management adopted.

The use of digital photography enables the inspector to review photographs taken; it is recommended that this is done to ensure that sufficient detail is captured prior to leaving the site. Digital photography also makes in situ referencing

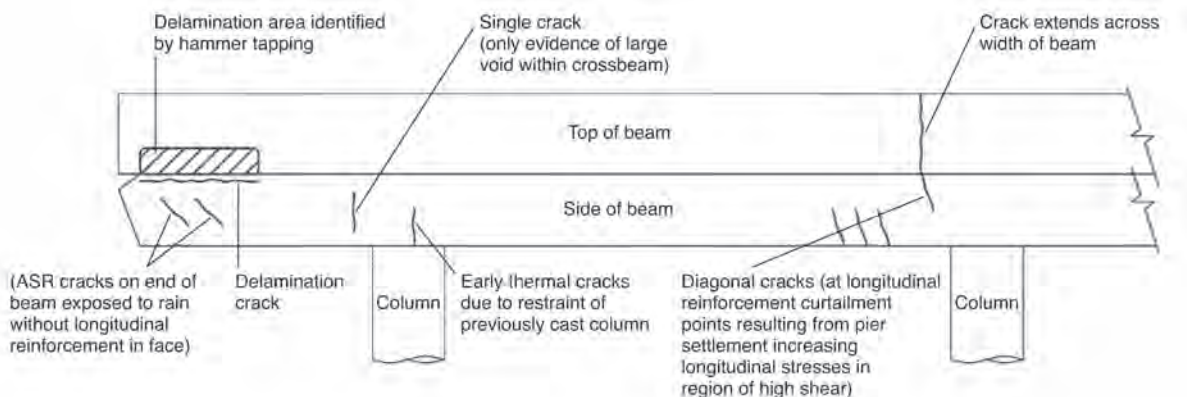


Figure 25.5 Typical defects in inspection report (explanations have been added for clarity)

straightforward; it is recommended that an accurate description of each photograph is taken on-site to support efficient reporting back at the office.

Inspection of different types of structure

CIRIA guide C764, *Hidden Defects in Bridges: Guidance for Detection and Management* (Collins *et al.*, 2017), gives details of common defects in bridge structures and culverts, including defects that might not be visible during an initial inspection. Some common defects and issues are given in the following sections.

Signs of distress in RC structures

Typical defects in concrete bridges include cracking and delamination of concrete, corrosion of reinforcement, alkali–silica reaction, sulphate attack, chlorination and carbonation and inadequate cover. Other defects include missing or incorrectly placed reinforcement. Figure 25.6 below shows some typical concrete deck defects.



Figure 25.6 Concrete deck defects

Signs of distress, as a result of overloading, construction defects or inadequate strength, are more likely to be noticeable on the sides or soffits of reinforced concrete beams. It can be difficult to detect structural distress in slabs, except that caused by excessive sagging moments, when transverse cracking may be evident at midspan.

The profile of a bridge deck may need to be checked for sagging or unusual deflection; this can be achieved by sighting along the parapet, taking sag measurements as required. Excessive vibration during the passage of a heavy load may also indicate the bridge to be defective. Structural health monitoring techniques, such as digital image correlation, can be used to monitor the behaviour of a structure under load by recording the deflections.

Where a superstructure incorporates horizontal transverse hinge joints, deterioration at the joint may be indicated from below by signs of either cracking parallel to the joint or leaking or staining below or adjacent to the joint. Where deterioration is

suspected, it may be necessary to instigate a special inspection.

Similarly, half-joints supporting suspended or ‘drop-in’ spans experience issues with cracking, staining and exposure to de-icing salts, leading to excessive corrosion of reinforcement and cracking or delamination of concrete. Cracks at the re-entrant corners need particular attention and may be an indication of inadequate, poorly placed or damaged reinforcement. Where deterioration is suspected, it may be necessary to instigate a special inspection; see also CS 466 (HE, 2020a).

Prestressed and post-tensioned bridges

Additional problems with prestressed bridges include the implications of cracking. Pre-tensioned and post-tensioned structures are normally designed to avoid cracks in the concrete. Thus, the development of cracks can be indicative of problems, as prestressed bridges can suffer serious loss of strength through hidden deterioration – and are less likely to show warning signs than reinforced concrete (Figure 25.7).



Figure 25.7 Post tensioned tendon investigations

Pre-tensioned beams are often laid side by side with small gaps between, and these are not always concrete-filled. The voids may be inspected using an endoscope.

Where the tendons in post-tensioned bridges are in grouted ducts, there may be voids in the grout, permitting access of moist air, water or de-icing salts, resulting in corrosion of the wires and perhaps severe deterioration of the tendons, with potentially serious consequences. Ingress of water and salts is most likely to occur at joints in segmental construction, and other construction joints and at anchorages, but may migrate along the tendon to voids elsewhere. Post-tensioned bridges constructed in the 1960s, in particular, may have issues with voids in the grout in the ducts. Detection of voids can be difficult (HE, 2020b). However, non-destructive tests (NDTs) could be used to identify and quantify voids. A high-resolution ground-penetrating radar (GPR) survey can be used to map the

post-tensioned ducts in the external faces of the post-tensioned deck. Subsequent impact echo testing can be employed to identify voids throughout the extent of the located ducts. If voids are found by NDTs, destructive testing may be necessary, following a risk assessment (HE, 2020c), including concrete breakouts with exposure of the ducts and tendon to establish the condition of the tendons. Destructive testing should only be carried out in accordance with CS 463 (HE, 2019); the person responsible for overseeing risk review, risk assessment and post-tensioned concrete bridge special inspections must have specialist experience of post-tensioned bridge design and construction methods and have proven and demonstrable wide experience of post-tensioned bridge management, inspection, testing and monitoring procedures.

Post-tensioned ducts may also become displaced during concreting; if this is suspected, it should be investigated.

Leakage

The underside of a bridge should be inspected for signs of leakage or staining, especially at the ends of deck slabs, at expansion joints, around drain outlets and at service troughs, as this might indicate damage to, or inadequate sealing of, the waterproof membrane. It should be noted that water can travel a considerable distance from its source before becoming evident, and that water contaminated with de-icing salts is a common cause of defects in concrete structures. In dry weather, it can be difficult to distinguish between fresh stains caused by current leakage and old ones caused by former, repaired leaks, so a return visit after wet weather is recommended (Figure 25.8).



Figure 25.8 Water leakage through deck soffit cracks

Hollow members

The drain holes at the low ends of hollow members should be checked to ensure that they are not blocked. Small members can be inspected inside using an endoscope, while large members should be accessed and treated as confined spaces. It is important to check that water has not collected inside as the increased weight can have serious consequences. Void formers suspected of having moved on concreting should also be checked.

Culverts

Culverts should be inspected at the end of a long dry spell, when the water flow is at a minimum. Precautions for working in or over water will have to be followed. With significant flow, the inspection may have to be treated as an underwater inspection. Care should be taken over the possibility of flash flooding. Many culverts constitute a confined space. Industrial waste and debris can collect in culverts.

Steel structures

Typical defects in steel bridges include cracking, section loss from corrosion (Figure 25.9), fatigue cracking, brittle failure, failure of connections (bolts and rivets), latent weld defects and metallurgical defects. In steel structures, structural movement or failure will probably first become evident as movement at connections, where there may be material or loading defects. Cracks are potential causes of complete fracture and usually occur at connections, changes in section and other fatigue-susceptible details, particularly when changes are abrupt. When a crack is identified, all other similar details should be examined.



Figure 25.9 Corrosion of a steel truss and plates on the Oakland Bridge (courtesy of Oregon Department of Transportation)

Failure of shear connectors may be indicated by separation between the top flange and the concrete slab, so that interface should be closely examined. Deformation or distortion of members may reduce the load capacity, and sighting along flanges should be carried out, taking maximum deformation measurements where required.

Internal surfaces of hollow sections may need to be examined. Otherwise the comments for hollow concrete sections apply.

Breakdown of protective systems is probably the most common defect; it is beneficial to detect early stages to reduce the maintenance required. Leakage of water onto horizontal surfaces or connections, where it may pond, is a common cause of defects in protective systems. Connections may move or slip, causing the protective system to fail by cracking. Where defects in the protection system cannot be identified, specialist paint inspectors may be required to provide further advice.

Corrosion is usually associated with the breakdown of protective systems. It is important to assess the magnitude, location and form of corrosion and, if possible, identify its cause. Inspectors should assess and record any loss of structural section.

Buried corrugated steel structures

The comments on culverts apply to buried corrugated steel structures, inspection of which is generally limited to exposed surfaces. Defects most likely to be observed in the steel components are seepage at joints or seams, corrosion through breakdown of the protective system or deformation of the arch shape through excessive or altered loading. The junction of the steel with a concrete invert or other surfacing is an area prone to corrosion, since chlorides or other contaminants are likely to be concentrated there.

The inspector should look for signs of bulging and deformation in the shape and line of the steel arch or ring and for signs of the settlement of fill in areas above or adjacent to the arch. An overview of the immediate surroundings should be undertaken to identify changes, such as erection of structures subsequent to the construction of the corrugated structure that may, for example, alter the loading or the level of the water table in the vicinity. Excessive corrosion and subsequent section loss in culvert inverts may lead to failure of the culvert and deformation, which can sometimes be observed at road level by settlement of the road pavement.

Masonry structures

Although strong in compression, masonry is very weak in tension and defects in masonry arch bridges manifest themselves as cracking or deformation (Figure 25.10). Inspection of masonry and bridges consists largely of visual inspection and measurement, supplemented by trial pits or coring to confirm details of hidden features.



Figure 25.10 Deformed masonry arch

By virtue of their age, most masonry structures will have been subject to prolonged exposure to large variations in temperature and humidity, freeze–thaw and atmospheric pollutants. Such damage to the surface layers of the structure should be relatively easy to identify and record.

Leakage or seepage of water is a common cause of defects in masonry bridges since waterproofing relies on an impervious surfacing or fill, unless a membrane has been added during refurbishment, is sealed at the spandrel walls and drains and has not been subsequently damaged. The arch barrel should be inspected for signs of leakage and account taken of water possibly travelling some distance to the leakage point.

Deformation or distortion of the arch ring might reduce the load-carrying capacity of the structure. Sighting from a distance may assist in checking the arch profile for deformation; measurements should be taken where appropriate.

Any defects reducing the thickness of the arch ring could reduce the load capacity of the structure, so the loss or delamination of masonry units or loss or deterioration of mortar should be noted.

Cracks in masonry can arise from a variety of causes. They might affect the appearance only or be indicative of a more serious defect. Recent or progressive cracks are more serious than those that might have occurred soon after the bridge was constructed. Evidence that cracks are recent might include clean faces to the crack and loose fragments of masonry or mortar. Cracks formed only in the mortar might be indicative of joint deficiencies. Inspectors will often need to map the extent of cracking so that comparisons can be made with previous inspections.

Evidence of movement may be obtained by sighting along lines of mortar joints or features built into the structure, as well as along lines of parapets.

Concrete saddles, tie-bars and relieving arches are some of the wide number of techniques used for strengthening masonry arches; these must be inspected for evidence of failure, particularly at the interface with the masonry. Gunite or shotcreting of the intrados of masonry arches is generally only carried out as a last resort, as this prevents future visual inspection of the arch barrel.

Earth-retaining masonry walls – spandrel walls, abutments, wing walls and retaining walls – commonly fail because of lateral forces from the retained fill. All such walls should therefore be inspected for signs of movement, which may manifest as cracking, bulging, tilting or lateral displacement. Spandrel walls can be affected by the lateral forces from the fill and from traffic loading.

Retaining walls

Retaining walls should be inspected as closely as any other structure. The principal defects that might occur are excessive movement of the whole wall, such as tilting or sliding, or of part of it, such as bulging or differential settlement, and problems arising from water seepage.

Structural defects leading to excessive movement may be observed during a close inspection, but they can also sometimes be detected by standing back and observing the wall as a whole, since signs of distress, movement or misalignment that might be overlooked during close inspection may be apparent from a distance. Sighting along parapets, string courses or other features is also a good method of detecting misalignments. Note, however, that walls are often constructed with a batter of around 1 in 50. The form of construction might influence the location and type of defect; for example, the face might bulge between counterforts. Defects in flexible masonry retaining structures might stem from changed conditions in the vicinity of the structure, for example, adjacent construction works or clogged drainage paths.

Reinforced retaining walls can suffer from chloride attack, either through salt spray from passing traffic or from run-off from a carriageway supported by the wall running over and down the face of the wall or seeping down the back of the wall.

Inspectors should be particularly alert to changes in the loads imposed on the walls. These can often be caused by raising the ground level or storing materials behind the wall. Where there is vehicular access along the top of the wall, any changes in use should be noted. Realignment of a carriageway or temporary diversion of traffic might impose heavy live loads close to the wall, causing overstressing.

Most walls rely partly on passive pressure from the ground in front of the wall. Thus, any excavation or lowering of the ground in front of the wall can have a serious effect on the stability of the wall; for example, excavation of a services trench could bring down the whole wall.

Fences and guard rails along the top of the wall should be included in the inspection.

Introduction to assessment

The purpose of assessment of structures is to check their adequacy for specific loading levels and to identify those bridges that have an unacceptable risk of failure, either in part or complete collapse, under extreme circumstances of loading and material condition.

Many of the factors that bring about structural collapse can-

not be considered in calculations, for example, undiscoverable conditions or freak events. However, calculation-based assessment is the only practical means available at present for gaining assurance about the adequacy of the whole stock of highway structures.

The absence of any apparent signs of distress in a substandard structure does not mean that it is structurally adequate. When the failure mode is likely to be brittle, there may be no early warning signs. Furthermore, end restraint or composite action, which cannot always be relied on in certain older structures, may temporarily prevent such a structure from showing distress.

In general, structures are assessed by the application of limit state principles, usually at the ultimate limit state, although more modern structures are often also assessed at the serviceability limit state. The modified Military Engineering Experimental Establishment (MEXE) method for masonry arch bridges determines allowable loads directly, rather than in terms of the limit state. Cast iron bridges are assessed on a permissible stress basis, which would exclude the risk of fatigue failure.

The assessment action effects S^*_a are given by

$$S^*_a = \gamma_{f3} (\text{effects of } (Q^*_a)) \quad (25.1)$$

$$Q^*_a = \gamma_{fL} Q_k \quad (25.2)$$

where γ_{f3} is a factor that takes account of inaccurate assessment of the effects of actions, γ_{fL} is a partial factor given in the standards for each action, Q_k represents the characteristic actions and Q^*_a is the assessment load.

The assessment resistance R^*_a is given by

$$R^*_a = F_c (\text{which is a function of } (f_k, \gamma_m)) \quad (25.3)$$

where F_c is a condition factor, less than or equal to 1.0, accounting for potential reductions in resistance associated with conditions that are not already accounted for in the values of material strengths, the partial factors for material strengths or the resistance model; f_k is the characteristic, nominal or worst credible strength of the material; and γ_m is the partial factor for material strength given in the standards. For steel and wrought iron, $R = (1/\gamma_m) \times \text{function } (f_k)$. For cast iron $R^* = \text{function } (f_p)$, where f_p is the permissible stress.

Structures shall be deemed capable of carrying the assessment load when $R^*_a \geq S^*_a$, the asterisk referring to the factored value.

The ratio of R^*_a and S^*_a is normally tabulated in assessment reports for each force action in each element and for each principal loading: R^*_a/S^*_a is known as the capacity ratio (CR) and S^*_a/R^*_a as the usage factor (UF). Thus, CRs less than unity or UFs greater than unity are unsatisfactory.

Assessment of an existing structure may be conducted, if the structure fails an initial assessment, in stages of increasing complexity, with the object of determining its adequacy with minimum effort. Early stages may utilise conservative means of determining load effects. Provided that a structure is shown

to be adequate at these stages, no further analysis would be required. However, if a structure is found to be inadequate at an early stage, then assessment work should continue, and later stages should seek to remove any conservatism in the assessment calculations. In some cases, results will quickly become self-evident and the process can be terminated at an early stage.

Assessments may be made at distinct levels, increasing in sophistication from levels 1 to 3; these are discussed in the following section. Beyond level 3, reliability-based methods of assessment can be used. Means for carrying out assessments at levels 1, 2 and 3 are contained within existing assessment standards and advice notes.

Levels of assessment

Level 1 assessment

This is the simplest level of assessment, giving a conservative estimate of load capacity. At this stage, only simple analysis methods are necessary, and full partial safety factors from the assessment standards are used.

To facilitate such an assessment, national highways assessment standard CS 454 has been developed (HE, 2020d). This document represents a considerable relaxation compared with design standards, which would be the only alternative in their absence. It should be recognised that the use of design standards for assessment would result in structures failing assessment unnecessarily.

In addition to CS 454, assessment versions of the design standards for concrete, steel and composite bridges, that is, CS 455 (HE, 2021b), CS 456 (HE, 2020e) and CS 457 (HE, 2020f), respectively, have also been developed by national highways for the purpose of providing more realistic assessment for these bridge types.

Level 2 assessment

This next level of assessment involves the use of more refined analysis and better structural idealisation, as indicated by the existing standards and advice notes.

More refined analysis may include more detailed analysis, such as shell finite-element models and non-linear and plastic methods of analysis (e.g. yield-line analysis or orthotropic grillages).

This level also includes the determination of characteristic strengths for materials based on existing available data. This may be in the form of existing mill test certificates or recent tests on another similar structure.

Level 3 assessment

Level 3 assessment includes the option to use bridge-specific assessment live loading (BSALL). For long-span bridges, where the 40t assessment fails by a small margin and the bridge is located on a lightly trafficked road, use of BSALL may be beneficial.

Level 3 assessment may make use of material testing to determine characteristic strength or yield stress and worst credible strength (HE, 2021b) or worst credible yield stress (HE, 2020e).

Reliability-based methods of assessment

Level 1 to 3 assessments are based on implicit levels of structural safety, incorporated in the characteristic values of loads and resistance parameters and the corresponding partial safety factors. The values of the partial factors used for the assessment of existing structures are generally intended to provide the same level of reliability as for the design of new structures.

Reliability-based methods can allow a direct assessment of whether the probability of failure of a structure is acceptably low. Reliability-based methods can be of benefit in cases where, for a specific structure or element of a structure, use of the standard partial factors leads to a particularly conservative probability of failure, compared with that required of similar structures or elements.

In a reliability analysis, the input parameters can be described using probability density functions (PDFs) and the output can provide a probabilistic assessment of the likelihood that the structure will satisfy a certain limit state.

The models used for analysing the effects of loads and assessing resistances, to establish whether a limit state is reached, are generally the same for both deterministic and reliability-based methods.

For example, in one method for undertaking a reliability analysis, called the Monte Carlo method, many separate analyses are made, sampling input parameters from each input distribution in proportion to their likelihood. For each sampled set of inputs, an analysis is undertaken in much the same manner as for deterministic analysis, with the output probability distribution constructed from the results of these many separate analyses. Because of the added numerical complexity of reliability-based methods, in some cases it can be impractical in a reliability analysis to use some of the more sophisticated analysis methods suitable for deterministic assessments.

Further information on reliability-based methods is provided in ISO 2394 (ISO, 2015) and BS EN 1990 (BSI, 2002).

Alternative approaches to a reliability-based assessment based on the use of reduced partial factors for assessment that are consistent with a reduced target reliability index are described in Hendy *et al.* (2018) and fib (2016). These approaches consider that the target reliability levels for the design of new structures are unlikely to be appropriate for the assessment of existing structures. For existing structures, the cost of increasing the reliability is significantly greater than that during design, as increasing reliability usually involves strengthening of a structure in service. Hendy *et al.* (2018) and fib (2016) propose smaller partial factors to be used to achieve lower target reliabilities, which still provide adequate human safety.

Preferred methods of analysis for assessment

Introduction

The choice of analysis approach is a key part of the assessment process. With increasingly powerful computers and analysis software, it is easy to assume that a more complex analysis will provide more accurate results. While this may be true in many cases, the questions that need to be asked are what level of accuracy is required for the analysis and how this can be achieved most effectively.

Consideration should be given to how the results of an analysis will be used. Increasingly powerful packages can make it tempting to try and recreate a structure as accurately as possible; this can lead to considerable wasted effort. Codified checks of sections are based on consideration of moments and shears in beams; these are often much more rapidly extracted from simplified grillage models than more complex shell or solid models.

Detailed finite-element models of structures can give the impression that every aspect of the structure is accurately represented but, if the model is linear elastic, stress concentrations may indicate failure that, in reality, local yielding or cracking will resolve. Codified ultimate limit state (ULS) equations account for these effects but to accurately include this in an analysis model would require the introduction of non-linear effects.

In the following discussion, it is stated that grillage analysis is generally the default initial assessment method. This has increased in use, compared with the strip method, owing to the increased performance of analysis packages and the ease of use of autoloader tools, which tend to be set up for grillage or shell models rather than line beam models.

Reinforced concrete slabs

Reinforced concrete slabs are usually analysed with grillages for level 1 assessment. The strip method may be used if the assessment is considered likely to pass; for example, for the addition of a concrete barrier or a minor change in carriage-way layout. For a level 2 assessment, where the transverse reinforcement is light, cracked section properties are used; if this still fails, yield-line analysis may be used. A shell model may be used for highly skewed or curved slabs, although the improvement over a grillage model may be limited. Anisotropic properties for the shells would be required to include the effects of cracking. Anisotropic elements are essential when assessing slabs, particularly for older structures, where transverse reinforcement was limited.

Yield lines can be used, sometimes with a computer program but more often by hand. This is encouraged by publications, but there is no standard in the Design Manual for Roads and Bridges (DMRB) concerning the procedure. Because such methods give upper bound solutions, there is some concern

about non-conservative answers being obtained from hand calculation methods.

For reinforced concrete slabs with very light transverse reinforcement, 'failures' are often identified when using linear elastic grillage and finite-element programs. (See also comments on the Wood–Armer calculations, next.) Far greater strengths can be obtained by modifying the transverse properties using cracked sections, torsionless grillages, arbitrarily reduced stiffnesses or (rarely) non-linear properties. There is a view that any stiffness is acceptable for a modified grillage if a larger load-carrying capacity results, as this is then approaching a non-linear analysis. However, ductility, serviceability and shear capacity should always be checked using a linear elastic analysis, particularly where the assessment is concerned with increasing the load rating of a structure. Also, reinspection of the structure for cracking in the areas initially identified as overstressed should be conducted, particularly where serviceability overstresses are identified, as these issues are likely to be visible.

Wood–Armer calculations are intended to provide the optimum balance between transverse and longitudinal reinforcement in design and so are not appropriate for assessment. A moment field approach is more suitable, but the previous considerations tend to obviate the need to use this approach.

Non-linear finite elements are sometimes used if yielding and redistribution of stresses can be accepted.

Standard CD 360 (HE, 2020g) covers compressive membrane action or arching but its use is not as widespread as perhaps it should be, partly owing to yield lines being applicable to the same types of structure; this often gives sufficiently increased capacity.

Reinforced concrete beams and slabs

Reinforced concrete beams and slabs with low skew are usually analysed by grillage, which is necessary for skew slabs. A strip method may be used where the assessment is considered likely to pass or where only one beam is of interest; for example, for the addition of a concrete barrier or a minor change in carriage-way layout. Grillage is the preferred technique; a yield line is used where the slab fails in bending. Shear is checked from the grillage analysis. Punching shear checks are carried out but are rarely critical. Membrane action has not been used. Pucher charts are sometimes used for local wheel loads, although these need to be combined with the global effects of the structure. A simple shell finite-element model of a single bay with fixed or pinned edge restraints may be used as an alternative to Pucher charts. Westergaard's method is another alternative, similar to the Pucher methodology, which can be used if the transverse moments cannot be determined from the grillages.

Steel beams and RC slabs

Steel composite beam-and-slab decks are analysed in a similar manner to RC beam-and-slab decks. Yield lines are generally restricted to slab failures.

Inverted T-beam decks

Inverted pre-tensioned T-beam decks are generally analysed on a strip basis or, for larger or higher skew structures, by using grillage. Transverse moments are rarely correctly determined using grillage and reduced transverse properties are introduced. The yield-line method is rarely used.

Shear key and filler beam decks

Grillage analysis is generally used, or the strip method if appropriate. Transverse effects are covered by limiting tensile stresses. Use is made of national highways CS 457 (HE, 2020f).

Single-span and multispan brick and masonry arches

Standard CS 454 (HE, 2020d) supersedes DMRB documents BD 21 (Highways Agency, 2001) and BA 16 (Highways Agency, 1997). It consolidates the material relating to masonry arch assessment in a single chapter with clearer requirements that are not as focused on the use of the modified MEXE method and gives additional restrictions on the use of the modified MEXE method.

The assessment of arch bridges is only as good as the accuracy of the dimensional survey and the condition survey carried out on-site to inform the assessment. The rise at the crown, the rise at the quarter points, the arch span (skew and square) and the springing height should be recorded, as well as the depth of the arch barrel and the depth of fill over the arch barrel. For arches with a non-uniform profile, additional measurements may need to be taken along the profile of the arch barrel. If intrusive investigations are carried out, the depth of the arch barrel can be established, either by excavating a trial pit from above the crown of the arch and taking a level survey to the extrados or by drilling pilot holes from the underside of the arch (the intrados). If no intrusive investigations are carried out, it is common practice to use a lower bound and upper bound approach where the lower bound assessment assumes that the arch barrel is 60% of the depth of the facing stones and the upper bound assessment assumes that the arch barrel is 100% of the depth of the facing stones. The condition survey of the bridge helps the inspector to assign condition factors to the bridge and considers mortar loss, friability of the mortar joints, materials, thickness of the mortar joints and direction and spacing of any cracks that may be present in the arch barrel.

The initial live load assessment is normally carried out using the modified MEXE method (for spans ranging between 5 and 18 m). However, where the arch is flat (the span/rise ratio exceeds 8), appreciably deformed or with skew greater than 35°, or where the depth of fill over the arch barrel is greater than the thickness of the arch barrel, the MEXE method is not deemed suitable. In addition, for brick arches where ring separation has been identified as likely, the MEXE method may not be used. If the results of the MEXE method need to be verified, if the

MEXE method gives unsatisfactory results or if the MEXE method is not deemed appropriate, alternative methods of assessment are carried out using computer software packages. The most common analysis packages use either the three-hinge limit analysis method or the rigid block method. Alternatively, finite-element packages have been used successfully for the live load assessment of arches using non-linear analyses. For multispan arches, MEXE has been used by assuming fixity at springing, particularly where piers are stocky (height/width ratio of 2) and where arches and piers are restrained by cross-walls at each end of the arch barrel. The software packages used for single-span arches, that is, the three-hinge limit analysis method and the rigid block method are also generally capable of modelling multispan arch structures.

The recommended phasing of the assessment is to first assess the arch structure considering the existing condition of the bridge with the assigned appropriate condition factors. If the results are not satisfactory, the arch structure is then assessed considering improved condition factors, assuming that repairs are carried out (such as repointing of the arch barrel). If the live load capacity of the structure is still below the required live load rating, backing can be considered in the analysis and confirmed on-site by visual inspection or by intrusive investigations (pilot holes through the wing walls or spandrel wall and or trial pits).

For arches, MEXE is sometimes used to give the loadings; these are then used in a space frame analysis program. Sometimes, if the arch is thin and flexible, soil-structure interaction can be modelled using a finite-element program. This gives much reduced bending effects and hence greater capacity.

In general, there is far less concern about arches, as they have strong inherent strength and distress can be observed. They tend to be condition-dominated and so poor-condition arches tend to be repaired or replaced.

Concrete post-tensioned beams or slabs

Grillage analysis is normally used, particularly if a strip analysis has been used and failed. For voided post-tensioned slabs, a shear flexible grillage has been used with a reduced shear area for transverse members. Finite-element shell analysis has also been used, but it may be more difficult to interpret the results.

Concrete post-tensioned box girders

A line beam with a shear flexible grillage and a transverse plane frame or a full grillage analysis model is generally used. An analysis package that covers post-tensioned construction can be used to determine stressing sequence and losses. The grillage or line beam approach will generally require a separate transverse model to quantify the transverse frame action occurring in the structure, potentially combined with Pucher charts or similar. Alternatively, a finite-element shell analysis may be used to quantify these transverse effects.

Steel box girders

Line beam or shear flexible grillage are used at level 1. Shell finite-element analysis is used at level 2. Non-linear finite-element models can effectively be used for higher level assessments.

Common problems in conducting assessments

The following text outlines assessment deficiencies commonly found within different structural forms. It also sets out common construction issues that can lead to assessment deficiencies.

Reinforced concrete slabs

- Early slabs with limited reinforcement and inadequate anchorage lengths.
- Transverse reinforcement in RC slab decks.
- Insufficient anchorage for shear enhancement at the end of decks and at the sides of pipe bay members.
- Low concrete strengths, causing insufficient resistance.
- Local effects from accidental wheel loading on footways and verges.
- Cantilever slabs in flexure under accidental wheel loading on footway.
- Service trenches failing under accidental wheel load.

Reinforced concrete beams and slabs

- Low concrete strengths, causing insufficient resistance.
- Early RC beam-and-slab bridges in shear and moment.
- Reinforced concrete beams and slabs in bending or shear at curtailment points.

Beams

- Verge members are often more critical than deck members, particularly for skewed decks.

Reinforced concrete beams

- Lack of longitudinal anchorage in RC beams, such that shear links are ineffective.
- Corroded RC cross-beam cantilevers in bond.

Box culverts

- Reinforced concrete box culverts in shear and flexure.

Prestressed concrete

- Prestressed beams in shear and flexure.
- Links too far apart to be effective in inverted T-beam decks.
- Corroded transverse tendons in precast beam decks without in situ slab where the waterproofing fails.

Post-tensioned beams and decks

- Voids in grout in post-tensioned ducts and subsequent corrosion of strands or tendons.
- Voids in grout or lack of grouting causing tendons to be debonded over longer lengths, thereby causing concrete and tendons to be acting independently rather than integrally.

Half-joints

- Original detailing can be very poor, leading to inherent structural issues.
- Half-joints in shear.

Composite beams

- Lack of shear connectors in composite beams.
- Lateral torsional buckling of continuous steel beam–concrete composite decks.

Filler beams

- Filler beams in bending.
- Encased steel beams in longitudinal shear.

Riveted beams

- Rivets in longitudinal and vertical shear in plate girders.
- Bearing stiffeners in riveted bridges failing, owing to being unfitted and with insufficient rivets.

Cast iron beams

- Cast iron beams in flexure.

Steel beams

- U-frame action in steel bridges.
- Connections of transverse non-composite steel members.
- End restraint inadequate for plate girders and through-girders to limit compression flange or chord buckling.

Steel box girders

- Flanges of steel box girders in bending.
- Web shear in steel box girders.
- Manholes in diaphragms of steel box girders.
- Diaphragm welds in steel box girders.
- Deck beam supports in steel box girders.

Jack arches

- Jack arches, owing to lack of transverse restraint, such as tie-bars.

Brick arches

- Brickwork supports in bearing.
- Pier stability in multispan arches.

Piers

- Piers failing, owing to collision loading, in both bending and shear.

Deterioration of reinforced concrete structures

For RC structures, the most common defects caused by deterioration are those of delamination of concrete cover and corrosion and consequent loss of section of reinforcing bars owing to chloride attack. As delamination can be widespread, the loss of anchorage bond strength and consequent weakening of the structure can be highly significant, and estimates of condition factor will not provide reliable or necessarily safe values of capacity ratio. Extensive research (Kennedy Reid, 2000), commissioned by Chakrabarti on behalf of the Highways Agency, recommended the following.

- The strength of sections (and the slenderness of columns) should be based on ignoring the cover when delaminated.
- Reinforcement strength should be based on the corroded cross-section.

- Sections a limited distance along a main bar from corrosion can be based on the corroded bar cross-section plus the strength provided by the delaminated bond over the intervening length, the bond strength depending on whether or not the bond is delaminated.
- Compression bar effectiveness in beams should be reduced in proportion to corroded link section versus minimum link section. Outer compression bars in beams should be ignored if the cover is delaminated and there is significant link corrosion. Longitudinal reinforcement in columns should be discounted where corrosion and delamination reduce the effectiveness of links to below the minimum.
- Where the main bar capacity is less than the force developed through moment and shear, the effectiveness of the links should be reduced.
- Where the ends of the links are corroded, their loss of effectiveness in shear may not be as much as the proportion of cross-section lost (Regan and Kennedy Reid, 2004).
- Without shear links, the shear strength of a delaminated member should be reduced and is dependent on the extent of flexural cracking and whether the delaminated bond strength is adequate.
- Shear enhancement of delaminated members can be based on strut-and-tie analysis to determine the force in the main reinforcement over the support and thereby the adequacy of the anchorage using the delaminated bond strength.
- Full allowance may not need to be made for delamination for punching shear in the punching shear zone other than the loss of cover, provided the reinforcement is fully anchored beyond the area considered.
- To determine anchorage strength, the force in the main steel should be derived from moment and shear. The bond capacity is based on a formula depending on cover from mid-barrel or flush with the bars as appropriate, stirrup spacing, diameter of main bar, area of stirrup leg adjacent to main bar, square root of concrete strength and degree of clamping pressure. Delaminated bond strength is very low.
- Rules are given so that bond strength of bars in different situations, that is, in or not in stirrup corners, clamped or unclamped, are not fully additive.
- The bond strength of corner bars exposed to half barrel on both faces is ignored.
- Perpendicular cracks with aggregate interlock will reduce shear resistance.

Seeking additional strength from assessments

Many structures of different types fail their assessment, some by quite large margins, and yet very few show actual signs of distress. Clearly, many structures have strength that is not accounted for in the standard methods of assessment. The techniques described next provide some means by which typical sources of hidden strength can be quantified for assessment.

Additional site investigations

Additional testing of the materials used and their dimensions or quantities may also be carried out to improve assessment findings. Often, as-built records for older structures will not show the full details of the as-built information. Standard CS 464 (HE, 2020b) sets out guidance for non-destructive testing that can be carried out to confirm the condition of the structure. However, additional intrusive testing may also be required. This could include residual stress testing, steel strength testing (Vickers hardness), additional concrete cores for concrete strength, confirming rebar orientation, layout, spacing, cover, and so on.

Use of monitoring to verify model behaviour

Various forms of monitoring may also be used to improve structural rating, where deflections or strains measured on-site can be linked to the behaviour in the analytical model, and thereby may be used to justify alternative load paths to reflect the findings made through on-site monitoring.

Reviewing loading on the deck

A review of the live loading applied to the modelling for additional capacity, it can be justified to consider the actual traffic lanes using existing line markings rather than the notional lanes set out in CS 454 (2020d).

For abnormal loads, it is worth considering reducing the vehicle speed of the abnormal vehicle to 15 miles per hour, in order to reduce or eliminate dynamic amplification factors.

Realistic models

Introduction

Simplified models must be conservative to provide an adequate factor of safety on analysis. The more accurate the model, that is, the greater the degree of realism provided by the model, the less need there is for conservatism and the greater the amount of strength that can be determined for the structure. However, greater complexities in models can lead to a false sense of security regarding the accuracy of the results. Care therefore needs to be taken when developing more complex models to ensure that they are likely to provide an accurate set of results.

Broadly speaking, there are two types of model. First, the global model of the bridge, which determines which elements are included, such as whether soil supporting abutments is also modelled, or whether surfacing is included. Whether elements of the bridge interact is also a factor, such as composite action between the deck slab and supporting beams. Composite action may be difficult to quantify, particularly where it was not

originally considered as part of the design.

Second, there is the mathematical model of the elements of the bridge, such as whether the deck is analysed by grillage or by finite-element analysis.

Global models

Soil–structure interaction

Soil–structure interaction can be modelled by finite-element analysis using geotechnical analysis programs to develop the effect of soil restraint on the structure. Typical examples where this soil restraint has a beneficial effect include brick arches and reinforced concrete box culverts. Particular care is required, however, with such an approach, owing to the variable value of the properties of the soil, and the risk that the relieving effect of the soil on the structure might not be as large as calculated.

With box culverts, in situ stress measurements can be carried out to confirm the relief afforded by soil action, for example, if the soil is arching over the culvert roof and relieving the roof of some direct vertical loading (see Figure 25.11).

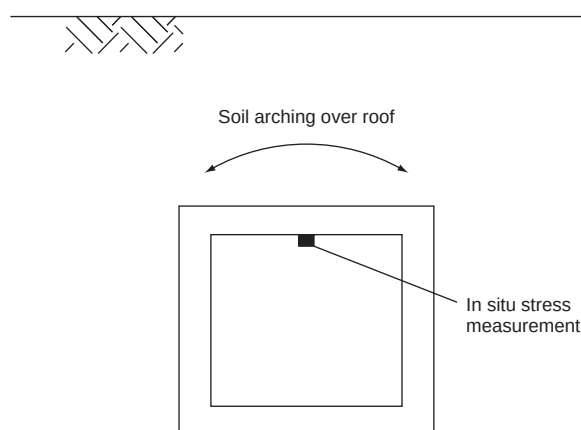


Figure 25.11 Box culvert

Currently, there are no assessment standards setting out how to consider soil–structure interaction in existing structures, and while design standards such as PD 6694-1 (BSI, 2004) (and its predecessor, BA 42/96 (DoT, 1996)) could be used, the estimated effects resulting from soil pressures are likely to lead to predicted theoretical overstresses in the structure, particularly for longer bridges. Inspections, therefore, should form a key aspect of an assessment of an integral bridge to identify whether the structure is displaying any signs of distress, owing to the continued expansion or contraction into or from the soil.

Surfacing

Incorporation of the bridge deck surfacing will stiffen decks and reduce stresses, as described in the section entitled ‘Surfacing’.

Inclined neutral axis

Where a bridge deck or deck beam soffit is arched, the neutral axis will be inclined towards the piers, providing a useful vertical component of concrete bending compression, which will reduce the applied shear stresses. This is covered in the section entitled 'Inclined neutral axis'.

Bearing clamping

Transverse pressure can increase bond strength; this can apply under deck beam bearings or over columns. The effect is particularly beneficial where cover has been lost and the bond strength significantly reduced. This is covered in the section entitled 'Bearing clamping'.

Redundancy

Structures occasionally have redundancy that is not allowed for in straightforward analysis. For example, if a bridge deck beam is damaged, the adjacent beams may be able to carry the load shed by the damaged beam and distributed by the surrounding transverse beams (Idriss and White, 1991).

Another example is where the deck is attached to the sub-structure through RC panel walls linking an RC cross-beam to the RC deck slab supported on steel deck beams. The frame action derived from the deck acting compositely with the cross-beam through the panel walls can provide some additional strength to the cross-beam. This can assist an interim situation where the cross-beam has deteriorated, prior to propping and repair.

Another area of potential redundancy is the tensile strength of the concrete. Although this is not usually an acceptable design or assessment parameter, it will provide additional strength to a concrete structure.

Composite action

Normally, concrete decks supported by steel beams develop composite action from suitably designed shear connectors, and decks supported by precast concrete beams develop composite action by the provision of sufficient shear links crossing the interface. Where these are not provided, or are provided but with deficiency, some composite action or additional composite action is provided by adhesion, friction or, in the case of a scabbled concrete interface, by a degree of aggregate interlock. Such effects are difficult to quantify, but they are known to be of value in enhancing the structural strength. Load testing of some bridges has confirmed a useful increase in capacity by demonstrating such effects.

Filler beam decks can provide composite action without provision of shear connectors. Rivet or bolt heads can also provide composite connection where cast into the concrete.

Bridge modelling following pier impact

Piers can be vulnerable to severe damage if subject to heavy vehicle impact. However, the removal of a single column or pier might not result in collapse of the deck. Reassurance may be obtained by modelling the deck without the support of the vulnerable column or pier. The structure may be shown to support its own dead weight in these circumstances, indicating that catastrophic collapse might not be inevitable.

Mathematical models

Bridge decks

When selecting a method to assess a bridge deck, it is advisable to consider the method used for the design. For example, if an RC deck slab was designed by the strip method, the longitudinal reinforcement should be heavy, and the transverse reinforcement may be light. Analysis by uncracked grillage would then indicate spare capacity longitudinally and the deck would fail transversely. For loading of similar intensity to that originally designed for, assessment by the strip method would be likely to indicate structural adequacy. However, if the current loading is heavier than when first designed, it is possible that an assessment by the strip method will fail longitudinally. Should a generous amount of transverse reinforcement have been provided at design, some transverse distribution over the strip method might allow the deck to pass in both longitudinal and transverse directions. This could be achieved using a grillage with cracked section properties transversely or, indeed, any reduced section properties that will allow the deck to pass. However, inspection is required to ensure that the serviceability of the structure is satisfactory transversely.

Alternatively, yield-line analysis can be used to assess the deck, and this is covered in the section entitled 'Yield lines'. Additional strength can be mobilised on slab or beam-and-slab decks using membrane action, as covered in the section on compressive membrane action. A flow chart for bridge deck analysis is given in Figure 25.12.

For more complex decks, such as box girders, additional strength may be obtained by carrying out a finite-element analysis, modelling the box sides and diaphragms as shell elements. In this way, the three-dimensional aspects of the box can be utilised to provide maximum strength.

However, there is a difficulty that areas of high stress concentration can be obtained using shell or solid finite-element analysis, whereas these would be smeared across the structure when using grillage analysis and failure would not be evident.

Where an area of stress concentration is surrounded by an area of spare capacity, redistribution of stress can occur, and the structure can have adequate ultimate capacity. In that case, unless the structure is brittle or prone to buckling, and provided

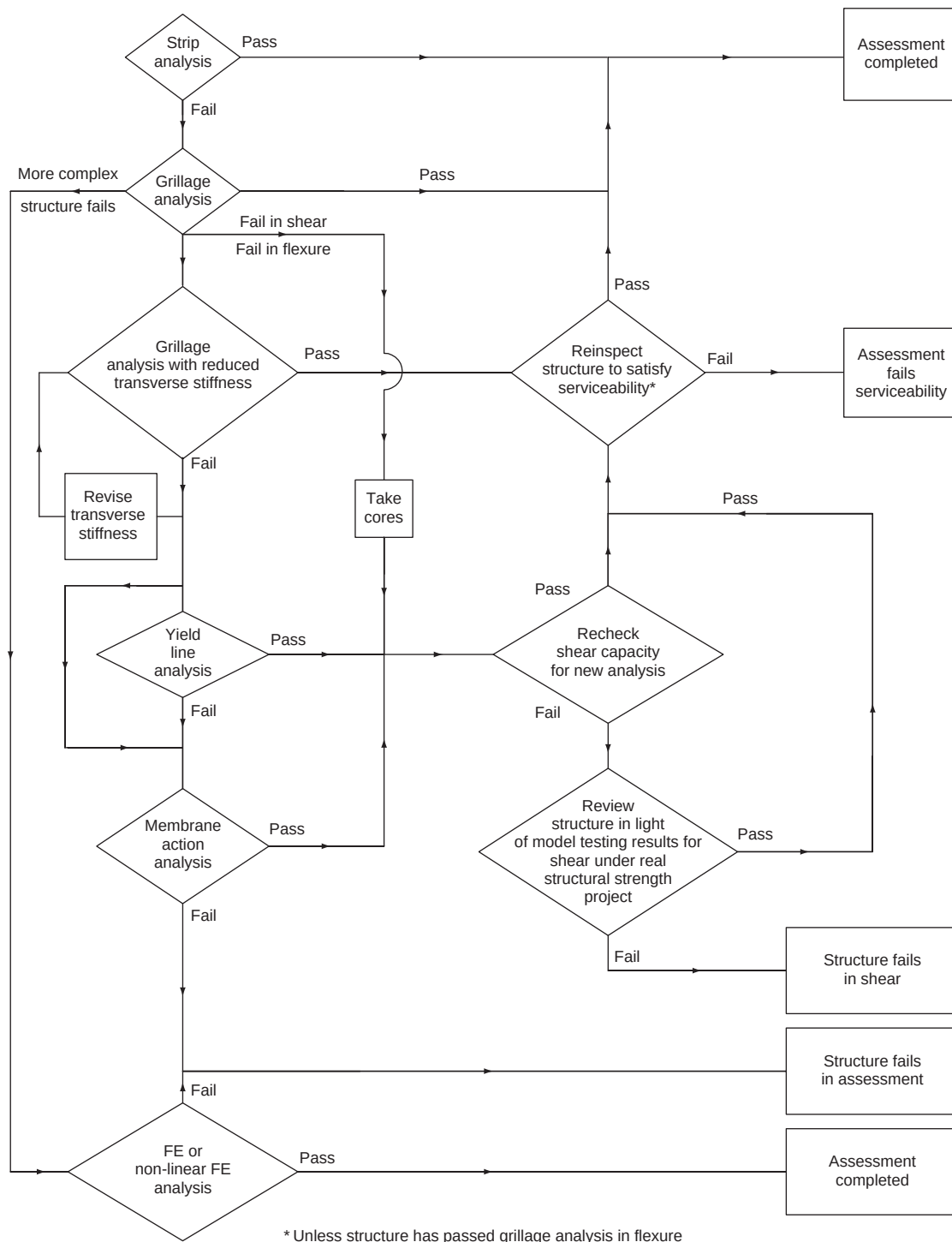


Figure 25.12 Reinforced concrete bridge deck analysis

inspection shows that there is not a serviceability problem, the redistribution should not cause a problem. Where the brittleness derives from concrete in tension, there may be sufficient reinforcement to provide enough ductility, and where it is derived in compression, increased strength may be obtained

if the element is subjected to two- or three-dimensional compression or is at least restrained in the third dimension. The capacity of shaped concrete hinges is a good example of this.

If the structure is prone to buckling, a non-linear finite-element analysis can be carried out to determine a more accu-

rate capacity. This is covered in the section entitled 'Non-linear finite-element analysis and initial imperfections'.

The finite width of bridge piers is a useful means of limiting peak moments in bridge beams and decks supported monolithically by columns (see Figure 25.13).

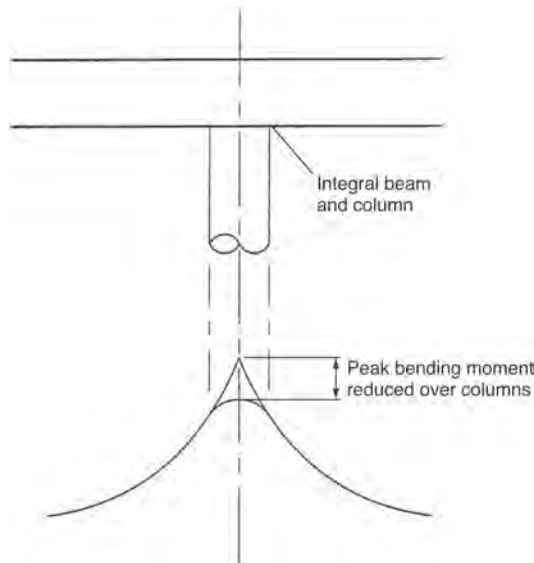


Figure 25.13 Width of bridge pier

Parapets often act to stiffen the edges of decks and can be included in the analysis, provided there is sufficient longitudinal shear connection to ensure that they act compositely with the structure (see Figure 25.14).

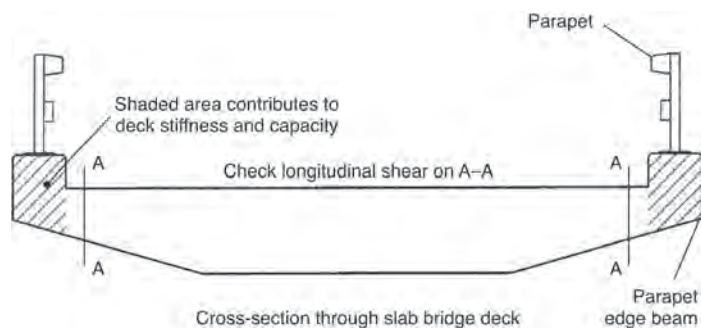


Figure 25.14 Parapet stiffening

Non-linear finite-element models

These are considered in the section entitled 'Non-linear finite-element analysis and initial imperfections', which covers both steel box decks and lengths of jointed viaduct under thermal movement.

Orthotropic action in stiffened web and flange plates

Orthotropic plates

The word *orthotropic* is an abbreviation of *orthogonally anisotropic*, meaning that the elastic properties of a plate are different in two perpendicular directions in its plane. Buckling of orthotropic plates has been extensively treated mathematically. It is thus useful to consider stiffened plates, with stiffening in either one direction only or in two directions, as orthotropic plates, and to use these mathematical equations. Despite the definition of *orthotropic*, plates with the same stiffening in each direction can be treated using the same method.

Overview of orthotropic action

If a stiffened plate with free longitudinal edges is compressed in the longitudinal direction, the whole stiffened plate will buckle as a strut into a single half sine wave in the longitudinal direction. This assumed strut mode is the basis of the longitudinal stiffener assessment equations in national highways CS 456 (HE, 2020e).

If, however, the longitudinal edges are simply supported, as is the case for an internal stiffened web or flange plate, the stiffened plate will buckle into a single half sine wave in the transverse direction and an integral number of half sine waves in the longitudinal direction. This behaviour is usually referred to as orthotropic action and the restraint given to the longitudinal stiffeners by the transverse flexure of the plate can increase their buckling load.

If the transverse flexural stiffness of the stiffened plate is small compared with the longitudinal flexural stiffness, there will be a single half sine wave in the longitudinal direction. If the transverse flexural stiffness is large compared with the longitudinal flexural stiffness or the panel length is large compared with its width, there will be more than one half sine wave in the longitudinal direction. This means that the buckling load of the longitudinal stiffeners will not decrease if the panel length is increased further (Bulson, 1970).

Method of assessment

In flanges, orthotropic action can be considered using the method set out in national highways CS 456 Subclauses 9.10.2.2 and 9.10.2.3 and Annex N (HE, 2020e). The basic concept is to represent the increase in strength from orthotropic action by reducing the stiffener slenderness. This is

achieved by replacing the stiffener's radius of gyration, r_{se} , in the buckling calculation by a modified value. This modified value is essentially the radius of gyration of the stiffener effective section multiplied by $\{\sigma_e/\sigma_{cr1}^*\}^{0.5}$, where σ_e is the Euler buckling stress of the stiffener acting as a strut and σ_{cr1}^* is the modified critical buckling stress for the whole stiffened panel when orthotropic action is considered.

Alternatively, stiffened flange panels can be assessed using the rules given in Section 20 of Part III of the Interim Design and Workmanship Rules (IDWR) prepared by the Merrison Committee (Merrison, 1974). This gives a fuller treatment of the subject, since much of the IDWR formed the basis for BS 5400: Part 3 (BSI, 2000), although the rules were simplified. The Eurocodes may also be used to carry out assessments, provided that adequate account is taken of the imperfections and material properties. The Eurocodes will generally allow for greater flexibility in the assessment methods. The Eurocodes will also generally be more aligned with output derived from modern analysis software.

Webs are generally more complicated to assess because of the presence of high shear stresses, bending stresses and transverse stresses, which all interact to produce a buckling failure mode. Standard CS 456 (HE, 2020e) allows orthotropic action in webs to be utilised using the Annex N expressions.

Benefits

Wide flanges and deep webs, which are longitudinally stiffened without intermediate transverse stiffening between main restraints (e.g. diaphragms or cross-frames) will buckle into a single half wave with a buckling load approaching that of a strut. This is intuitively obvious, since a longitudinal stiffener that is undergoing buckling deformation at the centre of a wide plate will cause little transverse curvature in the plate and will hence be provided with little restraint against further deflection. Orthotropic action is therefore of little benefit in plates that are relatively wide compared with their length in the direction of the major stress.

There can also be a major benefit from utilising orthotropic action where a plate is transversely stiffened but the transverse stiffeners are not stiff enough to restrict the buckling length of the longitudinal stiffeners to the length between transverse stiffeners, in accordance with Section 9.15 of CS 456 (HE, 2020e). If CS 456 were used in this instance, the transverse stiffeners would have to be ignored and the longitudinal stiffeners would be treated as struts with a buckling length equal to the distance between full restraints.

There can also be a benefit where the longitudinal stiffeners on a panel are of different sizes and it is only the smaller stiffeners that appear to be failing. Standard CS 456 does not allow for the restraint of the larger stiffeners. However, this can be allowed for using more complex modelling techniques.

The benefit of utilising orthotropic action for assessing stiffened panels can also be further enhanced by using measured imperfections, where they are better than the tolerances implicit in the design equations.

Example

All of these benefits were obtained in the assessment of Erskine Bridge in Scotland. The bridge is an orthotropically stiffened steel box girder with trough stiffeners on the top flange and bulb flat stiffeners on the bottom flange and webs. A variety of other departures from standard were applied in the assessment, including a modification to Appendix S of BA 56/96 (Highways Agency, 1996) on torsional buckling loads for outstands (Annex S was subsequently amended in BD 56/10 (Highways Agency, 2010)) and the derivation of a bridge-specific assessment live load. Initial assessment showed that the bridge required strengthening to the webs over almost its entire length. After the departures, the extent of strengthening was limited to the longest spans, constituting about one-third of the length of the bridge; within this length, only a relatively small percentage of panels needed strengthening.

Use of Eurocode approach

While not an assessment code, the Eurocode design approach can be applied to assessment and can improved resistances compared with CS 456 (HE, 2020e) if the longitudinal stiffeners can be shown not to be susceptible to torsional buckling.

Non-linear finite-element analysis and initial imperfections

Non-linear analysis

Non-linear analysis incorporating large displacements and material non-linearity (plasticity) can be used to assess the real strength of elements that fail to meet the geometric limitations of design or assessment standards. Many structural analysis programs allow for some form of non-linearity to be considered. For the work discussed here, it is essential to use a program that considers both of the following forms of non-linearity:

- large displacements, also sometimes referred to as second-order displacements
- material plasticity.

Non-linear finite-element methods can also be used where measured deformations are smaller than those permitted by BS 5400 Part 6 (BSI, 1999). Using smaller deformations than those originally required by the specification requirements at the time of construction is likely to lead to greater predicted resistances.

Steel structures designed prior to the implementation of BS 5400 Part 3 (BSI, 2000) may contain elements that do not comply with the geometric constraints of this standard or of its assessment derivative, BD 56/10 (Highways Agency, 2010) (now CS 456 (HE, 2020e)). One such aspect is the use of large web panel openings, for example, manholes (see Figure 25.15), without compliant stiffening around the opening. Large

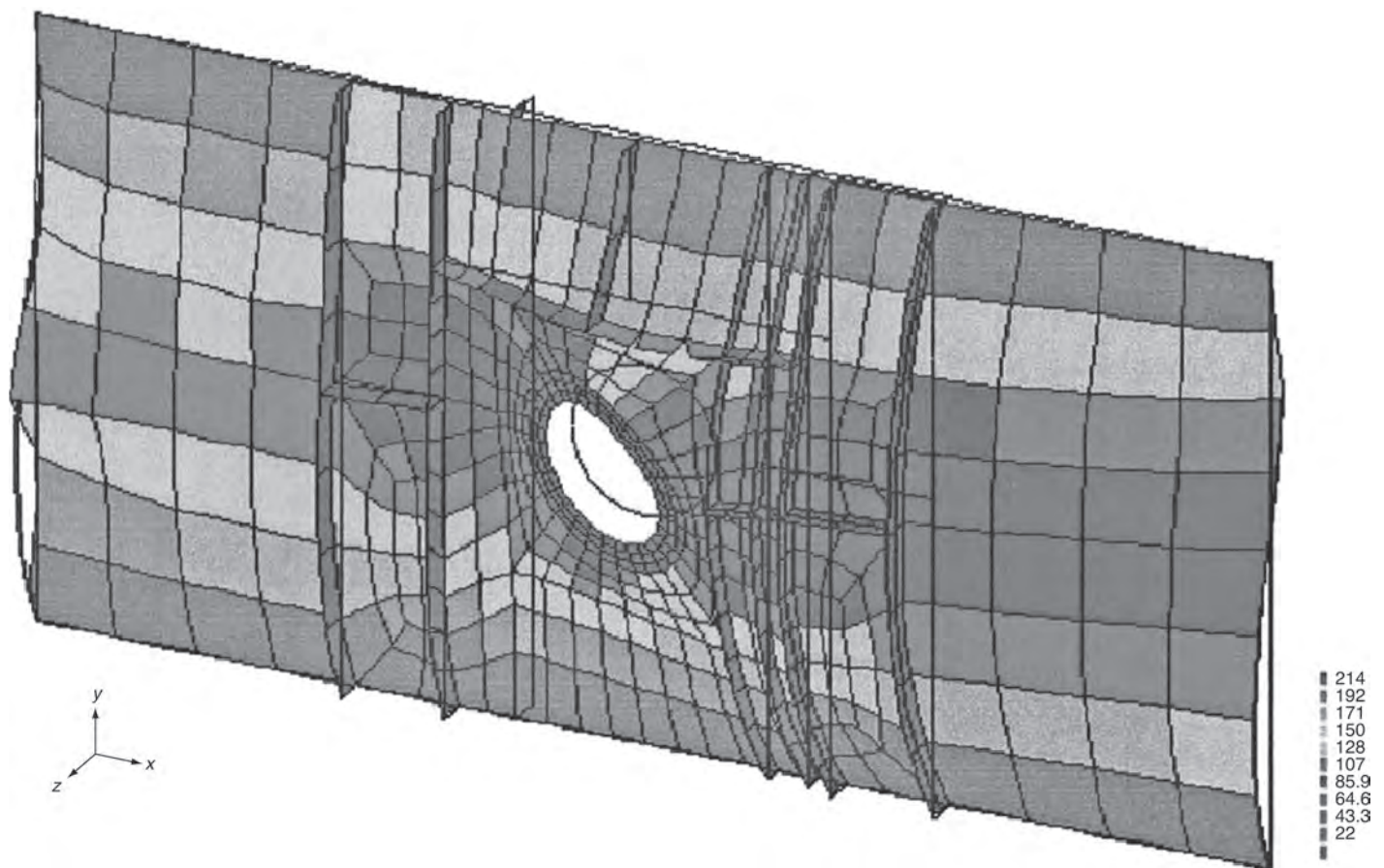


Figure 25.15 Finite-element analysis of web plate around manhole

manhole openings have been identified in both the diaphragms and webs of steel box beams, including structures that have been strengthened following the recommendations of the Merrison Committee (Merrison, 1974).

The webs of beams primarily resist vertical loads, in addition to those produced by bending. The failure of a web may be assessed under two criteria: the development of unacceptably high stresses in the material through yield, or by buckling, rather like a strut. However, unlike struts, webs may continue to bear load after initial buckling; consideration of this action is known as post-buckling analysis. This post-buckling behaviour is beneficial, particularly in respect of shear action, which tends to dominate in web sections. Ultimate failure, or collapse, does not occur until sufficient plasticity has developed in the buckled panel, provided the section is not excessively slender.

The analysis of web panels to establish ultimate collapse load makes use of the elastic and plastic properties of steel, that is, an elastoplastic analysis, and is geometrically non-linear. This form of analysis was used, in conjunction with model tests, to set the permissible values for the design code, taking into account initial out-of-straightness and plasticity. The values have been correlated against model tests, but they reflect the size constraints set up in the code.

Linear elastic finite-element analysis could be used to

determine the stress levels in the section, but this is not able to model stress redistribution. More importantly, for steel construction it would not be possible to investigate buckling effects. Therefore, linear elastic finite-element analysis often predicts localised stress concentrations, which would be redistributed plastically within the structure. It should also be noted that codified formulae for ULS resistances generally allow for this plastic redistribution, and therefore linear elastic shell models can often lead to more conservative predictions than the codified formula.

To model the effects of both buckling and stress redistribution, it is necessary to use non-linear analysis techniques. To model both the buckling and stress redistribution, the non-linear analysis program must be able to consider both geometric non-linearity (large displacements) and material non-linearity (elastoplastic).

Advantages of non-linear analysis

It is well recognised that large plates have significant additional resistance to offer loading even after initial elastic buckling. Non-linear analysis allows full use of membrane action and post-buckling strength.

The classical elastic analysis of plates can be used to determine critical buckling loads. However, elastic analysis becomes invalid when yield takes place and the plate is still below its ultimate load.

The ultimate limit state or collapse of the panel is reached when a sufficient area of the panel becomes plastic. The extent of this area is greatest for stocky panels and smallest for slender panels (as defined in BS 5400 Part 3 (BSI, 2000) or CS 456 (HE, 2020e): where using the Eurocodes, class 1 cross-sections have the greatest facility for plastic redistribution, with class 4 sections having least facility). It is not readily possible to determine an equivalent slenderness value of panels containing large openings. The codes therefore limit the maximum size of opening in an unstiffened panel.

Non-linear analyses are far more complex than linear elastic models. They require knowledge of the initial imperfections and can be unreliable if not carried out properly. They also require far better understanding of structural behaviour; it is far more difficult to identify at what point structural failure occurs, as this depends on the structural behaviour and the degree of redundancy in the structure. They are expensive in comparison with linear elastic analyses but are usually economic compared with the cost of strengthening.

Non-linearity

In linear analysis, it is assumed that displacements and rotations are small, stress and strain are proportional and loads maintain their original directions as a structure deforms. However, deformations in the structure may become large enough to change the way the load is applied or resisted in the structure. In addition, the material may lose stress or strain proportionality and become plastic; the stiffness of the entire system then changes with load increment.

The analysis is carried out in a two-stage process. Initially, a linear elastic buckling analysis is carried out to determine the buckling modes of the model under a given loading. This also allows scrutiny of the model under elastic conditions; modelling errors are more easily detected under this more familiar form of analysis. The analysis generates eigenvectors, or buckled shapes, which, through inspection, are selected to form the basis of an initial perturbed, or buckled, shape for the non-linear analysis. The eigenvectors are chosen to produce the most onerous initial buckled shape in the panel or member under consideration. Selection of eigenmodes is discussed later.

The non-linear analysis starts from the assumption that an unloaded structure is initially deformed in the manner described previously. The loading is applied to the model in small increments and the stiffness matrix is automatically updated by the program at each stage. Essentially, each increment can be thought of as a single linear analysis, with the exception that the matrices are revised to account for the geometric and material changes at each load increment, hence the requirement for small increments in load.

Initial deformed shape

The buckling strength of a structure is affected by the number of initial deformations of the components from a flat plate; BS 5400 Part 6 (BSI, 1999) gives limiting construction tolerances for various parts of steel structures. The design rules of BS 5400 Part 3 (BSI, 2000) make some further allowance for initial imperfection.

When considering the real strength of an existing structure, it can be beneficial to use the measured deformations rather than those from BS 5400 Part 6. The measured values should be adjusted in accordance with CS 456 (HE, 2020e) for use in analysis. The recommendations of Clause 9.15.6.2 are considered to be equally applicable to other elements of a structure in this respect. It should be noted that there may also be instances where the measured deformations exceed those given in BS 5400 Part 6. Some initial deformation of the structural model, whether based on measured or allowable values, must always be included when assessing a structure by means of non-linear analysis. Methods of applying deformations to the structural model are discussed next.

A common method of generating the perturbed or initial deformed shape is to use eigenvector analysis. Initially, the structural model is created based on the theoretical dimensions. Loads are applied to this model to produce critical loadings on the elements being assessed. An elastic eigenvalue analysis is then conducted for the structural model. It is usual to extract the first 10–15 eigenvalues from an analysis. The lowest positive eigenvalue is representative of the buckling mode shape that is most likely to occur. The lowest positive values may be located away from the area of interest and may, in part, be induced by end constraints on the model. All modes should be reviewed, preferably by viewing graphical representations of the mode shape, and critical cases selected for consideration in the non-linear analysis. Negative values are associated with load reversal and will not, in general, be of further interest. If no relevant positive modes exist within the selected set, it may be necessary to reanalyse to extract further modes, following a careful check on the model and its loading. Figure 25.16 shows typical eigenmode plots.

For the non-linear analysis run, the original model coordinates are replaced by 'perturbed' coordinates, which reflect the initial (construction) deformations, either measured or from BS 5400 Part 6. These perturbed coordinates are obtained by mapping the peak out-of-plane deformations to the maximum distortion of the chosen eigenvectors. A separate non-linear analysis will be required for each selected eigenvector.

Perturbed shapes can also be simulated by applying out-of-plane loading to the selected panels, the derived deformation then being factored as before. While this will give a compatible set of deformations, it may not fully reflect the buckling mode.

The solution of non-linearity is more demanding than linear problems, since it may not be possible to predict how the model will behave or the critical features of the analysis. At the onset, the types and extents of non-linearities may not be apparent, and a solution must be subject to an iterative process.

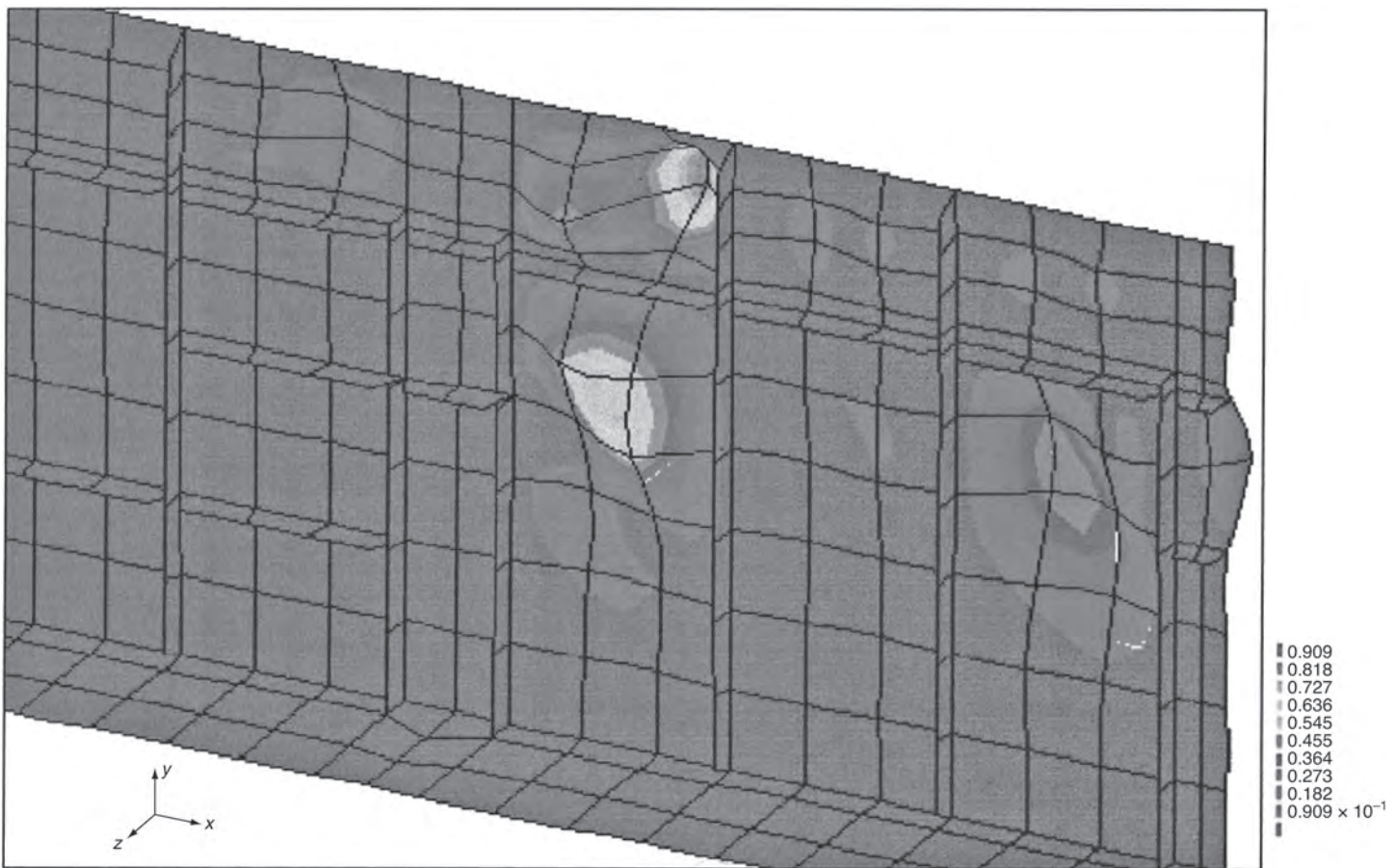


Figure 25.16 Typical eigenmode plots

The structural model

Conventional linear finite-element analysis attempts to solve the equation

$$\mathbf{K}\mathbf{D}=\mathbf{R} \quad (25.4)$$

where $\mathbf{K}$ is the stiffness matrix, $\mathbf{D}$ is the displacement matrix and $\mathbf{R}$ is the force matrix.

Under this situation, displacement is proportional to applied force. In a non-linear problem, $\mathbf{K}$ is a function of $\mathbf{D}$. Numerical methods are unable to solve non-linear equations explicitly for $\mathbf{D}$ as a function of $\mathbf{R}$. Non-linear problems are therefore solved in a series of steps.

The general approach for solving these problems is to apply incremental loading to the model and use linear approximation methods to solve at each stage. It is apparent that computing power is the limiting factor with this type of analysis. In effect, a complete analysis is being carried out at every load increment, in addition to the work required in meeting convergence criteria at each stage. Non-linear analysis must be carried out for a single loading case and superposition of load case results is not applicable.

As non-linear analysis requires models to be reanalysed many times, minimising the model size therefore has greater importance than for linear analysis. As such, it will rarely be possible to model the whole structure, and smaller parts should be isolated. It is very important to ensure that the resulting model is representative of the structure in terms of edge constraints and loadings. It is also necessary for the model to continue an adequate distance from the area of interest to ensure that end restraint effects are minimised on the zone of interest. When creating this type of model, most modern software will allow a combination of beam and shell elements to be used, where beam elements perform a useful method for restraining the more detailed shell model, while providing a good representation of the structural behaviour outside the main area of interest.

The analysis procedure involves incrementally increasing the load until either a specified load level is achieved, or the model fails to converge, owing to formation of an unstable mechanism. Concentrated point loads, such as external constraints, may induce a premature local failure and must be eliminated, either by local stiffening – taking care not to change the stiffness of the areas of interest – or by using appropriate distributed rather than point loads.

Partial factors and load increments

Since this analysis is considering the yield and buckling failures, it is obvious that the reference loadings will be those resulting from ultimate conditions. Much discussion surrounds how the partial factor γ_{f3} and the material factor γ_m should be applied. It is important to ensure a consistent approach. If nominal yield values (σ_y) are used for material properties with $\sum Q\gamma_{fi}$ for the reference load, then the analysis must achieve a load increment of $\gamma_m\gamma_{f3}$ to be able to safely carry the designated loads. Since the code does not use a consistent value of γ_m throughout, this fact also needs to be considered when deciding the approach to partial factors.

The specified loads are those resulting from ultimate conditions; therefore, it follows that some part of the model may be at yield at a load increment of 1.0. For this reason, the first few load increments should be below unity to allow plasticity to develop gradually. A starting load increment of 0.5 is suitable for most applications. The elastic (eigenvalue) analysis can be used to determine the extent of stress levels above yield.

Interpretation of results

The elastic (eigenvalue) analysis can be used to carry out basic checks on model behaviour and load resultants. A review of the mode shapes can also indicate inaccuracies in the model.

For the non-linear run, the results must be carefully inspected. It can sometimes be difficult to establish the exact failure mode, that is, whether it is buckling or yield. It is best to graphically review the development of stress and of displacements. Non-linear analyses may fail, owing to very localised inadequacies, which might be real defects in the structure or modelling deficiencies. Such localised failures might be remote from the area of interest and will often necessitate a modification to the model and reanalysis. This analysis will, in general, only be modelling the capacities of the plates and stiffeners. It is essential to ensure that connections can still transmit the forces between components.

Other uses of non-linear large-displacement elastoplastic analysis

The preceding discussion has been primarily concerned with manholes in web panels without compliant stiffening. Elastoplastic analysis of large displacements can also be used where panel deformations exceed those allowable by BS 5400 Part 6 (BSI, 1999). Similarly, the method can be used to assess additional strength when deformations can be shown to be smaller than Part 6 values.

Another example is interaction between components. Design code rules consider the design of components individually. There are often instances where one structural element is overstressed according to the code but adjacent components have a reserve of strength. An example of this is in the design of a load-bearing diaphragm. The code requires the out-of-plane

moments owing to bearing eccentricity to be carried fully on the diaphragm. Rotation of the diaphragm will also cause rotation in the bottom flange, hence reducing the level of moment that must be carried by the diaphragm. At a support, the bottom flange will be in compression, so this rotation may reduce its buckling capacity. Shedding of moment into the flanges cannot therefore be arbitrarily performed but it can be anticipated that this will improve the load-carrying capacity. This interaction will also affect the distribution of stress into the web.

Non-linear analysis can be used to assess this problem (see Figure 25.17). The process will be similar to that detailed previously for manholes, but it is often more difficult to determine the critical eigenmodes. Different critical mode shapes might exist for the diaphragm and flanges. Since these will be separate physical elements, their initial deformations will be independent; it is therefore appropriate to determine perturbed coordinates separately from differing mode shapes for each component.

Non-linear analysis for thermal movement of steel–concrete composite viaduct where bearings have increased friction

Non-linear analysis can be used to determine the longitudinal forces and movements in a viaduct, where sliding bearings are worn or rusted, resulting in increased friction.

Live load fretting, the small reciprocal movements on bearings caused by live load flexing the beams, can wear the sliding surface of bearings down to the underlying material of higher friction coefficient.

The effect of graphite lubrication on steel sliding bearings can disappear over time and the bearing surfaces can develop rust, leading to a gradual increase in the friction coefficient.

The effect on the viaduct of the increased friction coefficient is to develop much larger longitudinal forces under thermal effects, until the friction coefficient is overcome and the bearing slips. It then sticks until the thermal forces again build up and force the movement in either the same or the reverse direction. This phenomenon is known as stick–slip.

The thermally induced forces can be significant, and cause strains in the deck and substructure, which, to an extent, can relieve the expansion or contraction movements in the viaduct. Nevertheless, the resulting stresses in the deck or substructure might be significant and can adversely affect the structural integrity. Calculation of these is complex as the results depend on the extent to which the bearings slip and stick. Here, non-linear analysis can be used to determine which bearings slip and by how much.

A model of the viaduct can be set up, representing the decks and substructures in a two-dimensional frame along the length and height of the viaduct. The different materials are given their appropriate Young modulus. As the bearings act at the deck soffit, the bottom flange of the deck beams is principally

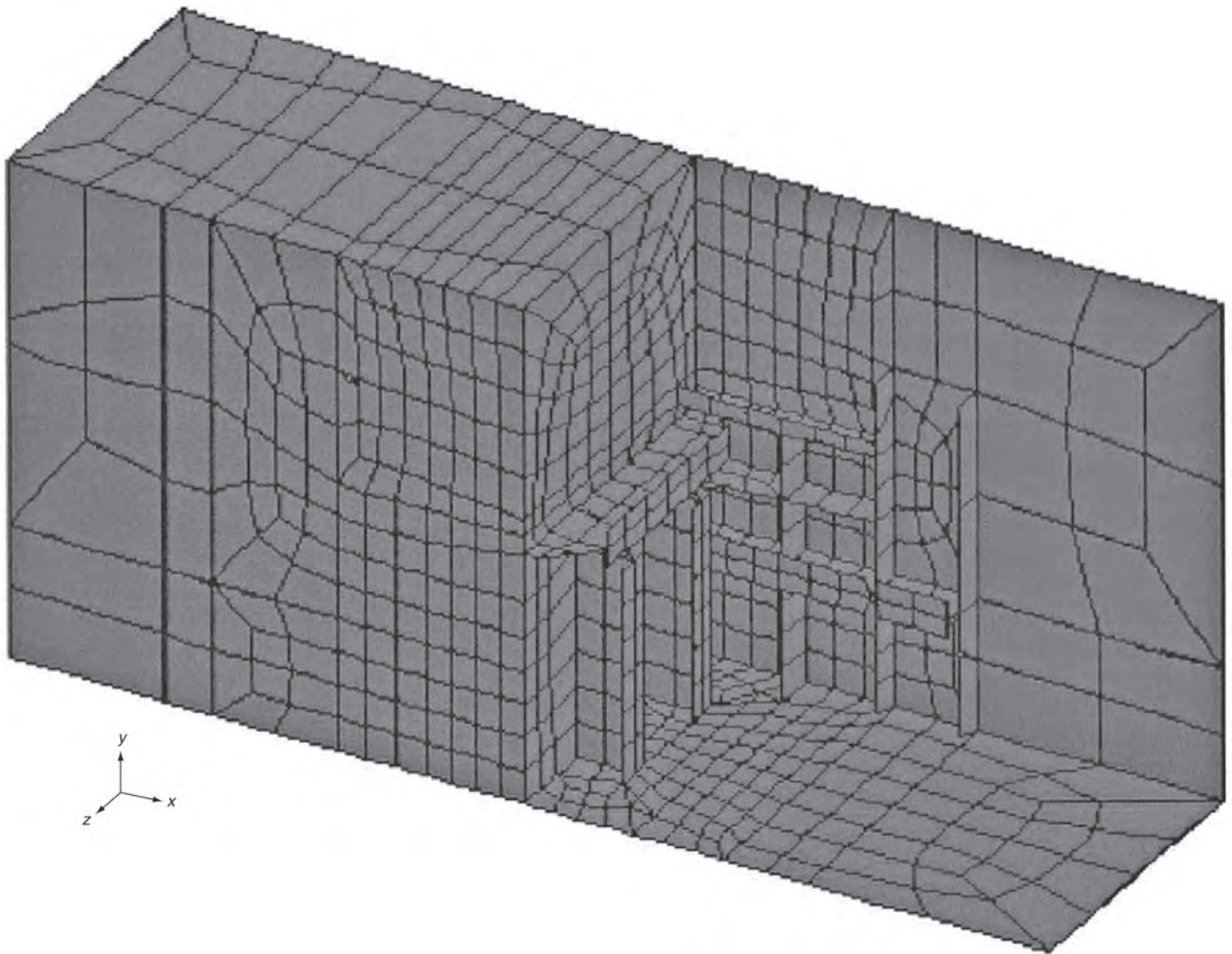


Figure 25.17 Finite-element analysis of diaphragm in steel box girders

stressed when the bearings stick and this results in flexure of the beam. The appropriate bending stiffness of the beams and deck needs, therefore, to be inserted in the model in addition to the longitudinal axial stiffness.

When the bearings do not slip, the decks can sway the top of the piers longitudinally; therefore, the bending stiffness of the piers must be included in the model. The bearings are represented by elements with non-linear characteristics, which are rigid until the force at which the bearing slips is exceeded. The element then moves until the force reduces to that at which the bearing no longer moves.

The viaduct model is subjected to incremental temperature increases or decreases to determine which bearings slip, by how much, and what force actions are developed in the elements of the viaduct under the varying temperature.

The integrity of the viaduct can then be ascertained under extremes of temperature, until such time as the bearings can be lubricated, maintained, renovated or replaced, as appropriate.

First principles

Strut-and-tie action

Strut-and-tie action has been used for the analysis of pile caps and anchorages (Leonhardt, 1965; Schlaich *et al.*, 1987). Eurocode BS EN 1992-1-1 (BSI, 2015) provides guidance on the use of strut-and-tie rules that can be used as a basis for assessment with agreement of the overseeing organisation; the approach can give benefit for pile caps, diaphragms, bearing areas and other details affected by local stresses.

Example: column head

An RC column of diameter D supported a steel box cross-beam carrying a motorway deck through a steel line rocker bearing (Figure 25.18). Below the top solid section of depth H of the column, the column was hollow, with RC walls of thickness W . Two layers of hoop reinforcement were provided in the solid section as anti-bursting reinforcement. The outer reinforcement was severely corroded and it was necessary to establish whether the inner hoops could resist the bursting.

The vertical load was assumed to be carried uniformly around the perimeter of the hollow section, and the resultant from each half of the column annulus was calculated to act at X , as shown in Figure 25.18(b). This position, being inwards of the thin walls, meant that when considering strut-and-tie action,

as illustrated on the column elevation, a lower horizontal force was generated than otherwise might have been considered. Since the bearing was a line rocker bearing, the critical mode of failure was transversely across a vertical plane through the longitudinal centre line of the bearing. From strut-and-tie action, a calculation of the triangle of forces provided the transverse tension T and hence the load in the hoop reinforcement, $T/2$, as shown on the plan.

It was established that the inner hoop reinforcement could carry the bursting. The upper part of the column was then strapped with steel rings to carry the bursting forces to compensate for the inner hoops beginning to corrode.

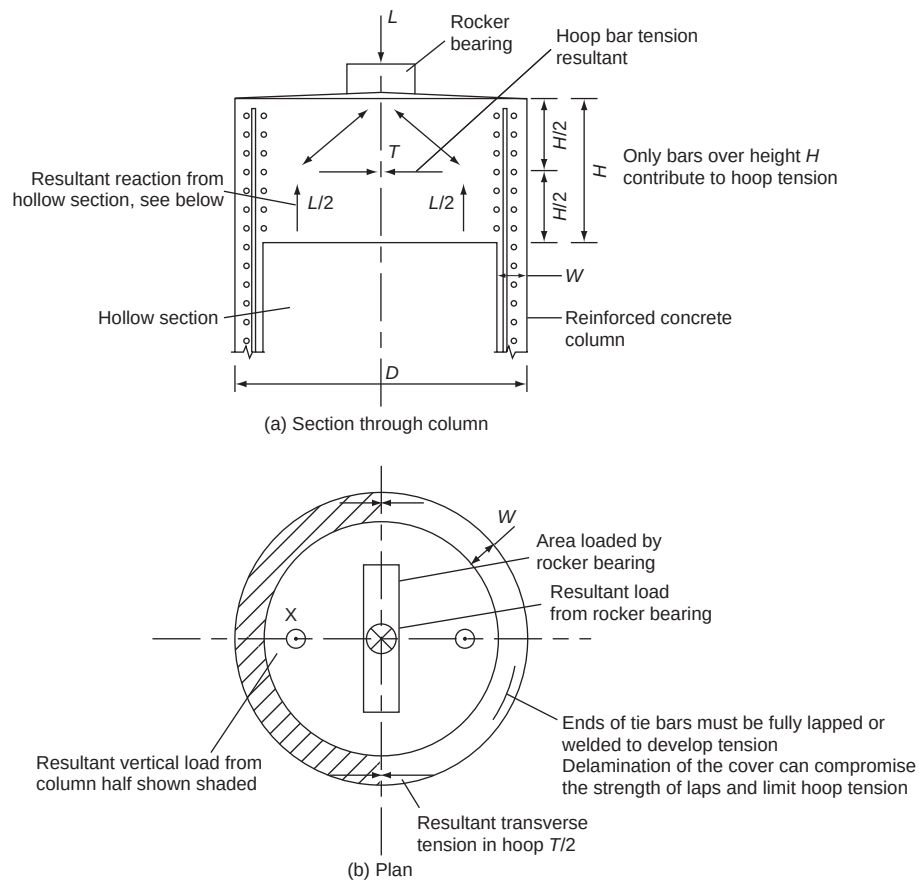


Figure 25.18 Strut-and-tie action

Regan's method: determination of actual force in reinforcement

The force in a reinforcing bar in a beam depends on both the bending moment and the shear force. This can be illustrated simply in a cantilever situation, as shown in Figure 25.19. (A similar effect occurs in a simply supported or continuous beam (Regan, 1985).)

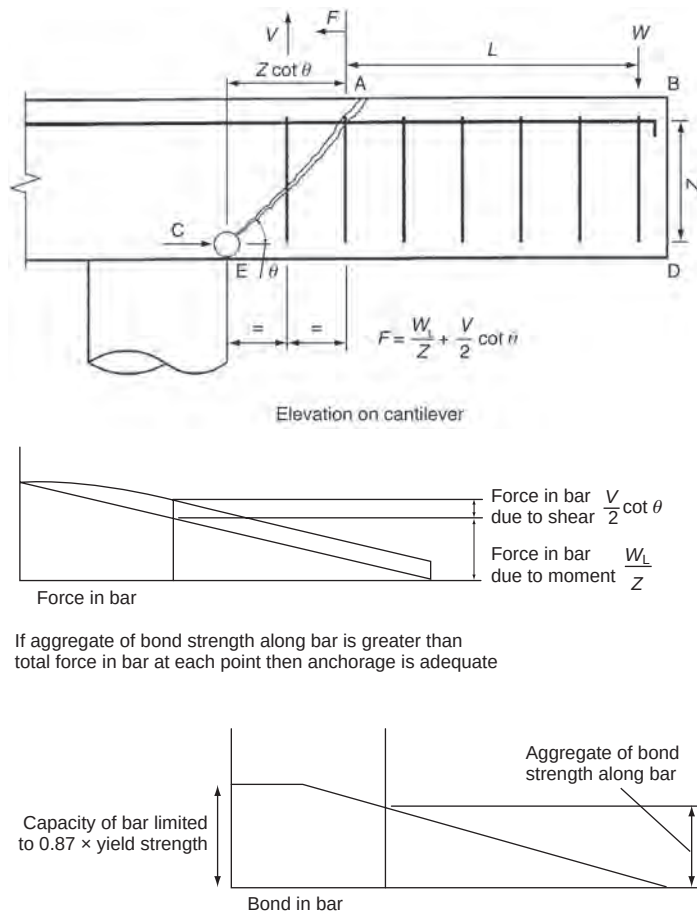


Figure 25.19 Regan's method

Assume that the cantilever fails by the development of a shear crack inclined at an angle θ such that the shear force V is just carried by the capacity of the links crossing the shear plane, and that the load W on the cantilever tip is just supported by a force in the main reinforcement F at the crack.

For equilibrium of the cracked cantilever ABDE, taking moments about the centre of concrete compression C above D :

$$W(L + Z \cot \theta) = FZ + \frac{VZ}{2} \cot \theta \quad (25.5)$$

The bending moment M at the point in the reinforcement where F is measured is equal to WL , therefore

$$M + WZ \cot \theta = FZ + \frac{VZ}{2} \cot \theta \quad (25.6)$$

$$F = \frac{M}{Z} + W \cot \theta - \frac{V}{2} \cot \theta \quad (25.7)$$

For vertical equilibrium, $W = V$:

$$F = \frac{M}{Z} + \frac{V}{2} \cot \theta \quad (25.8)$$

that is, the force in the reinforcement is a function of both the moment and the shear force.

For the first assumption to be true, the shear failure plane crosses the number of links that carry the shear force.

If A_{sv} is the cross-sectional area of all the stirrup legs at each stirrup location, f_{sv} is 0.87 times the yield strength of the stirrup reinforcement, and S is the stirrup spacing along the beam, for vertical equilibrium:

$$V = A_{sv} f_{sv} \frac{Z \cot \theta}{S} \quad (25.9)$$

Therefore,

$$\cot \theta = \frac{V}{A_{sv} f_{sv}} \frac{S}{Z} \quad (25.10)$$

Equations 25.8 and 25.10 are also applicable for simply supported and continuous beams; $\cot \theta$ shall not be taken as less than 1 or greater than 3.

The peak bending moment over a support is, however, never exceeded, as the angle of the shear failure plane is limited by the physical presence of the support, that is, when F is measured at the support, $\cot \theta$ is zero. The maximum bending moment in the span is never exceeded either, as at that point the shear V is zero.

Having calculated the force in the reinforcement, the adequacy of the anchorage to the curtailment point can be determined by summing the bond stress along the bar from the point at which the force is measured to the end of the bar. Where the cover is delaminated, the bond strength is significantly reduced, thereby correspondingly reducing the effectiveness of the anchorage.

Once the force in the reinforcement is known, the aggregate of the bond along the bar from its end to the point at which the force in the bar has been calculated can be determined. Again, this is a more exact analysis than is given by the curtailment rules in the code. The maximum benefit from the bar anchorage is therefore obtained.

Should the cover to the bar be delaminated, the delaminated bond strength should be used. This may indicate that there is insufficient anchorage for the bar. However, in using this more exact method of analysis, account will have been taken of the maximum strength available.

This method was used extensively in the assessment of cross-beams for the Midland Links motorway viaducts in the UK, and a departure from standard agreed. The method can be applied to spans as well as cantilevers.

Non-linear analysis of slender piers

Slender piers may appear inadequate when assessed to CS 455 (HE, 2021b); the method of analysis is likely to have changed from that assumed in the original design, in addition to a likely increase in vehicle live loading. The problem can also be due to the inherent conservatism in the CS 455 assessment approach.

The CS 455 (and BS 5400 Part 4 (BSI, 1990)) analysis requires additional moments to be added to account for buckling effects. This additional moment is derived by considering the deflections at collapse and multiplying the axial load by this resulting eccentricity. The additional eccentricity is based on the strains, and hence curvature, in the pier at failure when the steel yields and the concrete crushes. The resulting analysis can therefore give a good prediction of the moments and deflections at final failure of the pier. However, the peak load in the pier may be achieved at a deflection that is considerably smaller than this.

To determine the actual capacity of a slender pier subject to axial load and applied moment or transverse force, or both, it is possible to perform a non-linear analysis of the pier. Standard CS 455 permits this through reference to BS EN 1992-1-1 Clause 5.8.6 (BSI, 2015). The non-linear analysis must model both the geometric (large-displacement) non-linearity and the material non-linearity of the reinforced concrete section. Commercial software is available to model both of these. (An alternative to using a package that represents material non-linearity is to perform an iterative analysis in which the displaced shape is updated at each iteration to take account of large displacements and the section properties are revised as elements become plastic, continuing the iterations until the displacements converge. This is, however, tedious and time-consuming.) The column should either be modelled with an initial bow or the load should be given an initial eccentricity to account for the actual pier construction tolerances.

To illustrate the benefit of a non-linear analysis, consider a simple cantilevered pier subject to both axial load, P , and a transverse load, F , at the top. If the load P is applied as a constant load and the top of the pier is deflected under deflection control, the curve in Figure 25.20(a) is obtained.

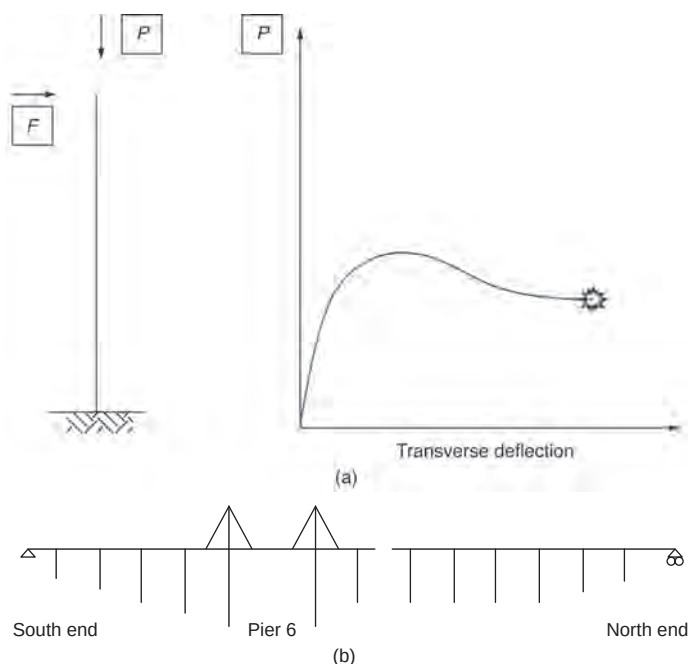


Figure 25.20 Slender piers

Initially, the behaviour is linear, as the concrete is approximately linear in compression. As the force F increases, the concrete cracks in tension and its tangent modulus in compression progressively reduces. Initially, the moment resistance of the section increases faster than the increase in moment, resulting from the growing eccentricity of the load. Consequently, the load F increases with deflection. However, as the pier becomes more flexible, there comes a point at which the rate of increase of moment resistance equals the rate of increase in the moment resulting from the growing eccentricity. This represents the peak in load F in Figure 25.20(a).

A real pier with this combination of applied axial and transverse load would fail at this peak load point. The modelled pier loaded under displacement control, however, would exhibit a drop in transverse load F until final failure occurred, when the concrete reached its crushing strain. This final failure point is likely to be close to the failure load predicted by CS 455. Depending on the pier geometry and reinforcement, the real failure load at the peak of the graph can be considerably higher than that predicted by CS 455 (Jackson, 1985). A partial factor of safety appropriate to upper bound methods should be used with this method of analysis.

Example

This technique was used in the assessment of Erskine Bridge, in Scotland. The bridge comprises 15 spans of orthotropic steel box girder with a 305 m cable-stayed main span and an expansion joint near pier 8. The southern half of the bridge is restrained against movement at the south abutment and the deck is pinned to piers 1 to 7. The piers are lozenge-shaped, with a width of approximately 11 m and a maximum thickness of approximately 2 m. The piers supporting the main span are approximately 50 m tall (see Figure 25.20(b)).

Under direct temperature loading, pier 6 deflects approximately 250 mm. Initial assessment to CS 455 (HE, 2021b) showed that pier 6 was overstressed by approximately 40% under combined temperature and assessment live loading. The moments due to temperature deflection and slenderness effects were added, although this was clearly conservative.

The southern half of the structure was reanalysed under the same load case using the iterative method described previously. The pier was split into several members within a computer model. Large-displacement analysis was carried out and the resulting moments and axial loads were used to determine the strain distribution in the pier for each member of the computer model. These strains were used to determine a new effective inertia for each of the pier members and the structure was reanalysed. This process was continued until convergence occurred. The revised analysis showed that the pier was, in fact, just capable of resisting the assessment live loading.

Recommendations

Going back to first principles can tailor the analysis of a structure to its actual configuration and therefore develop maximum

strength from it. The examples given of strut-and-tie action and Regan's method are useful examples of this and have been used in practice to avoid lane closures on the Midland Links motorways. The analysis of slender bridge piers has enabled the columns of Erskine Bridge to pass their assessment. The Eurocodes provide a framework for some of these analyses such as strut-and-tie and slender pier non-linearity.

Yield lines

Introduction

The following is a brief overview of yield-line analysis, following experience of both traditional and up-to-date methods of using this technique. Yield-line analysis is a long-established method of using plastic methods of analysis of RC slabs to determine greater strength than from an elastic approach. It was pioneered by KW Johansen in his doctoral thesis in 1943 (see Johansen (1972)). In current practice, yield lines are rarely used, as shell models tend to be preferred. Yield lines can, however, produce improved assessment ratings, provided that appropriate checks are in place to ensure that an appropriate yield-line model is selected.

Geometrically compatible plates of a bridge deck are deflected under load. Each plate is bounded by straight lines and the boundaries form plastic hinges with the reinforcement yielding (see Figure 25.21). The work done in deflecting the load is equated to the work done in yielding the reinforcement along the plate boundaries (see Figure 25.22). This method provides an upper bound solution because it is necessary to examine every combination and permutation of plate configurations to determine the failure mode requiring the lowest ratio of applied work to internal work. The method can also be applied to continuous and trapezoidal slabs.

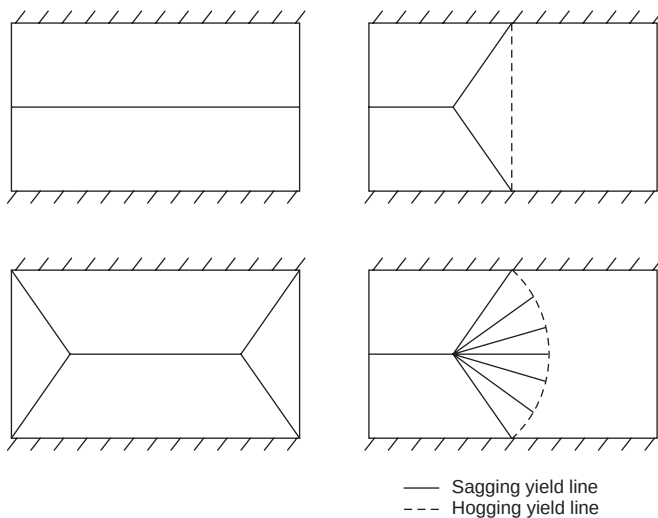


Figure 25.21 Typical yield-line patterns

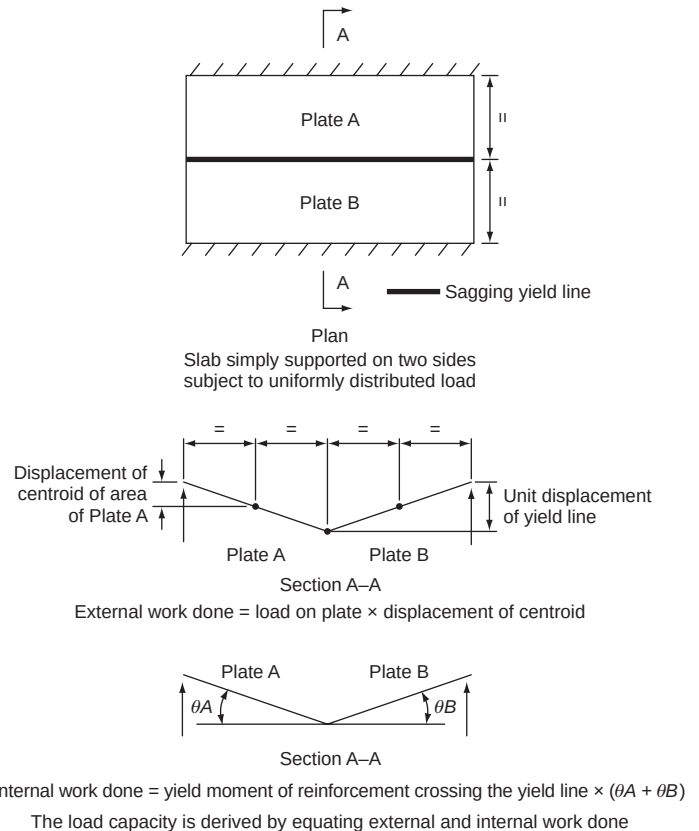


Figure 25.22 Simple example of yield line

Limitations of the yield-line method

Yield-line techniques require full ductility of the reinforcement; therefore, checks are required to ensure that the rotational capacity of all the yield-line sections is adequate. Thus, it is necessary to check that the section is under-reinforced and that the strain in the reinforcement is adequate at ultimate load. Furthermore, the bond strength must be adequate to any reinforcement curtailment points beyond the yield lines to ensure that premature bond failure will not occur.

Should any decks lift off their bearings in developing yield-line failure, the work done in lifting the deck should be used, rather than the work done in developing the moment in the relevant yield line. Where large voids are present in the slab, it is necessary to check that excessive local bending moments are not developed in the top and bottom flanges through transverse shear.

When yield lines are used for beam-and-slab bridges, owing to redistribution, global flange forces or global transverse moments in the deck slab can be ignored when assessing local strength, except in slabs that are highly stressed under both global and local effects. In that case, the brittleness of local failure modes can prevent redistribution. With concentrated loads, the failure mechanism is often formed as a fan. The work

done against torsion in the deck beam should be ignored; when the slab is used to enhance the plastic moment capacity of the beam, the longitudinal moment capacity in the slab should be ignored in the work done against yield lines in the slab.

Analysis programmes

This method requires a large number of hand calculations to check all the possible failure configurations. Most analysis packages do not cater for this method and, possibly as a result, its use has declined in recent years. The task can be accomplished by computer and can therefore be carried out quickly and economically.

Examples of yield-line analyses of culverts and bridges in Essex, UK

The eight culverts listed in Table 25.1 were reassessed by yield-line analysis and seven were found to have a capacity of full HA loading and 37.5 units of HB loading.

Culvert	Elastic analysis		Yield-line analysis (YLA)	YLA factor
	Applied moment	Resistance moment		
1	186	349	Pass	1.02
2	200	127	Fail	–
3	No data	No data	Pass	1.65
4	87	50	Pass	1.64
5	No data	No data	Pass	1.87
6	133	82	Pass	1.13
7	203	172	Pass	1.90
8	349	214	Pass	1.16

Table 25.1 Culvert assessments

Culvert 9

A ninth culvert was originally assessed as having a capacity of group one (fire engine), limited by flexure of the deck slab.

A yield-line analysis capacity of full HA loading plus 37.5 units of HB loading was determined, with a factor of safety of 1.25.

Bridge

The structure consisted of a beam-and-slab-type deck with an in situ deck slab spanning between universal beams with precast concrete encasement to bottom flange. The structure was originally assessed at a capacity of group one (fire engine), limited by the flexure of the deck slab. Yield-line analysis reassessment passed the structure for 40t loading.

Shear

The yield-line method assesses bending capacity; it does not deal with shear capacity. Shear capacity is normally checked by both punching shear and the shear's output from a standard elastic analysis, for example grillage. More recent research has concentrated on plastic methods for the assessment of shear, but the work is complex (Nielsen, 1984).

Serviceability

Being a plastic method of analysis, the yield-line method provides an ultimate bending capacity. Crack control and consequent durability are not covered. An elastic analysis should be carried out to determine the serviceability capacity, but the deck might be found to be overstressed. In practice, inspections have shown that slabs failing their ultimate capacity by elastic analysis but passing by the yield-line method might not be severely cracked, and their serviceability condition may be deemed to be acceptable based on the inspection. In such a case, continued monitoring to ensure durability is advised.

Recommendations

The yield-line method, although long-established, has not been used as widely as it might have been, owing to engineers' reluctance to rely on an upper bound technique that requires many analyses to ensure that the critical failure mode has not been missed, with the attendant risk that an unconservative solution has been obtained. Partial safety factors appropriate to plastic methods must be used for yield-line analysis.

The method may be limited by failure to comply with serviceability criteria – cracking and large deflections, which are best judged by inspection rather than calculation. However, serviceability failure is essentially a durability issue and can be addressed by increased protection rather than load restriction or replacement or repair (Cambridge Enterprise, 2014; Clark, 1984, 1994; Cope and Clark, 1984; Hillerborg, 1991; Johansen, 1972; Jones and Wood, 1967; Middleton, 1993, 1997).

Compressive membrane action

Introduction

Tests of loads on RC slabs restrained around the edges have recorded capacities four or more times greater than those derived from bending theory. The reason has been identified as the presence of inclined compressive struts in the concrete, providing an arch or dome effect (see Figures 25.23 and 25.24). The phenomenon is called compressive membrane action (CMA). The methods are underutilised because of the lack of usable methodologies and guidance.

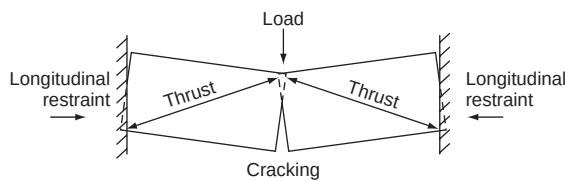


Figure 25.23 Arching action in concrete element

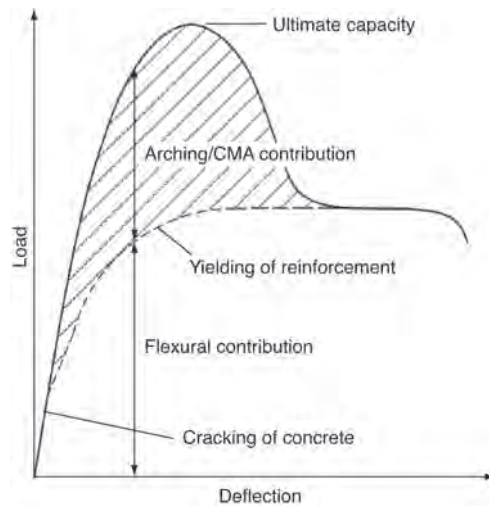


Figure 25.24 Interaction between flexural and compressive membrane action

Restraint

The restraint can be provided in several ways. In floor slabs or cellular structures, it can arise in internal slabs, as a result of the adjacent slabs around the perimeter acting as a continuum or as a result of a stiff frame surrounding the slab. In bridge decks, the restraint can be provided in the longitudinal direction by the bending stiffness of the abutment walls where the slab is connected to the abutment. In the transverse direction, the restraint is provided by the adjacent area of deck, assuming that friction or fixity prevents the deck sliding along the abutment. Restraint can also be provided internally by the reinforcement.

Compressive membrane action can form in deck slabs between deck beams, either where the beams are positioned below the deck or where (as in steel or wrought iron in-fill

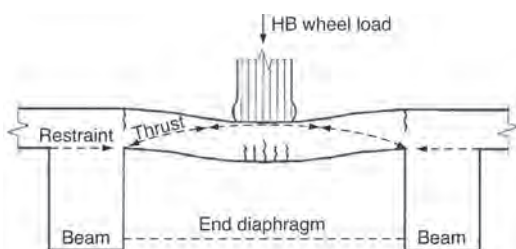


Figure 25.25 Compressive membrane action in bridge deck

beams), they are cast into the slabs (see Figures 25.25 and 25.26). In either case, restraint of the deck, generally at the abutment, is required, in the form of either diaphragms or external restraint.

Compressive membrane action can also be developed locally in a deck slab under discrete loading, using the surrounding unloaded area of deck to provide the restraint.

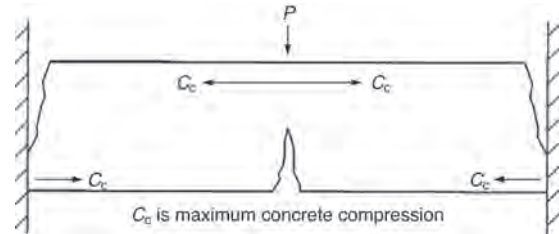


Figure 25.26 Maximum arching capacity of slab

Park and Gamble's method

Various papers are available that provide formulae for assessing CMA. Park and Gamble (1989) provide a formula backed by test results for slabs spanning in one direction, and includes an elastic end restraint, which is appropriate for bridge or culvert deck slabs spanning between abutments, where the stiffness of the abutment wall, while contributing, cannot be considered to be fully rigid.

Recent assessments of culverts using Park and Gamble's method have established that stiffer abutments can provide useful restraint for developing CMA, but that more flexible walls do not provide sufficient benefit. Where CMA is developed, an enhancement in the yield-line strength of the culvert roof has been obtained, enabling some culverts to pass assessment that would otherwise have failed.

Where culvert decks are subjected to a single heavily loaded lane, say by an abnormal vehicle, the CMA could spread the load transversely to adjacent, more lightly loaded, lanes of traffic. Park and Gamble's method has been developed to consider CMA acting in both directions.

Restraint provided by backfill

Clearly, the restraint offered by the abutment wall backfill is beneficial but, with the elasticity and variable properties of soils and gravels, it is not prudent to rely on them to develop the restraint required to mobilise compression within the depth of the concrete slab.

The point has, however, been made that where the deck slab is cast before the abutments are backfilled, the active or at-rest earth pressure acts to precompress the slab, and that this precompression can be utilised to develop membrane action without needing to rely on the stiffness of the external restraint.

However, shrinkage of the deck slab would tend to reduce that precompression. In the studies published, it seems that no one has yet relied on such precompression to develop CMA in practice.

Work at Queen's University Belfast

Considerable work on CMA has been carried out at Queen's University Belfast, particularly in bridge decks; a reduced amount of slab reinforcement has been recommended in beam-and-slab decks, owing to CMA acting in the slab between the beams. Tests have also been carried out on slabs spanning in both directions, with rigid peripheral restraint provided by a frame. More flexible restraints are dealt with by analysing an equivalent longer slab as a three-hinged arch with rigid restraint (Rankin and Long, 1997) (see Figure 25.27).

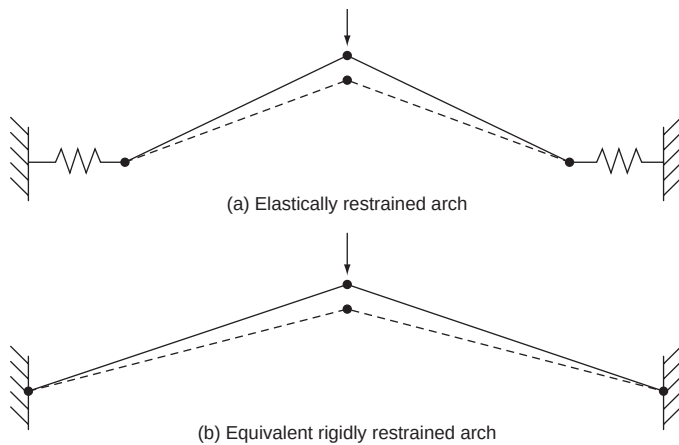


Figure 25.27 Compressive membrane action with elastic restraint

Bridge decks

Kirkpatrick *et al.* (1984a) of Queen's University Belfast investigated whether the beam spacing of M-beam decks could be increased, by looking at the strength of the standard 160 mm

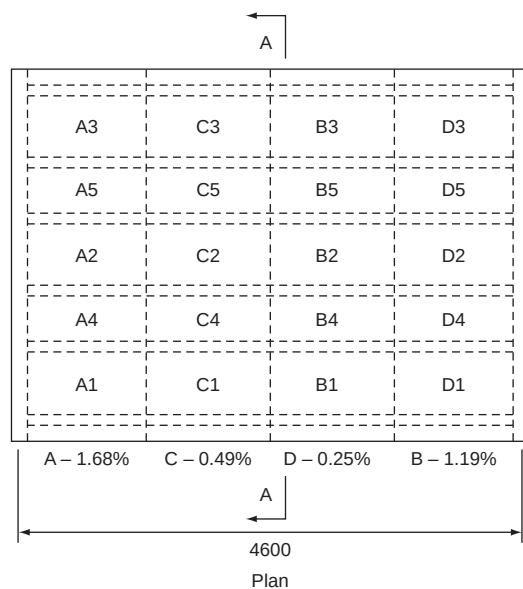


Figure 25.28a Details of bridge deck model

deep RC slab under the action of abnormal loads. They built a one-third scale model with different reinforcement percentages and different beam spacings (Figure 25.28), and tested 20 panels, all of which failed in punching shear.

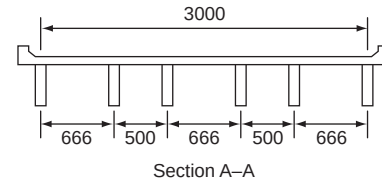


Figure 25.28b Details of bridge deck model

In analysing the results, it was found that significant additional strength was derived from compressive membrane action, leaving the failure load almost independent of the percentage of transverse reinforcement. Assuming that slabs are fully restrained transversely, Kirkpatrick *et al.* (1984a) proposed a means of calculating the ultimate capacity based on a modified punching shear equation developed from an analysis of McDowell *et al.* (1956) of the arching of rigidly restrained strips (see Figures 25.29 and 25.30). The increase as a result of CMA is covered by an equivalent percentage reinforcement factor, while the reinforcement itself is neglected. The method of calculation closely matches the model test results. The formula is given as follows.

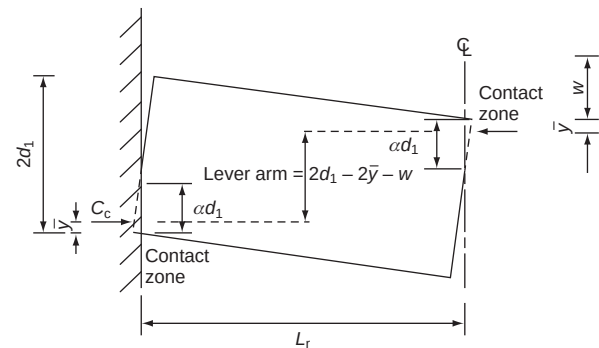


Figure 25.29 Idealised mode of arching action

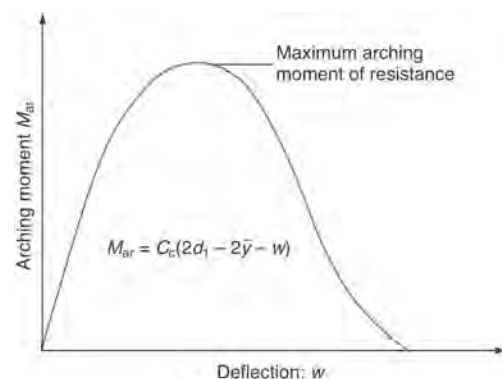


Figure 25.30 Arching moment of resistance

The punching strength of a rigidly restrained slab can be predicted using

$$P_p = 1.36(\phi + d) d \sqrt{f_{cu}} (100\rho_e)^{0.25} \quad (25.11)$$

where P_p is the ultimate load, ϕ is the diameter of the loaded area, ρ_e is the effective reinforcement ratio, f_{cu} is the characteristic cube strength of the concrete, d is the average effective depth to tensile reinforcement and

$$\rho_e = \frac{k f_{cu} h^2}{1.25 \times 320 \times 0.75 d^2} \quad (25.12)$$

where h is the overall depth of the section, k is the arching moment coefficient (taken from Figure 25.31), f_c is the cylinder compressive strength of concrete (taken as 80% of the cube compressive strength) and L is the slab span.

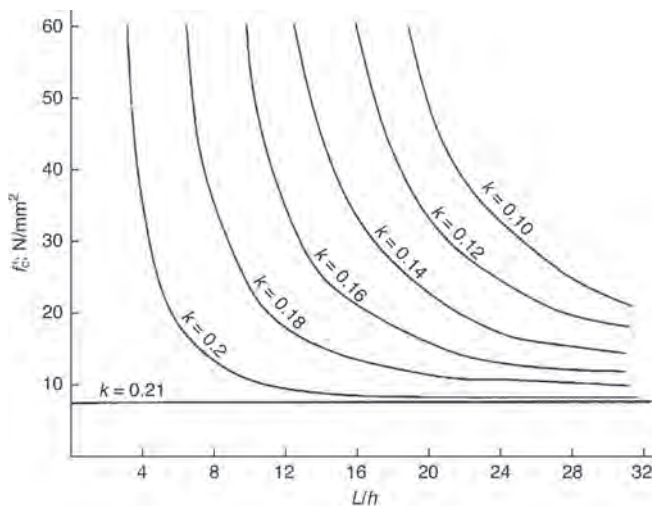


Figure 25.31 Curves of arching moment coefficient (k)

The punching strength so calculated is based on the following assumptions.

- The existing flexural reinforcement has no influence on the capacity of the slab.
- The arching enhancement can be represented by an equivalent percentage of flexural reinforcement ρ_e .
- The critical section is at $d/2$ from the perimeter of the loaded area.

Serviceability

A certain amount of deflection is required to mobilise CMA; therefore, care must be exercised to ensure that the structure is acceptable at the serviceability limit state, as discussed in the section on yield lines.

Recommendations

Calculation of compressive membrane action (CMA) can show significant increases in slab capacity over yield-line techniques and has much promise. Its benefit for bridge decks has been developed at Queen's University Belfast. Non-linear computer analysis broadens the scope for the use of this method; as computing becomes more powerful and more economical, the incorporation of CMA into analysis is likely to become cost-effective. However, the development of more versatile non-linear programs that can encompass CMA is likely to be a necessary forerunner to the more widespread use of CMA. The method may be limited by a failure to comply with serviceability criteria – cracking and large deflections, which are best judged by inspection rather than calculation. However, serviceability failure is essentially a durability issue and can be addressed by increased protection rather than load restriction or replacement or repair (see Das, 1993; Eyre, 1997; Jackson, 1993, 1996; Jackson and Cope, 1990; Kirkpatrick *et al.*, 1982, 1984b; Long *et al.*, 1994; Low and Ricketts, 1993; Jackson, 1995; Park and Gamble, 1989; Rankin *et al.*, 1981; Taylor *et al.*, 1998).

Non-linear concrete modelling

Introduction

Non-linear concrete modelling is usually used to investigate the behaviour or resistance of a particular detail. The non-linear analysis may be required when substandard reinforced detailing has been identified. Non-linear analysis should generally only be adopted when simpler (more conventional) analysis methodologies, such as strut-tie modelling, have been considered without being able to provide an adequate solution.

Considerations

Non-linear concrete modelling represents advanced modelling and analysis processes, which require an experienced engineer. It is recommended to simplify a three-dimensional problem to a two-dimensional plain stress or strain problem where possible. Where simplification is not possible, an extensive three-dimensional model can be created. However, the engineer should be aware of the time requirements for the model development and the analysis itself. A key criterion before setting out to develop a non-linear concrete model is identification of the key outputs required. The complexity of the model should then be tailored to achieve the required model.

Where non-linear concrete modelling is to be used, it is necessary to model both concrete body and reinforcing bars. The bars need to be fully connected with embedding concrete, while retaining nodal independency where orthogonal bars

cross; full force transfer between the bars cannot be permitted within the model. The concrete material model must reflect accurate compressive and tensile behaviour. Non-linear finite-element software packages allow the use of various concrete material models, depending on the software developer. An engineer creating non-linear concrete models should first understand the theoretical background of the concrete model implemented in the chosen software package. A proper understanding of the chosen concrete model is essential in processing the obtained results from the analysis. Apart from stress contours, many software packages present a crack pattern visualisation, developed from acting loads and calculations of crack widths. However, crack widths should be verified against the applicable national standards, as they might not be based on the same crack theory as that implemented by the software chosen for the analysis.

An engineer carrying out the analysis should decide whether bond or slip attributes need to be applied to the bars where applicable. The most common method for applying the bond or slip attribute utilises springs connecting the slipping bar to the embedding concrete. Stiffness of the bond or slip springs is a non-linear property. The property needs to be derived either experimentally or may follow one of the standardised bond or stress models, according to specific instructions.

It is apparent that concrete detail analysis consists mainly of material non-linearities, such as concrete material, steel material or bond or slip models, and may also include non-linear boundary conditions. Introduction of a large number of non-linear attributes makes the model sensitive to many parameters controlling robustness of the analysis convergence. The non-linear parameters must be controlled by experienced users during the analysis execution. Each additional non-linearity introduced to the existing model should be treated with caution.

Conclusions

Non-linear analysis can provide significant benefits in the overall bearing capacity of the structure. However, care needs to be taken in the initial modelling stages to ensure that the model is sufficiently complex to provide the level of detail needed, while still ensuring that results can be easily interpreted.

Surfacing

Introduction

The use of asphalt surfacing to provide additional stiffness and strength to bridge decks has so far only been applied to extend the fatigue life of orthotropic steel decks. A paper on testing mastic asphalt to find a suitable mix for that purpose cites the use of an indirect tensile strength test, an indirect tensile creep test and an indirect tensile fatigue test (Mohammed and Paul, 1996). Following a review of 300 titles, under the key words *surfacing*, *asphalt* and *bridges*, no other papers have been found over the last 20 years that refer to the assessment of the strength of surfacing in contributing to the strength of the bridge deck.

Review of properties of surfacing materials

Referring to the issues described in the previous sections, the reasons for this appear to be as follows. First, asphalt surfacing is widely known to be flexible. Its success is based on its ability to flow, sealing up cracks that form as a result of temperature and other weather effects and from the action of traffic wheels in loading, compressing, braking, accelerating, turning and skidding on the surface. As such, it is clearly not a rigid material – it moves under load. Therefore, none of the literature examined derives a value of E for asphaltic materials. Presumably, creep under load is such a significant factor that the derivation of a Young modulus would be academic.

Second, temperature effects are highly significant, asphalt being much softer in high summer temperatures and much more rigid in cold winter temperatures. It is therefore a very variable material.

Third, asphalt materials vary widely. While the cube strength of concrete will largely determine its contribution to the strength of a bridge deck, and its value of E will not vary significantly for this purpose even as the strength varies, this cannot be said for asphalt. The tests for asphalt surfacing to determine its suitability are generally compaction-related rather than specifically strength-related, so if asphalt were to be used to contribute directly to the strength of bridge decks, new tests would most probably have to be devised to ensure that adequate strength and stiffness were being provided.

The fourth reason is the bond between the asphalt surfacing and the bridge deck. There are many cases of surfacing peeling away from the deck; therefore, there would be a question of reliability if the surfacing were to contribute to the integrity of the bridge deck. The content and laying conditions (temperature and humidity) of both the waterproofing membrane and the surfacing base course will affect both the bond between the deck and the membrane and that between the membrane and the base course.

Conclusions

It is likely that if a suitable method were available for determining reliable properties of asphalt surfacing for use in enhancing bridge deck strength, it would have been developed many years ago. It is therefore considered that there is little merit in pursuing this source of hidden strength, although it might be worth making some hand calculations to determine whether surfacing could benefit the serviceability limit state. (Note that when carrying out a dynamic analysis of Friarton steel box composite bridge, in Scotland, a higher neutral axis was detected, and this was attributed to the surfacing.)

Recommendations

Asphalt surfacing on bridge decks, being by its nature a flowable material, does not readily lend itself to a reliable increase in concrete bridge deck strength. Suitable tests would have to

be developed to determine and control the properties of asphalt surfacing appropriate for strength enhancement and with the necessary reliability under temperature, creep, ageing and peeling effects. The future for using surfacing to enhance concrete bridge deck strength is therefore at best uncertain (Mohammed and Paul, 1996; Road Research Laboratory, 1962).

Shear in prestressed concrete flanged beams

Two sources of low live load rating, both concerned with shear, are frequently encountered in the assessment of prestressed concrete flanged beams. Flanged beams, such as box girders, often appear to have problems with shear capacity, since the webs are usually necessarily thin to minimise weight. Previous revisions of the assessment code BD 44 had a number of conservatisms, which were removed in BD 44/15 (Highways Agency, 2015) and are now contained within CS 455 (HE 2021b). These include:

- reducing the reduction for web thickness for grouted tendons
- using the shear distribution in the beam calculated by elastic theory
- increasing the allowable concrete strength
- allowing use of the Eurocode variable truss shear approach.

These changes considerably improve the assessment results for shear in prestressed connector beams.

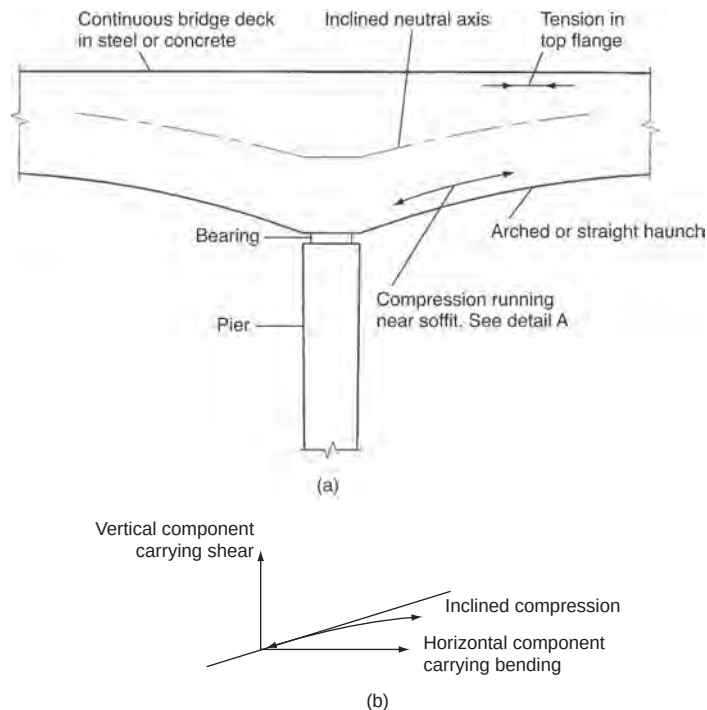


Figure 25.32 Inclined neutral axis: (a) elevation on bridge deck; (b) detail A

Inclined neutral axis

Many beams and slabs forming bridge decks are haunched or arched, particularly if the spans are of continuous construction. This results in an inclined neutral axis towards the piers.

In hogging bending towards the piers, the tensile force in the top of the section generally runs horizontally, while the compressive force running near the soffit is inclined. The vertical component of this compressive force can be utilised to carry part of the shear force and hence increase the shear capacity (see Figure 25.32). Alternatively, the deck analysis may be carried out with an additional vertical dimension to reflect the profile of the neutral axis of the deck. The inertia of the deck will vary along the span.

In prestressed concrete, the inclination of the tendons should also be taken into account.

Bearing clamping

In prestressed concrete, the inclination of the tendons should also be taken into account.

Transverse pressure may be exerted on the soffit of beams or slabs by supporting bearings, piers or abutments. On the top surface of cross-beams, transverse pressure may be exerted by the loading through deck beam bearings (see Figure 25.33).

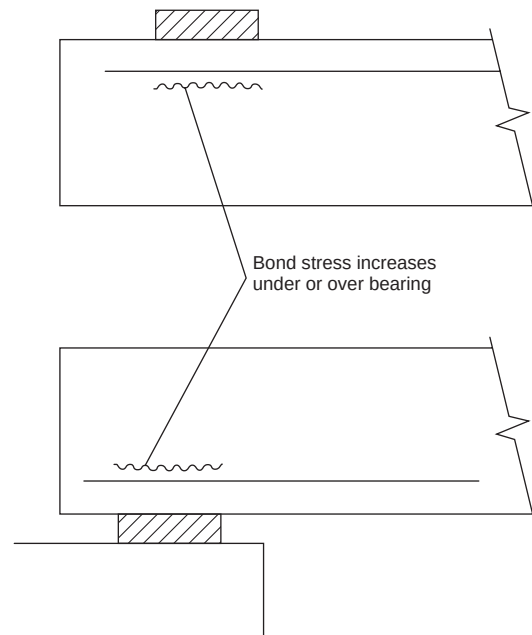


Figure 25.33 Clamping effect of bearings

The increase in bond strength as a result of transverse pressure depends on several factors, including the depth of cover, concrete strength, bar diameter, transverse pressure and whether the bars are plain round or deformed. The effect is greater as the cover reduces.

The references cited in this section provide research results giving the increase in bond strength caused by transverse

pressure. Testing at the University of Westminster (Regan, 1999) has been conducted to determine the effect on bonding of transverse pressure where the concrete under the bearing is delaminated. In this case, although the enhancement from clamping is more significant for delaminated than for non-delaminated cases, the unclamped delaminated bond value is very small. This results in clamped delaminated bond values being smaller than clamped non-delaminated bond values.

Clamping by rigid bearings can be more effective in increasing the ultimate bond stress than clamping by flexible bearings, such as elastomeric bearings.

Care must be taken, when allowing the enhancement of bond strength in beams, that failure of side cover does not weaken the assumed bond strength. Indeed, the vertical pressure from the clamping of the bearing can reduce rather than increase the bond strength for edge bars, which are subject to failure of the side cover.

For clamped delaminated cover over deformed bars in tension (type 2 deformation), the bond strength derived from clamped pull-out tests at the University of Westminster (Regan, 1999) using polythene to simulate delamination can be analysed as follows, provided the side cover is not susceptible to failure:

$$\beta = 0.25 \frac{\rho}{\sqrt{f_{cu}}} \leq 0.7 \quad (25.13)$$

where ρ is the transverse pressure (N/mm^2), f_{cu} is the cube strength (N/mm^2) and

$$\frac{\rho}{\sqrt{f_{cu}}} > 0.05 \quad (25.14)$$

where the allowable bond stress over the clamped length is $\beta \sqrt{f_{cu}} \gamma_{mb}$ (Regan, 1999).

The following restriction is required in a delaminated anchorage containing a mixture of bars or bar lengths that are in or not in stirrup corners and are clamped or unclamped. In a delaminated anchorage length, where some bars are not in stirrup corners, either the delaminated bond stress for bars not in stirrup corners should be used for all bars, including those clamped, or the bond stress should be taken as zero for all bars or part length of bars not in stirrup corners and not clamped.

For type 2 deformed bars in tension with delamination to half barrel:

$$\beta = 0.1 + 15 \frac{A_{ss}}{s\phi} \leq 0.7 \quad (25.15)$$

where s is the stirrup spacing, ϕ is the diameter of the main bar and A_{ss} is the area of a stirrup leg immediately adjacent to the main bar (the combined area of both legs in the case of a hairpin link around a single main bar or, per main bar, half the area of a single stirrup leg adjacent to a pair of lapped bars) (Batayneh, 1993; Gambarove and Rosati, 1997; Navaratnarajah and Speare, 1986; Taylor, 1974; Untrauer and Henry, 1965).

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Chapter 26

Repair, strengthening and replacement

L Canning Jacobs

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Maintenance of bridge structures has undergone rapid development over recent decades. New deterioration mechanisms have become evident as bridge stocks age, requiring new testing and rehabilitation techniques. Bridge assets are of critical value to the economy and society, and demand maintenance with the minimum disruption to the flow of traffic. Repair and strengthening may also provide a lower carbon footprint option than complete replacement.

This chapter considers the techniques required for particular common materials – concrete, metal and masonry – but also newer materials, such as fibre-reinforced polymer composites.

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Introduction

The resources devoted to repair, strengthening and replacement of bridges are now very significant, and in the UK exceed those required for new routes. In this climate, it is easy to forget that this flourishing sector of industry is relatively new. As recently as the 1960s, maintenance was largely confined to painting of steelwork and pointing of masonry. At that time, reinforced concrete was generally considered to be a maintenance-free material. The growth in bridge repair, strengthening and replacement is due to the following factors.

- Large numbers of bridges built to expand the infrastructure are now reaching an age at which progressive deterioration becomes evident.
- Salt was introduced for de-icing during the 1960s, with dramatic effects on the durability of concrete reinforcement.
- There has been an increase in the weight of vehicles permitted unrestricted access on the highways, and an increase in the number of movements of abnormal loads requiring specific approval.

The need for improved and consistent asset management of bridges is clear, and the foundation of this is good quality information on bridge condition and defects. Recognition of this by the industry has improved over time, and the availability of trained and competent bridge inspectors is commonly ensured by the use of qualification and certification schemes in many countries (e.g. UK Roads Liaison Group, 2016).

Increasing activity has brought about the need for greater knowledge of deterioration mechanisms, and a wider choice of materials and methods designed to satisfy particular needs. Although general deterioration mechanisms for common bridge construction materials are becoming better understood and widely documented (Concrete Society, 2000; SCI, 1997;

Tilly *et al.*, 2008), the determination and prediction of material deterioration rates of specific bridges and bridge elements, and hence planning of future interventions, is difficult, owing to changing microclimates (past and future) and the lack of consistent detailed inspection records over many decades.

However, selecting the right intervention extends beyond consideration of purely technical and economic matters. The need to consider sustainability (e.g. traffic disruption, increased emissions owing to diversions) also has a major influence on contract planning and material selection.

Cost has always been the most important factor in determining the most appropriate remedial measures, but there is now a welcome move towards the consideration of life-cycle costing instead of considering only the lowest initial cost. Management systems incorporating these aspects are covered in Chapter 23. Initial work has been undertaken on developing such tools specific to certain common subsets of bridges (Harwood and Smith, 2020). Attention to low-cost works covered in Chapter 22, such as waterproofing, drainage and impregnation, is likely to be particularly cost-effective. However, there will inevitably remain a large number of structures requiring more extensive works of the kind discussed in this chapter.

Repair, strengthening and replacement covers such a wide area of specialist works that it is impossible to cover the subject exhaustively in this chapter. Emphasis will therefore be given to factors influencing selection of the most appropriate maintenance action. Repair, strengthening and replacement should not be considered as unconnected topics. Instead, they should be seen as alternative strategies to achieve a common objective – that of maintaining an effective structure and effective transportation network. The choice between them is based on an evaluation of current defects, predicted deterioration and consequences, and the cost of remedial measures at each stage. The options could be simplified as: repair now, repair later,

strengthen later or ultimately replace. Delay in intervention will generally result in increased cost, but it would be incorrect to assume that early action is therefore always the most appropriate. Management decisions are inevitably also influenced by funding and affordability, as well as operational factors.

This chapter will therefore concentrate on an outline of potential solutions, and the implications of their selection. While this should assist the initial selection of effective interventions, the technical and financial circumstances applicable to particular structures are essential factors in final evaluation and comparison of alternatives. Whole-life costing tools (e.g. the Structures Toolkit (Hesketh *et al.*, 2015)) can evaluate the consequences of alternative strategies, and facilitate an objective comparison.

Repair and strengthening of concrete structures

Repair of concrete structures

The effective repair of concrete structures requires an understanding of the cause of deterioration and an appreciation of the influence of the chosen repair technique on future dura-

bility. This should be based on carefully planned inspection and testing of the existing structure (Concrete Bridge Development Group, 2002; Pearson and Patel, 2002).

The most common reason for concrete repair is corrosion of reinforcement, in which case the parameters of particular interest are

- reinforcement cover
- carbonation depth
- chloride contamination profile
- concrete mix details
- age of concrete
- cracking or voiding in concrete
- environmental factors influencing contamination and condition.

The interpretation of results from an inspection and testing programme should lead to an understanding of both the extent of the existing defect and the cause of deterioration and, ideally, an estimate of deterioration rate. By consideration of these factors, the most appropriate repair interventions may be selected, as shown in Tables 26.1 and 26.2 (see also BRE, 2000a, 2000b; Concrete Society, 2009).

Extent of defect	Potential repair intervention					
	Mortar repair	Placed concrete	Sprayed concrete	Cathodic protection	Desalination	Impregnation
Spalling or contamination replacement over small areas	*					*
Spalling or contamination replacement over large areas <25 mm deep	*					*
Concrete replacement over large areas >25 mm deep		*	*	*	*	*

Table 26.1 Concrete repair interventions arising from chloride-induced corrosion

Extent of defect	Potential repair intervention					
	Mortar repair	Placed concrete	Sprayed concrete	Cathodic protection	Desalination	Impregnation
Spalling over small areas	*					*
Spalling or carbonation over large areas <25 mm deep	*					*
Carbonation over large areas >25 mm deep		*	*	*	*	*

Table 26.2 Concrete repair interventions resulting from carbonation-induced corrosion

Extent of repair	Potential repair intervention		
	Mortar repair	Placed concrete	Sprayed concrete
Concrete replacement in small areas	*		
Concrete replacement over large areas <25 mm deep	*		
Concrete replacement over large areas >25 mm deep		*	*

Table 26.3 Concrete repair – application of repair materials

Concrete repair may also be required for a number of reasons other than reinforcement corrosion, such as that resulting from fire or frost damage. In such cases, the same general advice applies, the extent of repair giving the initial indication of the most economic method of application, as shown in Table 26.3.

Fire damage to Secklow Gate Viaduct in the UK in 2010

necessitated concrete repairs to the soffit of the cellular concrete deck, crossheads and tops of supporting columns (together with bearing replacement), as shown in Figure 26.1. Inspection and testing of the fire-damaged areas of concrete using industry guidance (Concrete Society, 2008) confirmed the extent of damaged concrete to be replaced. Owing to the severity of the



Figure 26.1 (a) Fire damage to concrete viaduct (left) and (b) concrete repairs with temporary jacking for bearing replacement (photographs courtesy of Jacobs)

Material	Properties	Main applications and comments
Concrete	Flow characteristics and strength designed to meet need (such as materials specified in BD 27/86 (Highways Agency, 1986)).	Minimum practicable depth to flow >25 mm, and repair keyed beneath the reinforcement. Structural effects of major replacement require consideration. Access considerations control detailing, perhaps requiring increased local thickness.
Sprayed concrete	Sprayed concrete with aggregates above 10 mm is generally called <i>shotcrete</i> , and with aggregates below 10 mm generally called <i>gunite</i> . 20–30 N/mm ² typical strength for wet process, and 40–50 N/mm ² for dry process. Good density and bond, with low permeability with dry process. HA loads require characteristic strength.	Minimum practicable depth 25 mm. Good access required. Continuous process requiring good workmanship, particularly with the dry process. Thick sections built up in layers, with the wet process when previous layer has stiffened, and in dry process when previous layer has gained strength. Smooth finish cannot be achieved, and the final coat is normally left unscreeded. Excess spray may be carefully sliced off, and second coat with fine sand aggregate produces more pleasant finish.
Hand-applied cementitious mortars and polymer-modified cementitious mortars	Polymer-modified mortars are specified in BD 27/86 for highway works, and must have a w/c ratio <0.4 and cement content >400 kg/m ³ .	Well-placed cement sand mortars have proved durable over 25 mm thickness. Polymer-modified mortars have improved properties and are appropriate for thicknesses down to 12 mm. Of very high cost and rarely used. Different expansion characteristics to cementitious materials, and rely on chemical bond.
Polyester resin and epoxy resin mortars	Develop high mechanical strength within 24–48 h. Provide impermeable coating to encapsulate reinforcement.	Rely entirely on impermeability for protection, thus requiring good materials and workmanship. With lightweight fillers produce low-density thixotropic mortar suitable for overhead applications.
Epoxy resin for crack injection	High-modulus, high-strength resin. Available in low-viscosity thixotropic systems.	Use for structural cracks where no further movement expected. Thixotropic system required for cracks that cannot be sealed on all sides.
Styrene butadiene rubber (SBR) or acrylic latex emulsion for crack sealing	Low-viscosity material disperses water into surrounding concrete to leave rubbery mass.	Appropriate for narrow cracks, or repeated application to wider cracks, to form water-resistant seal under low head where future movement limited.

Table 26.4 Properties of materials influencing selection for the repair of concrete structures

fire, areas of reinforcement also had to be replaced with new lapped reinforcement bars.

Repair of concrete may take the form of crack sealing or structural crack repair. Such repair should only be undertaken if the cause of the cracking is understood, the reasons for repair are well-defined and subsequent movements will not render the repair ineffective (Concrete Society, 1984). The expected design life of any concrete repairs should also be considered and compared with that of the overall structure.

Chemical degradation, such as alkali–silica reaction (BRE, 2004; Concrete Society, 1999), can also be a cause of durability issues in concrete structures. In such cases, where the cause of degradation might be pervasive throughout a concrete structure, more extensive mitigation or repair methods are often required and such repairs might have limited design life. Replacement (partial or otherwise) or strengthening of concrete exhibiting severe defects, although more expensive and disruptive, might provide a more effective long-term option in such cases.

Materials for the repair of concrete structures

The geometry of the repair will give initial guidance on the method of applying the repair material; there remains a wide choice of materials. It is also necessary to meet any relevant client specifications.

Table 26.4 summarises the principal characteristics that might influence selection. Owing to the poor track record of many past concrete repairs and inconsistency in selection and application of materials, the need for improved generic classification and specification guidance is clear and improved consistent standards are now available (BSI, 2005).

This guidance covers such topics as material types (cementitious, fibre and polymer materials) and their required properties, existing concrete substrate requirements and assurance testing and application methods and limitations. It is also noted that corrosion-resistant reinforcement (such as stainless steel, glass fibre, carbon fibre and basalt fibre bars)

Material	Basis of method	Advantages and disadvantages
Cathodic protection	Chloride-induced corrosion is loss of metal at anodic areas from which corrosion currents flow. A DC protection current is impressed between new anodes and the reinforcement, thus opposing and stopping the natural corrosion current.	The principle is well tried and proven to be effective. Prevents corrosion caused by chlorides and carbonation. Systems are expensive to install, and require ongoing maintenance and ultimate replacement. Modern systems and materials extend life spans to over 30 years.
Desalination	The surface of the structure is covered with external anodes and an electrolyte reservoir. The negatively charged chloride ions migrate to the external electrolyte away from the negatively charged reinforcement.	Desalination provides a durable long-term solution only if future contamination is prevented. Concrete to be desalinated must lie between reinforcement cathodes and the anode. Desalination of concrete is completed in 8–13 weeks. Desalination is more expensive than cathodic protection.
Realkalisation	The physical arrangement is similar to desalination, described previously. An alkali electrolyte is used, which causes alkaline ions to migrate to the reinforcement. This re-establishes the passive layer around the reinforcement and increases the pH.	Realkalisation provides a durable long-term solution only if further carbonation is prevented. The process is normally completed within 7 days.
Anti-carbonation surface treatments	Carbonation results from ingress of carbon dioxide into concrete, reacting with calcium hydroxide and reducing alkalinity. Surface treatments designed to exclude the carbon dioxide molecule while allowing the passage of water vapour can inhibit carbon dioxide ingress.	Low-cost method of extending the life of structures, which would otherwise suffer from corrosion owing to carbonation. Cannot prevent corrosion if concrete already carbonated at reinforcement depth.
Migrating corrosion inhibitors	Coating applied to surface of concrete, the inhibitor penetrating by liquid and gaseous diffusion to form a thin layer on the reinforcement surface.	New technology; long-term effectiveness unproven. More expensive than silane but likely to be cheaper than concrete removal or realkalisation.
Impregnation surface treatments	Materials such as silane, when impregnated into concrete, can prevent ingress of water in the liquid phase, but permit water vapour to exit the structure. Thus chlorides no longer enter carried by the liquid phase, and concrete slowly dries, inhibiting corrosion.	Very low-cost solution. Only effective in inhibiting corrosion if water can also be prevented from entering the concrete from alternative faces. Limited number of instrumented trials demonstrating effectiveness.

Table 26.5 Comparison of alternative methods of inhibiting corrosion

is available, and its use in conjunction with concrete repairs should be considered where appropriate, for example, in particularly severe environments where marine or de-icing salt exposure is high.

Methods of inhibiting corrosion

Concrete replacement is expensive because it is labour-intensive, and may not be economic if the chloride contamination extends to a great depth. Provision of temporary support may also be impracticable or uneconomic. Alternative solutions are thus required.

Table 26.5 summarises the methods of preventing reinforcement corrosion that do not require concrete removal.

Strengthening of concrete structures

Strengthening of concrete structures is often a difficult and costly operation because changing the proportion of internal reinforcement and internal prestressing is impractical once a concrete member has been cast, without its effective demolition. Strengthening should only be considered once rigorous structural assessment and testing, and consideration of a full range of options, has confirmed it to be the most appropriate solution. In these circumstances, the choice may be limited to

- increasing the depth of beams or slabs, including reinforcement in the extra concrete, and providing shear connection, perhaps by drilling and bonding
- increasing the width of beams or, more usually, providing intermediate beams
- providing external prestress by external steel cables and anchorages or Macalloy bars
- drilling and grouting additional reinforcement, perhaps in the form of stressed bars
- providing buttresses or extra thickness to walls
- using composite materials bonded to the surface or placed in slots cut in the surface (Concrete Society, 2012).

Developments of new techniques and materials in recent years have opened up new possibilities. In particular, modern epoxy resin adhesives enable additional reinforcement to be

added externally and minimise the need for concrete drilling or excessive concrete removal (with consequential cost, environmental and safety benefits). However, successful structural adhesive bonding depends fundamentally on the strength and condition of the concrete substrate (and the bond of cover concrete to existing steel reinforcement). The use of structural adhesive bonding and fibre-reinforced polymer (FRP) materials can also be limited by the need to maintain minimum ductility requirements and robustness against accidental (e.g. fire, vandalism, impact) events. The new methods may be categorised as follows:

- steel plate bonding
- unstressed composite plate bonding
- stressed composite plate bonding
- composite column wrapping
- deep embedment composite bars.

This chapter will concentrate on these more recent developments, because they will have widespread application. Other methods will certainly have application in certain circumstances, perhaps in combination with external reinforcement, but that will depend on the particular details of the structure. Further information on the variety of strengthening methods for concrete bridges are given by the Concrete Bridge Development Group (2008).

Steel plate bonding

The technique of steel plate bonding for bridge strengthening was given official encouragement in 1994 with the publication of BA 30/94 (Highways Agency, 1994). However, this was preceded by over 15 years during which major bridges in the UK were plated and the results monitored. The M5 Quinton and M25 Swanley Interchange bridges, in the UK, were plated in 1975 and 1977, respectively, followed by bridges on the M1 and A10 from 1982 to 1986. At that time, there were many more applications elsewhere, particularly in Japan.

Steel plate bonding provides the section with additional tensile capacity at maximum eccentricity, where it is most effective. Structures that would otherwise have required demolition have been preserved. However, there are also significant limitations and disadvantages, as may be seen from Table 25.6.

Advantages or benefits	Disadvantages or limitations
Strengthening with minimal increase in dimensions	Sound substrate required, free of risk of expansive cracking owing to reinforcement corrosion.
Tensile capacity at maximum eccentricity where most effective	Steel plate liable to corrosion. Exposed surfaces require maintenance, and there is a risk of adhesion failure if adhesive does not protect against rusting at interface. Section requires excess compressive capacity to avoid brittle failure.
External member relatively easy to inspect and check of effectiveness	Extensive bolting required for temporary support and to resist end peeling. Risk of buckling in compression, resisted if stress is low by bolts. BA 30/94 (Highways Agency, 1994) restricts method to structures already able to support dead load and unfactored nominal live load.

Table 26.6 Factors influencing the selection of steel plate bonding for strengthening

The use of steel plate bonding is now less common, owing to the benefits of FRP plate bonding, described later. However, there are still a small number of situations where steel plate bonding may be considered (e.g. where the environment is benign, where thin steel plates can be used so weight is less of an issue, and access and application is simple).

Practical aspects of steel plate bonding

The effectiveness of plate bonding depends entirely on the composite action between the plates and concrete substrate, and hence on the adhesive bonding. High-quality workmanship and supervision are essential. Both the steel and concrete surfaces must be grit-blasted, dry and free of dust. The steel will quickly deteriorate after grit-blasting, and bonding should take place within 4 h or a shorter period to avoid rust blooming. Alternatively, the plates may be blasted, cleaned and primed away from the site. After transport within clean protection, they may be bonded after degreasing and drying.

Unstressed fibre-reinforced polymer (FRP) composite plate bonding

The use of plate bonding for repair and strengthening has increased as the benefits of FRP have become apparent. All of the benefits outlined for steel plate bonding are applicable, and the disadvantages or limitations reduced. Fibre-reinforced polymer plates are usually manufactured by the pultrusion process, and comprise carbon, aramid, basalt or glass fibres within a polymer matrix, such as vinyl ester or epoxy. In the UK, the largest research project into FRP bonding, which encouraged further field applications, was the ROBUST project (Holloway and Leeming, 1999).

Although the material cost of composite plates is greater than that of steel plates, most applications have confirmed that FRP plate bonding is preferable when all costs are taken into account.

The use of in situ laminated FRP fabric as an alternative to bonding of preformed FRP plates is also common and, although generally slightly more labour-intensive, can be particularly useful where there is a need to minimise the bond stresses between the FRP and concrete substrate.

An example of the advantages and adaptability of FRP strengthening technology is shown by its use on a small historic concrete arch bridge, Minsterley Bridge in the UK (Lowe *et al.*, 2014), and possibly the most significant application worldwide, West Gate Bridge in Australia (Kalra and Neubauer, 2003). Further developments in FRP strengthening technology can be expected, and research is ongoing in the use of cementitious and inorganic bonding materials and quick mechanical fixing methods, as well as new fibre types, such as basalt fibre (Concrete Society, 2012). Recent research and industrial development has also enabled more cost-effective manufacturing methods for carbon fibre to be developed (Holmes, 2017).

The potential advantages of FRP plate bonding may be summarised as follows.

- **Strength:** FRP plates generally have high strength, and may be purpose-designed with varying proportions of components.
- **Weight:** FRP plates are 20% of the density of steel, and typically 10% of the weight for the same ultimate strength.
- **Transport:** FRP plates may be easily transported, owing to their low weight, and those of small thickness may be coiled for transport.
- **Versatile design:** the small thickness enables plates to cross over each other, one plate being kept within the adhesive thickness of the other.
- **Surface preparation:** steel plates must be quickly installed after preparation. By contrast, FRP plates can be manufactured with a protective ply that is peeled just before fixing to expose a prepared surface.
- **Mechanical fixing:** the need for fixing to counteract peel is reduced because of the reduced thickness compared with steel; in addition, there is reduced weight to support in the short term.
- **Durability:** FRP plates do not corrode, and therefore do not suffer the same risks as steel of corrosion at the glue line or exposed surfaces. Maintenance costs are reduced.
- The cost of FRP plates has reduced as the volume of applications has increased.
- There are reduced construction costs and periods, owing to the low weight and easy handling and fixing.

Stressed FRP composite plate bonding

Another relatively recent development has been the use of prestressed FRP plates. This technology further extends the range of strengthening problems for which plate bonding is the most efficient and economical solution. A proprietary system has been developed in the UK (Darby *et al.*, 1999, 2000) in which anchorages are fixed to the structure and preformed tendons of predetermined length are stressed until their end tabs engage with the anchorages. The first application of this method has



Figure 26.2 End anchorages and prestressed composites providing strengthening to Hythe Bridge in Oxford (photograph courtesy of Mouchel Parkman)

Advantages of stressed FRP plates

Resistance of materials below neutral axis is mobilised, such that all dead load effects may be neutralised before live loads are applied. The quantity of FRP plates required is greatly reduced compared with that of unstressed plates.

May be used for applications where anchorage bond lengths are limited.

Disadvantages of stressed FRP plates

Additional cost of anchorage manufacture and fixing above cost of unstressed plates.

Specification, supervision and workmanship must recognise the specialist nature of prestressing and the precautions required for reliable bonding.

Table 26.7 Comparison between stressed and unstressed FRP plate bonding

been to a cast iron bridge, Hythe Bridge in Oxford. This is illustrated in Figure 26.2. Stressed composite plate bonding is equally applicable to concrete structures. The capacity of Hythe Bridge was increased from 8 t to 40 t vehicles by the application of four plates, each stressed to 16 t. The end anchorages shown in Figure 26.2 were enclosed within a galvanised casing and fully grouted.

Stressing is particularly effective with FRP tendons because the system mobilises the resistance of materials below the neutral axis, while retaining a high safety margin in the tendons below their very high ultimate strength. The advantages and disadvantages of stressed FRP plates compared with unstressed plates are given in Table 26.7.

A number of commercial prestressed FRP plate bonding systems, characterised by proprietary anchorage systems, are now available and have been used to strengthen many concrete bridges (Millar *et al.*, 2004). However, owing to the more complicated application for the anchorage systems, the use of prestressed FRP plate bonding is much less common than unstressed FRP plate bonding.

Summary of FRP plate bonding

Plate bonding is a versatile solution to strengthening of bridge structures. The applications are summarised in Table 26.8.

FRP composite column strengthening

Systems have been developed for the external strengthening of columns using glass, carbon or aramid reinforcement within polymers (Jolly and Lillistone, 1998). Such systems are often

used to enhance seismic behaviour by extending ductility, and research in North America and Japan has been directed, in particular, towards meeting this need. However, ultimate strength and failure strain are also improved, and systems that fully enclose a column by wrapping will provide a protection against further ingress of water and chlorides. Fibre-reinforced polymers have also been used for strengthening of reinforced concrete columns against vehicle impact (Scarth and Keble, 2003).

Repair and strengthening of metal structures

Metal bridges in the UK

The first metal bridge structures were constructed in the late eighteenth century. Large numbers of cast iron and, later on, wrought iron bridges were built during the canal and railway expansion of the nineteenth century. Many of these early examples remain in use to this day, often carrying loads greatly in excess of what they were designed for. Modern metal structures are constructed almost exclusively in steel, but thousands of bridges built of cast iron, wrought iron and early steels are still in service today on highway and railway networks. In the UK, there are currently approximately 16 000 metallic bridges, either over or carrying the railway, many almost or over 100 years old. Wrought iron and early steel structures were usually of riveted construction. Welded construction became established in bridges in the period following the Second World War, while riveted construction,

Structural deficiency

Corrosion of reinforcement

Strengthening of structures in flexure

Uncertain durability of steel prestressing tendons

Inadequate stiffness or serviceability, owing to cracking under load

Strengthening of structures in shear

FRP plate bonding solution

Replacement of corroded reinforcement by external FRP, providing concrete remains sound

External FRP reinforcement added, providing there is sufficient section capacity in compression

External FRP may be added as a safety net, or stress to replace prestress which may have been lost

Stressed external FRP to remove tensile stresses

Web may be reinforced externally, but this aspect less researched than other applications

Table 26.8 Structural deficiencies for which FRP plate bonding offers a potential solution

although becoming less common, did not finally pass out of use until the 1960s.

Metal bridge structures are generally durable if properly maintained. The deterioration processes to which they are subjected are well understood and can be readily rectified if caught early enough. The primary causes of metal bridge deterioration include

- corrosion
- fatigue or other cracking
- deflections or distortion caused by loading or impact damage.

Of these, corrosion caused by breakdown of the protective system, often aggravated by chlorides from road salting, is by far the most common cause of deterioration.

Corrosion and fatigue damage are both often exacerbated by poor design detailing, which creates corrosion traps and stress concentrations in members. Good inspection procedures will detect most forms of metal deterioration early enough for action to be taken before serious damage – affecting the load-carrying capacity of the structure – has resulted, unless particularly problematic design details are present (e.g. hidden critical details). Poorly designed fatigue details that are difficult to inspect are an example of this (Collins *et al.*, 2017). One advantage of metal bridge structures over concrete bridge structures is that deterioration is readily detectable on the surface of the members and visible to the trained eye with a minimum of equipment. The exceptions to this are buried structural members where access is difficult (particularly in railway carrying bridges), closed-box members and other enclosed details, such as load-bearing pins.

Ultrasonic gauges can be used to measure the thickness of plates and members during inspection. This equipment may also be used by specialists to detect the presence of cracking in elements that might not be accessible by other means, such as load-bearing pins. Other methods of detecting cracking in metal structures include X-ray photography and magnetic particle testing. These techniques should only be undertaken by specialist testing companies. Cracking that is not visible to the naked eye can be detected using dye penetrant testing, a simple technique that can be used by a bridge inspector. Acoustic emission techniques have also been developed for detecting flaws in metallic elements (such as those used at the Hammer-smith Flyover, as described in Webb *et al.* (2014), to detect breaks in prestressing tendon strands), but there is little current guidance on this method.

If the strength and mechanical properties of the metal structure are not known, they can be determined by cutting a small sample or coupon from a low-stressed or unimportant area. The coupon can be laboratory tested for tensile strength and composition. If repairs or strengthening involving welding are contemplated, the composition of the metal and its weldability must be established. This can be determined by a laboratory from a coupon or from shavings obtained by drilling. However, care is required in interpreting the test results and ensuring

a statistically significant sample size to provide reliable estimates (see Tilly *et al.*, 2008).

Repair of metal structures

General

Repair of metal structures suffering from deterioration caused by one or more of the deterioration processes listed previously should only be attempted when the cause of the problem has been clearly established and understood. In most cases, the cause of deterioration will be immediately apparent; however, simply repairing the defect might result in its recurrence if the original underlying cause is not identified. It might be that an inappropriate inspection and maintenance cycle has resulted in complete breakdown of the paint system, which could have been avoided by earlier attention and overcoating. It is possible that localised corrosion has resulted from inadequate or faulty deck drainage, or the incorporation of a corrosion trap in the original design, trapping dirt and debris and leading to early onset of paint breakdown and severe corrosion. Cracking as a result of fatigue and other effects might be due to unexpected loading or environmental effects or – more commonly on older structures – fatigue details not meeting current fatigue detail standards.

In these cases, the design of remedial works and repairs should include the rectification, if practically and economically possible, of the original underlying cause of the damage to the structure to prevent its early recurrence. Other examples would include fatigue cracking caused by a poor connection detail, resulting in out-of-plane bending of a girder web plate. Simply repairing the damage without amending the detail would lead to the same type of cracking appearing again.

Economics of repair

An important consideration when designing repairs to a metal structure is the economics of repair against replacement – either of the affected member or the entire structure. Metal bridge repairs typically involve the use of small quantities of material and large amounts of labour. It is therefore usually a false economy to concentrate on minimising the material quantities in the design of repair works: of far greater importance is the ease of fitting on-site. Repairs are thus often very expensive in comparison with the construction of a replacement structure with inherently lower future maintenance costs and a longer useful life. A life-cycle cost comparison between the repair and replacement options will be necessary to determine the best option. However, where disruption to the public owing to reconstruction would be great, for example on busy highway bridges or railway structures, or a large number of buried services are present to the structure, with high diversion costs, repair methods are often shown to be a cost-effective option; in some cases, the only option. In addition, due cognisance must be taken of heritage structures, where reconstruction is not a viable option.

A further consideration is that details on metal bridges are very often repeated many times. If cracking or serious corrosion is found in one particular detail on a metal bridge, other locations where similar or identical details occur should be carefully investigated: similar problems might be detected in earlier stages at other positions. In this case, a repair or modification that can be applied at all repetitions of the affected detail might be required, even those not currently exhibiting the defect. The costs of mobilisation, providing access and possibly traffic management might make the wholesale modification of a suspect detail over an entire bridge during a single repair contract economically preferable to carrying out the work in smaller packages over several years.

Repair techniques

The most appropriate repair technique for an individual structure will depend on a large number of parameters, including

- the construction material
- the form of construction (cast, welded, bolted or riveted)
- the degree of redundancy, risk and consequence of failure
- the cause of the defect (corrosion or fatigue, for example)
- the severity of the damage
- the ease of access, and costs of closure, traffic management or possessions
- the existence of buried services
- the extent or number of repairs.

Repairs by steel or FRP plating

Sections that have been weakened by corrosion can be repaired by applying new metal plates to replace the section loss. Additionally, FRP plates and fabric (particularly carbon FRP) are also now used for reinstating section loss, and are adhesively bonded to the grit-blasted surface of a metallic substrate under controlled environmental conditions. Fibre-reinforced polymer plates are significantly stronger and lighter than steel plates and do not corrode; these advantages enable easier handling and access to difficult areas, although the design life is currently limited, based on the track record of structural adhesives. Galvanic corrosion between carbon FRP plates and metal must be prevented; this can be achieved by applying a layer of insulative glass FRP to the bonding surface of the carbon FRP plate. The new plates are bonded (in the case of FRP plates), bolted or welded over the corroded or under-strength areas, allowing sufficient overlap to ensure the effective transfer of stresses to and from the intact member. Designers should appreciate that new plating of this kind will not contribute to the dead load-carrying capacity of the structure, and will itself add additional dead load to the structure being strengthened (although this will be minimal if FRP plates are used).

Plating can also be used to repair members that have suffered displacement and distortion of web and flange plates,

perhaps by impact from over-height vehicles or vessels. New web or flange plating can be spliced over the distorted section. In severe cases, the bent and distorted plates will have to be cut away prior to splicing in new sections of plate. If new plates are spliced over distorted areas of old plate, care must be taken to fill and seal the areas between old and new plates to avoid the creation of a corrosion trap.

The practical difficulties of bolting new plates onto a corroded older structure should not be ignored: the surfaces of older sections are usually pitted, and achieving good contact between the plates, allowing satisfactory transfer of stress and avoiding the creation of corrosion traps, can be difficult. Both surfaces to be joined should be painted with a primer with a sufficiently high coefficient of friction to allow high-strength friction grip (HSFG) bolts to work efficiently. The pitted plate should be filled with high-strength metal-filled epoxy putty, which will effectively fill and seal the gap between the plates and will allow full transfer of stresses between the old and new plates. Seating bolts in old pitted and uneven plates can also cause difficulties, which can be overcome by using metal-filled putty to build up a flat seating surface. The design of the repair must, of course, allow for the weakening effect of the new bolt holes on the original section.

Plating by bolting is not suitable for cast iron structures: drilling and bolting through parts of cast iron members subject to tensile stresses is inadvisable. Much of this material is variable in composition; it may contain flaws or inclusions, which, in conjunction with new bolt holes, could seriously reduce its ability to resist tensile stress. In this instance, adhesive bonding of FRP plates is often the only alternative. Figure 26.3 shows an example of the use of FRP bonded plates to strengthen the tensile flanges of cast iron girders, together with curved and tapered FRP cruciforms bonded to deck plate downstands. In this example, FRP strengthening was undertaken during possession of the railway under controlled environmental conditions.

Further examples of FRP plate bonding to strengthen cast iron girders in London Underground tunnels are described by Silva and Church (2005) and show the versatility of this technology in severely constrained situations. Fibre-reinforced polymer plate bonding has also been used to strengthen more modern steelwork bridges (useful guidance and examples are described in Cadai *et al.* (2004)).

The repair of riveted and bolted structures by plating requires particularly careful planning and design to maintain structural integrity and to effect trouble-free site operations. Corroded rivets can be drilled out and replaced with bolts; however, the member or entire structure must be relieved of load or the operation must be performed in a strictly controlled fashion, with strictly limited rivet removal at any time, to avoid weakening the structure. Strengthening plates or sections for riveted structures must be detailed with clearance holes to clear rivet heads. Plating the underside of riveted bottom flanges over cover plates or splice plates might be impracticable without relieving the member of load, drilling out the rivets, removing the entire cover plate and replacing it with a heavier section.



Figure 26.3 FRP plate bonding of cast iron girders and deck plate downstands (left) using curved tapered FRP cruciforms (right) (photograph courtesy of Mouchel Parkman)

Attaching new steel plates to corroded structures by welding is generally more problematic than attaching by bolting. In the first place, the designer must be satisfied that the material to be strengthened is weldable. Cast iron, wrought iron and many early steels are not of weldable quality. Secondly, the designer must ensure that the welded connection detail will not cause stress concentrations in a location subject to cyclic loading and lead to future fatigue problems. The effect of the welding process on the stresses in the member to be strengthened must be carefully evaluated and the welding procedures carefully planned to avoid creating distortion and additional stresses.

In repairing members subject to tensile or compressive loading by plating, particularly those composed of built-up sections, care must be taken not to apply additional plating eccentrically as this could produce unforeseen eccentric load effects, resulting in an effective reduction in section capacity, rather than the strengthening intended.

Repair of cracking

Cracking in metal bridges results from several causes, depending on the type of structure and the material. Fatigue cracking is likely to be most serious in modern welded steel structures and is less common in riveted construction. Cast iron, being a less ductile material than steel or wrought iron, is more prone to cracking caused by impact and overstress; low temperatures can be a contributory factor.

Brittle cracking in cast iron is difficult to repair effectively because of the problems of bolting and welding to the parent material. Proprietary systems for the repair of cracks in cast iron structures, such as the Metalock system, can achieve good results. The system works by machining both edges of the crack to produce an interlocking profile and driving in a stitching element to connect the two edges. The technique depends on specialist equipment and licensed operators and might be difficult where access is a problem. The costs and likely success in

terms of eventual load capacity of the member should be evaluated with care. The technique may be particularly appropriate for older decorative fascia beams, when preserving the appearance of the structure is important, and loading can be relieved by additional structural members provided behind the fascia.

Fatigue cracking caused by stress concentrations or out-of-plane bending in web or other plates is usually more severe in welded construction, but is also found in riveted members. Cracks originating in welds will often propagate into the parent metal of the section if not repaired. Minor fatigue cracking in plates caused by stress concentrations and out-of-plane displacements can be effectively repaired by hole drilling at the end of the crack – thus reducing the stress concentration to a point at which no further propagation will occur. The hole can be filled with sealant and painted over. This technique might not be effective in stopping crack propagation if the out-of-plane displacements are large. A more effective long-term remedy may be to provide an alternative load path, stiffening the connection and preventing the displacement of the affected plate. Alternatively, it may be possible to increase the flexibility of the connection, provided this results in a reduction in the relative displacement of the affected plate. Major cracking that has seriously weakened the section might require a repair of the affecting area by cutting out and replacing the damaged plate, or by plating over the cracked section.

Cracking in welds caused by fatigue or faulty welding is usually repaired by grinding out the affected length of weld and re-welding. Some success in the repair of small (less than 3 mm deep) weld cracks that have not propagated far has also been achieved by peening the weld with a mechanical air hammer, making the weld more resistant to fatigue. Peening plastically deforms the weld and induces a local residual compressive stress zone in the weld and parent metal, preventing the formation and propagation of cracks.

All of these repair techniques might be ineffective in the long term if the underlying cause of the fatigue damage is not

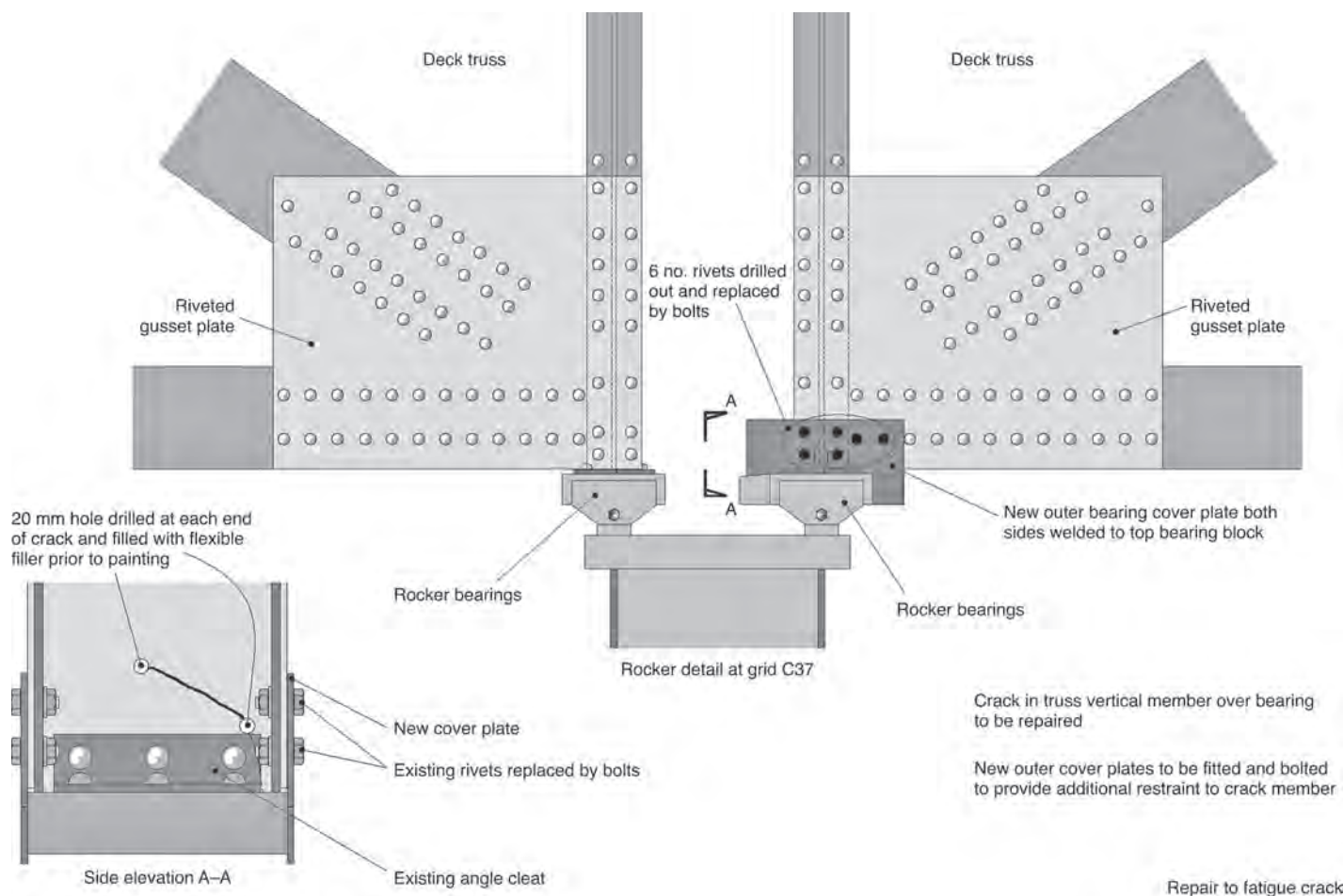


Figure 26.4 Repair of fatigue cracks caused by out-of-plane displacement (courtesy of Mouchel Parkman)

addressed in the repair. It is essential that a thorough evaluation is made of the load effects and displacements that have caused the stress cycling at the affected area. It may be possible for alternative load paths to be provided to prevent displacements and stresses being induced. Alternatively, making a connection more flexible may actually reduce its proneness to fatigue damage, provided the relative displacements in the affected part are reduced. An example of measures taken in response to fatigue cracking caused by out-of-plane displacements is shown in Figure 26.4. Holes were drilled at the termination of existing cracks, rivets replaced and new cover plates fitted to stiffen the joint and provide alternative load paths.

Member replacement

The most effective and economical repair strategy for severely damaged or corroded members may be complete replacement. Should replacement of an entire member be contemplated, the integrity and stability of the whole structure during removal and replacement will require careful consideration. Analysis of the

original structure will indicate the levels of load to be expected in the affected member under dead load alone. Provided that the original member dimensions, properties and fixing details have been accurately duplicated, and the replacement scheme induces displacements across the whole structure that are identical to the originals, the loads in the replacement member and the other members will be the same as in the original structure. A means of checking the level of loading in the new member, using strain gauges, is often provided as part of the replacement scheme to ensure that the original distribution of loading between all members of the structure has been reproduced. In some cases, it might be necessary to provide a means of adjusting loading in the replacement member, by jacking, for example, to ensure the correct final distribution of loading between members.

The example of member replacement shown in Figure 26.5 illustrates the complex sequencing and temporary works required to replace severely corroded members forming part of the main deck truss of a suspension bridge while the bridge remains open to traffic.

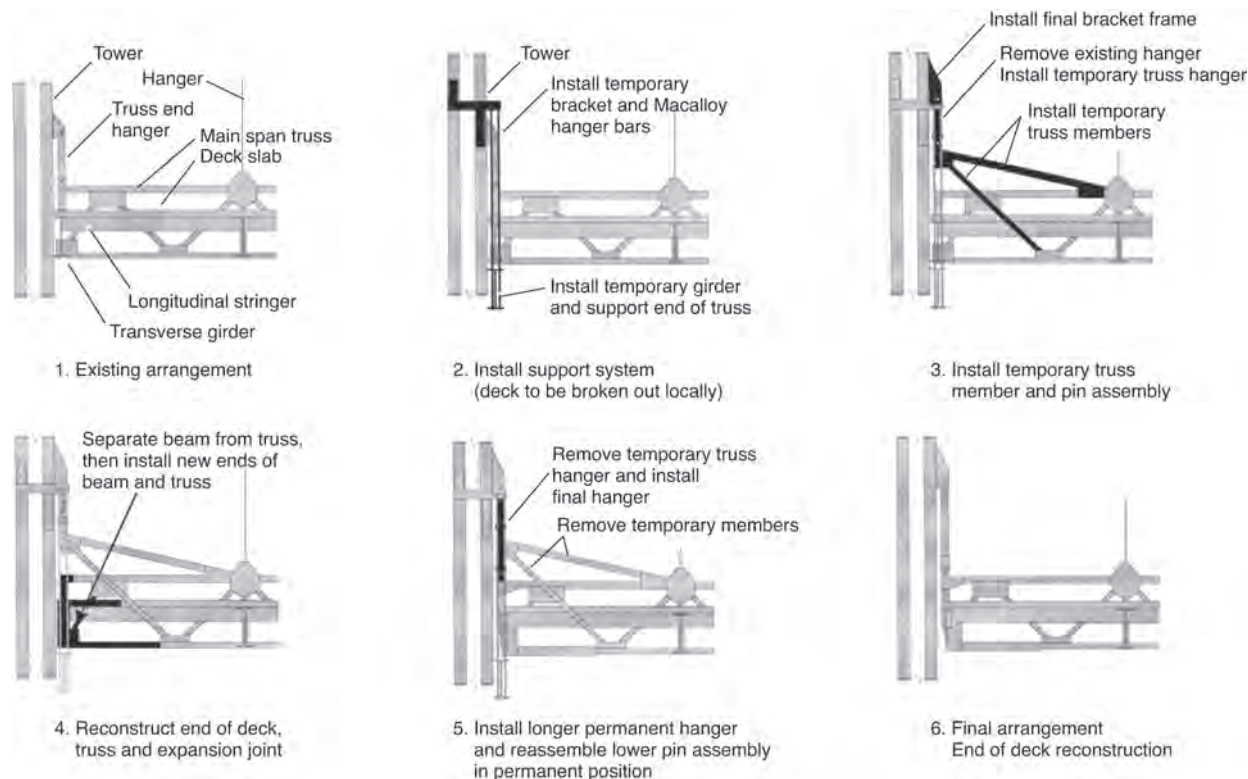


Figure 26.5 Example of member replacement – reconstruction of the end of the main deck truss of the Duarte Bridge in the Dominican Republic: the reconstruction involved the replacement of parts of the truss longitudinal members and the end transverse girder; the end of the truss had to be supported at all times during reconstruction, enabling the bridge to remain open to traffic (courtesy of Mouchel Parkman)

Strengthening of metal structures

Strengthening of metal structures employs many of the same techniques as repair. Plating of under-strength members is often used to increase their live load capacity. As previously stated, the application of additional thicknesses of new steel or FRP plate to a member cannot increase its dead load capacity (unless the dead load is temporarily relieved during strengthening), and will actually contribute to the dead load to be carried by the original member (although this will be insignificant for FRP plate bonding). Strengthening against bending by steel plating in this way is therefore relatively inefficient. Designers might find that the additional capacity they seek cannot easily be achieved by steel plating, in members in which dead loads predominate, as the new steel plates increase dead load stresses so much that the gains in live load capacity are almost cancelled out.

Strengthening by steel or FRP plating

Strengthening plates can be fixed to the original member by bonding (for FRP plates in particular), bolting or welding. The detailing of fixings to the member is very important to avoid the creation of traps for water and debris, which would lead to future corrosion problems. In the case of welded fixings, there is the potential to create fatigue problems if the joint is incor-

rectly designed. Carrying plating into joints is also a problem. Bolted or riveted joints have usually been designed to allow only sufficient fasteners to be positioned to resist the originally anticipated loads. Strengthening joints can be a very difficult exercise, often requiring partial or complete reconstruction of the joint to allow new and larger gusset plates to be inserted, with more space for additional bolts.

A notable example of strengthening of steelwork on a major bridge is the truss end link strengthening works undertaken on the Forth Road Bridge in the UK. Fatigue cracking of critical steelwork elements caused the bridge to be closed for a period and required additional steelwork stiffeners and sliding bearings to be installed (Scottish Parliament, 2016). Strengthening by increasing the depth of an existing girder by bolting new steelwork sections, or introducing new cross-girders, is also a useful method that is often used where space and headroom permits (Collin *et al.*, 2010).

Prestressing or load relieving

The problems of strengthening structures in which dead loading exceeds live loading can be addressed by stressing additional or replacement members into place, and thus altering the load distribution in the existing structure. Several different methods of achieving the desired result have been developed, for example, jacking additional sections into box or built-up

column sections in order to reduce the compressive stresses in the existing member and thus provide additional capacity. In contrast, large truss bridges have been strengthened by the addition of new prestressed tension members in place of existing unstressed members in order to alter the load distribution in the truss and reduce the levels of dead load stress in critical members.

The design of schemes involving the strengthening of compression members by jacking load into new sections, which then become a part of a new composite member, must pay particular attention to the local and global buckling of the new and existing members. The new member, unless particularly stocky, will need to be restrained by the existing member at points along its length to prevent buckling. The restraints must be designed to act effectively as lateral restraints, while allowing the new member to slide vertically as load is jacked into it. Use of this jacking technique, in conjunction with load measurement systems, allows a high degree of control in the proportion of vertical load carried by the new and existing members. Careful detailing of the baseplate and capping plate areas will be required to ensure even transfer and distribution of load. Flat jacks are often used as the means of loading the new members. This technique is illustrated in Figure 26.6, which shows the insertion of additional strengthening members in the main towers of a suspension bridge, preceding re-cabing and general strengthening works.

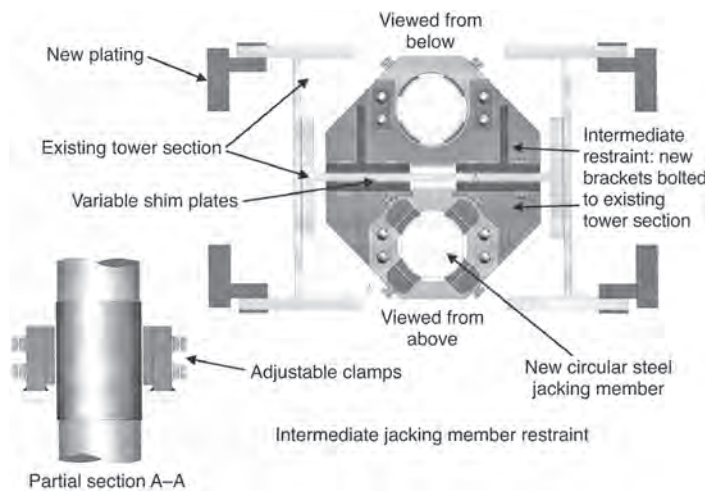


Figure 26.6 Strengthening of compression member – the strengthening of the towers of the Duarte Bridge in the Dominican Republic is illustrated: the figure shows the new circular members that will relieve the existing section of load and the intermediate restraint brackets within the existing towers (courtesy of Mouchel Parkman)

The principle of relieving load by prestressing new members into a structural system can also be used to strengthen individual members. Large steel plate girders have been strengthened in flexure through the addition of stressed bars positioned below their lower flanges. The bars were stressed from fabricated anchorages bolted to the underside of the beam's lower flanges and extended over the overstressed length of the central part of each span. Strengthening in this way relies not on the

addition of large amounts of supplementary material but on the alteration of the member's existing stress state. The prestress, by reducing the levels of tensile stress in the bottom flange of the beam, effectively releases some of the 'locked-in' dead load stresses, which can then be used to resist increased live loading.

A new variant of this technique has recently been used to strengthen a cast iron highway bridge using prestressed carbon FRP plates. In this application, fabricated steel anchorages were clamped to the lower flange of the cast iron beams. The composite plates were then stressed and attached to the anchorages, which transferred the prestress force to the cast iron beam. The composite plates were also fully bonded to the underside of the flanges by epoxy adhesive and thus acted compositely with the beam.

A further means of altering the existing state of stress in a structure is by reprofiling. This reduces levels of tensile or compressive stress in critical members, and thus achieves an increase in structural capacity of the structure, by inducing controlled amounts of load – generated by displacement of the structure – in critical members. For example, this principle has been used to increase the live load capacity of deck trusses in long-span suspension bridges. By adjusting the lengths of hangers in order to produce a greater upward curvature of the deck under dead load alone, compressive forces are induced in the bottom chord of the truss, and tensile forces are induced in the top chord. These forces are in opposition to the forces resulting from dead and live loading; therefore, the net effect is to reduce the forces in the truss under dead loading alone, increasing the live load capacity of the truss. The forces induced in the truss can be readily calculated to enable the required amount of adjustment of deck levels to be determined. The operation must, of course, be carried out in a carefully controlled manner to avoid local overstress of truss members.

In a similar manner, if it is possible to adjust the level of the supports of a continuous bridge girder or truss by jacking, moments can be induced over intermediate supports, which can counteract the hogging effects of dead and live load. The usefulness of this technique is often limited by the practical difficulties of achieving the required amount of vertical movement at the abutments or intermediate supports within the constraints of highway or railway vertical alignments.

Load relief on highway bridges over the railway comprising cast iron girders (which are particularly sensitive to dead load) and fill has been undertaken by replacing the fill with lightweight concrete; this method can be used to effectively encase the upper section of the cast iron girders to improve the section modulus. This method has also been used in combination with FRP strengthening of the cast iron girders to improve flexural capacity.

A more recent load relief method utilising FRP cellular decking systems has been used in North America to replace a deficient steel deck on a steel bascule bridge (Figure 26.7). Fibre-reinforced polymer decking systems are very lightweight, typically approximately one-fifth of the weight of a concrete deck for the same construction depth. Using this



Figure 26.7 Replacement of steel grating deck with lightweight durable FRP deck on steel bascule bridge (photograph courtesy of Martin Marietta)

method, 1200m² of deck was replaced in only 2 days using sections of FRP deck installed by small cranes. This method of load relief is becoming more common, owing to the benefits of simple installation and durability provided by FRP decking systems. This is particularly the case for lifting bridges – the replacement of a timber deck with an FRP deck on the steel-work Grasshopper bascule bridge in Denmark enabled load relief and improved footpaths to be installed, and a similar project was completed at Broadway Bridge, USA (both described in Gradeci, 2013).

Repair and strengthening of masonry structures

Introduction

The masonry arch (see Figure 26.8) is the most common form of structure in the UK bridge stock, amounting to some 40% of the total (Page, 1993). Most of these structures have been subjected to prolonged exposure to environmental effects. They have also been subjected to loading well beyond the expectations of their original designers. Therefore, it is not surprising that these bridges suffer from a variety of defects. Effective repair and strengthening of masonry arch bridges is essential if their future is to be safeguarded.

The inspection and investigation process

Guidance on how to carry out the inspection of masonry arches is well documented (DoT, 1983; McKibbins, 2006) and includes excellent advice on aspects of wildlife conservation and environmental impact relevant to arch inspection and maintenance. The inspection should seek to identify defects and their causes; helpful checklists of faults have been produced (DoT, 1983; Welch, 1995). The presence of water plays a significant part in the deterioration process of masonry arch bridges, so the management of water is vital to the successful maintenance of arch

bridges (Pearson and Cuninghame, 1998). Therefore, it is particularly important to identify potential causes of water penetration, such as seepage from broken or blocked drainage systems. Seepage may be evident on an arch soffit, through cracks at the edges of the arch barrel (Newbegin and Shelley, 1995) or through spandrel walls.

The condition of the road, footpath and verge surfaces should be examined for faults that allow water into the fill. Service trenches cut adjacent to the spandrel and wing walls might allow water into a particularly vulnerable part of the structure (Page, 1996). Excavations relating to service company activity might also have damaged the arch ring itself. The effectiveness of historical road widening methods should also be noted.

A visual inspection should be supplemented by carrying out a hammer tap test. This is an acoustic method of detecting separations between the brick rings of a masonry arch by striking the masonry with a hammer. Ring separation is indicated by a hollow sound. The method is subjective and the sounds detected can vary within areas of any one arch, between arches of

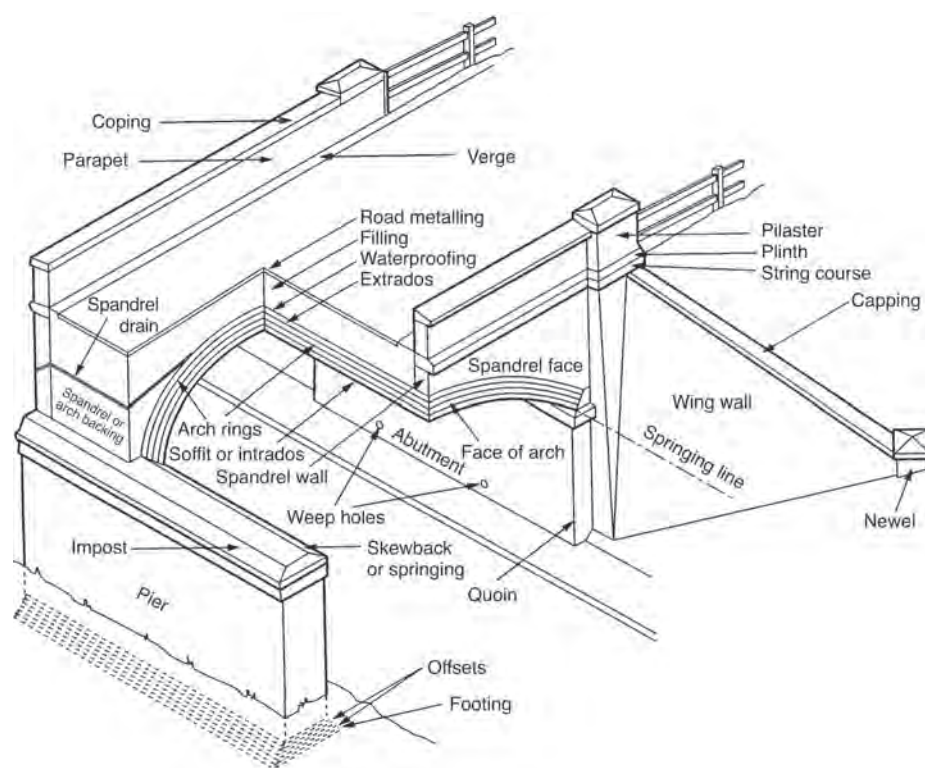


Figure 26.8 Typical construction of a masonry arch bridge (Sowden, 1990)

the same bridge and between different bridges. The test can only pick up separation nearest to the intrados of the arch.

Further data can be obtained by coring through the arch ring to confirm the thickness of the arch barrel, which may vary across the width of the bridge. Particular care should be taken in this regard when assessing a bridge. The condition of the brick core will give a reasonable indication of the presence of ring separation but the coring method can disturb the parent material such that it becomes difficult to assess the condition of the brickwork itself. Information on fill material will have to be obtained from trial pits excavated in the carriageway. This activity causes traffic delay and care is required with the reinstatement to avoid the damaging effects of dynamic wheel impacts.

A useful tool for gathering information on masonry arch bridges is impulse or ground-probing radar. The arch can be surveyed from the deck level alone but a more complete picture can be obtained by surveying the intrados. Information can be obtained on the thickness of different material boundaries and the thickness and extent of any saddling and haunching (Millar and Ballard, 1996). Ideally, the survey should be calibrated by trial pitting in non-sensitive areas and by taking a number of cores. Highways England guidance gives additional useful information on the use of non-destructive testing techniques for surveying the structure of masonry arch bridges (HE, 2020a).

It is essential that the cause of deterioration is understood before the most effective repair or strengthening method can be determined. The effect of a repair on the behaviour of the existing structure must be considered. If the inherent articulation of the stonework or brickwork is lost as a result of the repair, it may have a long-term detrimental effect on the fabric of the structure. A series of defects and a range of possible repair and strengthening solutions (Page, 1993) is given in Table 26.9. Further details are given elsewhere (McKibbins, 2006).

Identification of defects

Deteriorated pointing and brickwork

Masonry arches can suffer from a loss of pointing, mortar decay, spalling and delamination of brickwork. In the more severe cases, splitting and disintegration might occur, leading to a complete loss of bricks. If left unchecked, there will be a loss of strength in the arch, leading to barrel deformation and cracking. The main cause of this deterioration is often weathering and frost action assisted by water penetration (Newbegin and Shelley, 1995). The standard of bricks plays a significant part in the rate of deterioration. A good quality brick can often provide a second line of defence following the breakdown of a waterproofing and drainage system. Test cores can be used to determine the extent of frost action and damage, which may be limited to the outer courses of brickwork.

Sulphur compounds in the air or in rainwater can play an important part in the deterioration of brickwork. Mortars might be attacked by sulphates derived from the bricks themselves. Attack is gradual and occurs when the brickwork remains

Defect	Possible solution
Accumulation of debris and vegetation	Routine maintenance
Deteriorated pointing and brickwork	Repoint Brickwork repairs Coating of masonry
Arch ring thickness assessed to be inadequate to carry required traffic loads	Saddle Sprayed concrete to soffit Prefabricated liner to soffit Retrofitting of reinforcement Grouted anchors Asphalt overlay
Internal deterioration of mortar, e.g. separation between rings of a multi-ring brick arch	Grout arch ring Radial anchors
Weak fill	Replace fill with concrete Grout fill if it is suitable
Foundation movement	Mini-pile Grout piers and abutments Underpin
Scour of foundations	Underpin Construct invert slab
Outward movement of spandrel walls	Fit tie-bars Replace fill with concrete Take down and rebuild Grout fill if suitable
Arch ring – splitting below spandrel walls	Stitch (short tie-bars spanning the crack)
Water leakage through arch ring	Waterproof road surface Waterproof arch ring extrados Improve drainage
Progressive deterioration	Waterproof and strengthen

Table 26.9 Defects and remedial options (Page, 1993)

wet for long periods. Salts crystallise within the pores of the masonry, causing stresses to develop, which can cause local fragmentation of the stone or brickwork.

Arch ring thickness assessed to be inadequate

The behaviour of masonry arches is well documented (Heyman, 1982; Page, 1993) and is described in Chapter 13. The strength of these structures is heavily influenced by the quality of the materials used, the quality of the workmanship involved in the construction and the geometric proportions of the design (DoT, 1983).

It is the wide range of configurations and materials that have been used in the construction of these bridges that makes it difficult to assess their structural condition accurately. Even though these bridges have inherent reserves of strength, the real problem is the progressive wear and tear of the structure. Consideration of assessment methods is detailed elsewhere (Hughes and Blackler, 1997) and is beyond the scope of this chapter. However, it is pertinent to note that work continues to improve the reliability of assessment methods (Fanning and Boothby, 2003; Harvey, 2006).

Arch ring separation

Ring separation is a common problem with multi-ring brick arches and is associated with the loss of bond between successive rings caused by weathering or stress cycling of the mortar. Unless a detailed examination has been carried out, as described previously, engineers have only been able to make allowance for this defect through the use of a subjective condition factor. Tapping with a hammer is a simple but limited way of attempting to detect its presence.

Work at Bolton Institute (Melbourne, 1990; Melbourne and Gilbert, 1993) has attempted to determine the significance of defects on the load-carrying capacity of masonry arches by testing both models and full-scale two-ring brick arches. The conclusions of these studies were that ring separation caused a reduction of between 56% and 33% in the ultimate load-carrying capacity of models and full-scale arch bridges, respectively, thereby confirming the importance of this defect.

Weak fill

The behaviour of fill material, particularly for deep arches and spandrel walls, is critical to the performance of the bridge. The major problem likely to affect fill is that the road surface waterproofing or the drainage breaks down and the fill becomes saturated. Fines may be washed out of the fill, leading to voids. Water percolating through the arch ring is likely to lead to deterioration of the mortar. Saturated fill will substantially increase the lateral pressures on spandrel walls, with even greater pressures if the fill freezes in winter, perhaps leading to outward displacement of the wall (Pearson and Cuninghame, 1998). Deformation of the fill will lead to an uneven pavement with consequent increase in dynamic wheel loads and possible damage to services, such as gas pipes.

Foundation movement

Arch rings generate pressure on their abutments, which might lead to outward movement. The fill behind abutments will resist this movement, which might in itself cause inward movement. The effect on the arch ring will depend on the direction of movement and whether it is accompanied by rotation of the abutments. Transverse cracking in the arch ring is likely to occur. Recent cracks are a cause for concern, as they indicate that fresh movement is taking place.

Movement of foundations is likely to cause the loss of bedding mortar between components of an arch and, in severe cases, the displacement or loss of brick and stone blocks (DoT, 1983). If one edge of the bridge settles, longitudinal cracks will occur in the arch ring. This may be serious if the ring divides into effectively independent segments. If one abutment tilts relative to the other, diagonal cracks are likely to occur, starting near the side of the arch at a springing and spreading towards the centre of the barrel at the crown (Page, 1993).

Scour of foundations

An assessment of scour should be included in an overall review of an existing bridge. Scour is probably the most common cause of collapse of masonry arch bridges (Page, 1993). The foundations are generally shallow and therefore susceptible to scour. The potential consequences of scour (May *et al.*, 2002) include

- pier settlement owing to loss of support to foundation
- pier tilting
- abutment settlement or tilting
- damage owing to hydraulic loading, perhaps aggravated by debris accumulation
- damage owing to sediment abrasion, boulder impact and debris abrasion
- scour hole or washout of embankment behind abutment
- twisting of arch owing to differential abutment or pier movement
- loss of intrados or spandrel masonry owing to suction or washout
- total or partial collapse.

Scour can be difficult to detect because it is likely to be at its worst when the river is in flood and access is impossible. However, proven methods are now available (Meadowcroft and Whitbread, 1993; Riddell, 1993). In particular, river bed profiles may be determined by poling or using a lead line. More recently, a number of further useful documents have been published on this topic (Highways Agency, 2012; May *et al.*, 2002; McKibbins, 2006). It should also be noted that the assessed capacity of a flooded arch can be significantly lower than that obtained from a conventional assessment (Hulet *et al.*, 2006). It is recommended that when assessing masonry arch bridges, the risk of flooding should be considered.

Outward movement of spandrel walls

Spandrel walls suffer from the normal problems associated with exposed masonry, such as weathering and loss of pointing. They are also frequently affected by dead and live load lateral forces generated through the fill or as a result of vehicle impact on the parapet or by freezing of the fill. The effect may be outward rotation, sliding on the arch ring or bulging. Cracking of the arch ring beneath the inside edge of the spandrel wall is more likely to be caused by flexing of the ring, as described next. Spandrel walls are assessed visually as no formal analysis method exists, largely because of the very complex nature of their behaviour. It can be very difficult to ascertain the construction details. Lateral arching within the spandrel may increase its flexural strength. The strength of masonry has been shown (Thompson, 1995) to have a significant influence on the stability of the spandrel wall.

Arch ring – splitting below the spandrel walls

Spandrel walls stiffen the arch ring at its edges. The mechanism for cracking is thought to be differential movement between the relatively flexible brick arch and the stiff spandrel walls under live loading exacerbated by earth pressure on the spandrel walls. This type of failure might be assisted by rain-water getting into the structure at the parapet or surface joint and causing damage to the arch ring mortar where the spandrel wall meets the ring (Pearson and Cuninghame, 1998).

Water leakage through arch ring

The presence of water can cause a loss of strength in arch rings. Seepage of water washes out mortar and, when associated with freeze–thaw cycles, is responsible for spalling and splitting. When the water carries salts, it may leaving inorganic crystals behind on evaporation from the surface; these can push the surface off as flakes.

Progressive deterioration

There is considerable concern that some masonry arch bridges might be deteriorating rapidly as a result of high-frequency loading caused by increasingly heavy traffic and axle loads (Lemmon and Wolfenden, 1993; Pretlove and Ellick, 1990). It has been reported (Powell, 1997) that bridges that have shown no signs of distress for many years can suddenly deteriorate. The former British Rail Research department tests also show fatigue to be a problem, particularly when the masonry becomes wet. It has been recommended that this could be avoided by limiting the serviceability limit state for brickwork compressive stress to 50% of the ultimate compressive stress (Lemmon and Wolfenden, 1995). More recent work at the University of Salford (Melbourne *et al.*, 2005) showed that failure occurred under cyclic loading for certain tests at values of approximately half of that achieved in static load tests. It should also be noted that for some tests the failure mode changed from a classic four-hinge mechanism under static loading to one of ring separation under cyclic loading. The findings of further research at the same university suggest that where masonry arches start to rely on higher lateral earth pressure in the backfill to the arch to resist load effects, long-term degradation might start to occur, owing to significant changes in soil–structure behaviour.

Repair techniques

It is vital that repair work is carried out that is sympathetic to the appearance, structural behaviour and existing materials of the bridge. There have been many examples in the past where unsuitable materials have been used for remedial works that were incompatible with the stonework or masonry (Ball, 1997). Timely intervention is also required. Typically, a combination of comparatively minor individual items, such as inadequate pointing, open joints or outward movement of spandrels, together with water damage from a burst water main over the

barrels of the arches, can leave a structure in urgent need of repair.

Routine maintenance

Routine maintenance involves modest expense compared with the possible consequences of neglect. The following items should be carried out during routine maintenance:

- keep road surface in sound condition to avoid dynamic loading
- maintain road profile to assist in shedding water
- maintain waterproofing system (where one exists)
- maintain drainage system
- remove vegetation from structure
- make good small areas of deteriorated mortar.

Repointing and mortar

Repointing is widely regarded as essential and may improve arch load capacity by restoring the structurally effective arch ring thickness to full depth. If properly done when it is needed, it may prevent the bridge from deteriorating to the point where it needs more expensive repair work. If incorrectly done, it can accelerate deterioration of the structure. The mortar should not, for instance, be harder than the brick or stone. If it is too soft, the arch will continue to behave with a reduced effective thickness. The cost of repointing is modest compared with other techniques. However, if pointing is neglected for too long, the cost of restoration will increase dramatically. The appearance of the bridge can be enhanced and the work need not disrupt traffic while being done.

It has been shown that, in general, the most effective method of repairing old brickwork and stone structures is to use lime mortars and renders (McDonald and Allen, 1997). Unfortunately, many current repair schemes are having to reverse inappropriate work that was carried out earlier this century, for example, by replacing very hard impervious cement pointing (Blackett-Ord, 1996). Older masonry structures were invariably built using lime mortar, which is flexible and porous, and any moisture that entered the structure was allowed to evaporate through the joints. The introduction of cement had the effect of sealing any moisture into the structure, keeping it saturated and susceptible to frost damage. In turn, water pressure built up behind the facework, forcing it outwards. Lime-built structures can tolerate seasonal and minor structural movement without damage to masonry or joints. Any movement is taken up by minute adjustment within the flexible mortar beds over many courses of brick or stonework. Further advice can be found in the literature (McKibbins, 2006; Welch, 1995).

Lime mortar takes longer to harden than cement, and until it is fully set it is susceptible to frost damage. Lime gains strength slowly through reabsorbing carbon dioxide from the air. It will tend to dry out well before its full strength is reached, which can cause two problems:

- shrinkage: this can be controlled by the use of sand and the addition of fibres for render
- rapid drying: shrinkage can be exacerbated if drying is not controlled.

A combination of curing under damp hessian and adequate wetting of the substrate can solve these difficulties.

Brickwork repairs

Where the surface of the stone or brickwork has been eroded, the material can be reinstated using mortar repairs (Newbegin and Shelley, 1995); the following points, taken from Newbegin and Shelley (1995), should be taken into account.

- Employ an experienced specialist firm.
- A highly skilled workforce is required.
- Prepare a detailed specification.
- Grit blast the masonry and wash down dust and contaminants.
- Soak with an approved inhibitor to restrict fungal growth.
- Use a silicone-enriched mortar repair reinforced with a polypropylene mesh.
- Reinforce repairs deeper than 75 mm with stainless steel mesh.
- Secure the mesh using stainless steel pins.
- Use a dry mix with minimal shrinkage.
- Finish to profile by hand to match the texture of the adjacent masonry.
- Colour-match the surface layers to the adjacent masonry.

Brickwork delamination can be repaired by stitching the intrados back to sound brickwork using stainless steel reinforcing bars prior to filling the interspace with a free-flowing, non-shrink, cementitious grout. This technique has been successfully used to restore a railway viaduct (Mathews and Paterson, 1993).

Coating of masonry

Generally, exterior stonework or brickwork is not coated or impregnated by such materials as silane. However, in certain circumstances, such a process can assist in the water management of a structure. Coatings must allow the masonry to breathe, otherwise a build-up of water pressure might occur, leading to a failure of the coating or spalling of the masonry. The material should

- be effective at reducing water uptake into stone
- be effective at allowing salts into solution to pass through it
- not alter the original colour of the stone.

It is recommended that specialist advice be taken before any masonry coating systems are specified for use as a water-repellent (Pearson and Cuninghame, 1998).

Arch grouting

This technique is used to fill voids in the arch ring that are caused, for example, by ring separation. This will ensure that the full depth of section is available for load-carrying. It does not affect the appearance of the bridge unless grout extrudes from cracks and is not removed. The grout, which is usually cementitious, needs to be carefully designed to avoid premature setting before it has completely filled the voids and to ensure that its properties are compatible with the existing arch material (Page, 1993). High-pressure grouting might damage weak structures. To minimise this possibility, ensure that repointing of bricks is carried out and radial ties are installed (as appropriate) before carrying out the grouting operation.

Radial anchors

Grouted radial anchors have been used for many years to repair masonry structures. Their use as a strengthening technique following a load assessment is further considered later in this chapter. Radial anchors can be used to repair ring separation, arch face cracking and movement cracks between the arch barrel and the spandrel walls (see Figure 26.9). They can also be used to stabilise spandrel walls and for reinforcement in a wide variety of masonry situations.

Diamond drilling techniques are used to core through masonry barrel arches, or alternatively through the spandrels and fill, to allow the introduction of a variety of anchor systems. These systems usually employ either cementitious saturation anchors or cementitious anchors that utilise a polyester sock to prevent grout loss (Wardle, 1995). Anchors of different specifications can be mixed in the same project; the choice will depend on the site conditions.

This type of anchor can also be used to repair longitudinal cracks. Typically, alternate voussoir stones are cored laterally (30 mm diameter) and the cores retained. A 30 mm hole is then drilled normal to the spandrel and at mid-depth of the arch ring, to a length of some 760 mm beyond the crack to be tied (up to a maximum of 12 m). The anchor is then installed; the grout injected down the bar fills the sock, expanding it to key into all recesses. The cracks are then pressure pointed and the ends of the stone cores reinserted to plug the holes at the arch face (Welch, 1995).

Environmental impact

An appropriate repair technique should be selected, taking environmental impacts into account. A comprehensive treatment of the topic is given elsewhere (McKibbins, 2006). It is particularly important to note that, in many instances, it will be necessary to carry out a detailed survey of the potential implications for protected species, including bats, unless mitigation measures are implemented.

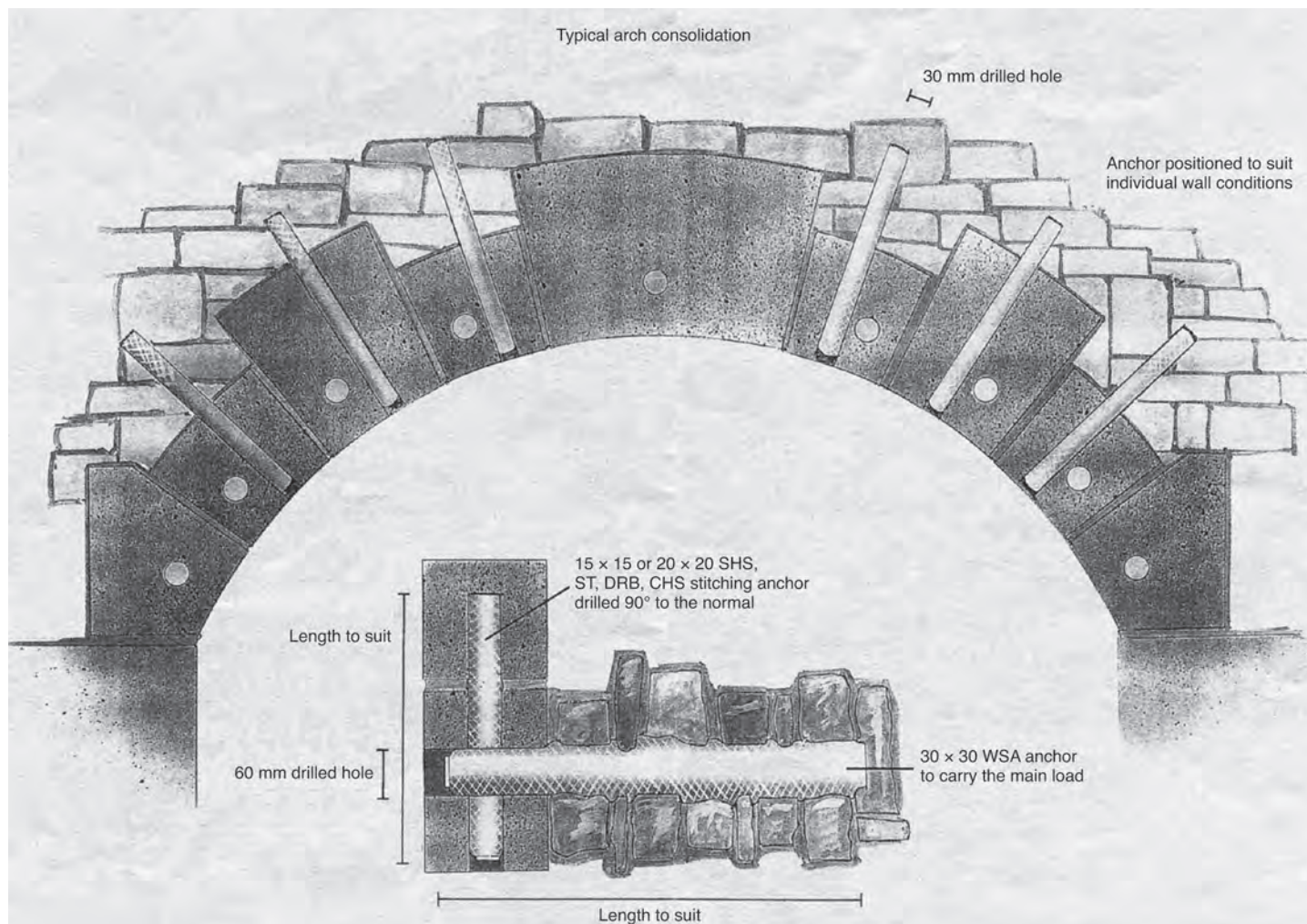


Figure 26.9 Radial anchors to repair arch face rings (courtesy of Cavity Lock Systems Ltd)

Further work has been carried out to assess the impact of various bridge maintenance options using the life-cycle assessment (LCA) technique, which provides the opportunity to compare environmental impact with more traditional socio-economic factors. The results of one study (Steele *et al.*, 2003) showed that the adoption of a regular maintenance regime, as an alternative to allowing a structure to deteriorate before a major intervention became necessary, offered a very significant reduction in environmental impact.

Parapets

British standards provide specifications for the design of highway parapets for bridges and other highway structures in reinforced and unreinforced masonry (BSI, 1999, 2010). Part of the derivation of these standards was based on earlier research and guidance (CSS, 1995; Gilbert *et al.*, 1995; Middleton, 1995). It should be noted that the standards only apply to unreinforced walls or panels longer than 10 m and to reinforced walls longer

than 2 m. The masonry arch design standard (HE, 2020b) presumes against the use of masonry parapets unless specifically approved. More recent guidance on masonry parapets is also available (DfT, 2012).

Parapets constructed in the course of repair or reconstruction should have an adequate coping. The use of concrete copings to the top of the parapet will prevent the downward penetration of water and will additionally direct water clear of the wall below. The use of half-round bricks and creasing tiles achieves a similar function and is aesthetically more pleasing. A capping, such as bricks on edge, does not direct water clear of the wall but is less liable to vandalism.

Strengthening techniques

It should be appreciated that the various sections of this part of the chapter are not mutually exclusive. Many of the repair techniques listed will also be used in a strengthening contract. The combination of applications will depend on the extent of defects on each particular bridge. It is, of course, of primary

importance that safety and arch stability are fully considered when evaluating a strengthening solution. A fuller treatment of techniques can be found elsewhere (Ashurst, 1992; Page, 1993, 1996). Careful programming is required to ensure that the various stages of the repair or strengthening project are carried out in the correct sequence.

A whole-life costing or life-cycle analysis philosophy is recommended. The objective should be to complete a durable solution with minimum traffic disruption at the time of construction. More recent advice on the use of all of these techniques is available (McKibbins, 2006).

Relieving slabs and saddling

The introduction of a secondary structure, such as saddling or a relieving slab (see Figure 26.10), may provide an excellent structural solution, particularly where historical widening can be integrated and the saddle can be waterproofed. Saddling involves removal of fill and casting an in situ concrete arch, which is often reinforced, on top of the existing arch. As such, the system will require relatively long periods to complete. The reasons for signs of distress and deterioration should be determined. For instance, movements of the abutments might cause cracking in the arch barrel. The addition of a saddle would lift the line of thrust, which might increase abutment movement and make the problem worse.

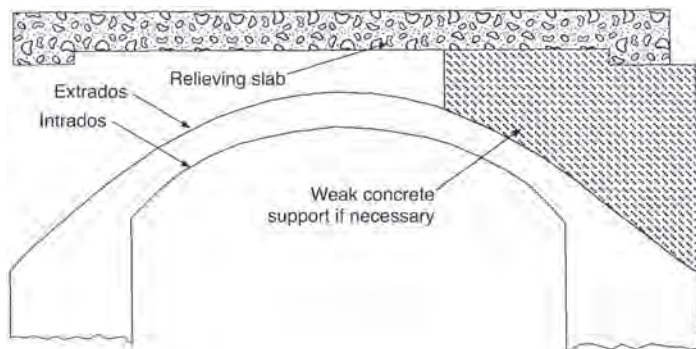


Figure 26.10 Arch relieving slab (Highways Agency, 1997a)

Codified rules (HE, 2020c) for the assessment of highway bridges indicate that arch barrels may be strengthened by means of a reinforced concrete saddle; however, there is no guidance for the assessment, or indeed the design, of the strength of such arch bridges. Work carried out at the University of Nottingham (Choo *et al.*, 1995) showed that, for the particular circumstances investigated, a 33% increase in arch barrel thickness could double the load-carrying capacity of the arch. The effectiveness of the technique was shown to be highly dependent on the bond condition between the brick and concrete. A parametric study was carried out using a two-dimensional finite-element program, which had previously been validated against tests carried out by the Transport Research Laboratory and others. The results were compared with experimental data

from two tests on segmental circular arches of 2.5 m span. The project indicated that the results obtained from a modified MEXE analysis were extremely conservative. Choo *et al.* (1995) cautioned that this method needed to be verified against further experimental data. In practice, a good bond is provided between the concrete, the arch and the spandrel walls, either by the use of ties and dowels or by the rough state of the extrados brickwork.

Consideration should be given to the transverse behaviour of the saddle (Page, 1993). Unlike in the longitudinal direction, there is little or no induced compressive stress; in fact, it is more likely to be in tension. The transverse restraint at the springings might be enough to cause cracking of the saddle. It is necessary, therefore, to give careful consideration to the sequence of casting.

At its simplest, over-slabbing merely consists of providing a load-spreading slab to reduce local load intensity. It is of benefit to the barrel, in that it allows the option of high-level waterproofing and it reduces lateral pressure on the spandrel walls (Welch, 1995). However, care is needed to prevent load being concentrated at the crown. When it is necessary to remove the intervening fill between the slab and the saddle, this should be replaced with weak concrete (7.5 N/mm²), sufficiently weak to allow some movement and allow arch action to take place (Welch, 1995).

The extent of excavation required may preclude the use of these techniques because of traffic management needs or the existence of utilities, or safety on grounds. However, the provision of a sound concrete surface does allow the arch to be effectively waterproofed. When this is carried out in conjunction with the provision of a maintainable drainage system, many of the water related defects described previously could be avoided.

Sprayed concrete

Sprayed concrete, or guniting, is an ideal material to use on circular sections, such as the intrados of an arch, to increase arch ring thickness. Premixed concrete is sprayed at high velocity and adheres on impact, filling crevices and compacting material already sprayed. A layer up to 300 mm thick may be applied. The concrete can be reinforced and, with care, satisfactory finishes can be achieved (Minnock, 1997). Unfortunately, the addition of a skin of concrete will add unwanted weight and stiffness, with reduced headroom. It does not enhance the appearance of the bridge and is very unlikely to be permitted on a listed structure.

Unfortunately, the lining may separate from the original arch by shrinkage of the concrete or by further deterioration of the arch material at the interface, which would mean that it would not increase the load capacity as much as if it were fully attached (Page, 1993). The method fails to address the most common cause of arch defects, water ingress from above. Rusting of the reinforcement would be a serious concern and every effort should be made to exclude water from the structure by other means.

Prefabricated liners

Arch ring thickness can be increased by attaching a corrugated metal or glass reinforced cement lining to the soffit as permanent formwork, and filling the space between it and the arch ring with concrete or grout (Page, 1993). Care needs to be taken that the void is fully filled. It is quick to apply and involves no disruption to services, but it reduces the size of the arch opening and does not enhance the appearance of the bridge.

Embedded reinforcement

Embedded reinforcement is also known as retro-reinforcement. Originally developed by the University of Bradford, Brunel-Atkins Ltd and Bersche-Rolt Ltd (Garrity, 1995a), it is a method of reinforcing existing masonry structures with little disruption to the original exposed finishes. It involves the installation of small-diameter stainless steel reinforcing bars into grooves, up to 75 mm deep, that have been previously cut into the surface of the existing masonry. Each reinforcing bar is encapsulated in a proprietary grout. A series of unreinforced and reinforced 2 m span clay brick model arch bridges was tested in the laboratory at the University of Bradford (Garrity, 1995b). Retro-reinforcement was simulated in the tests using thin strips of steel glued to the surface of the brickwork with an epoxy adhesive. The test results offered the following conclusions.

- Adding spandrel walls and parapets to an unreinforced arch ring produced an increase in the strength and stiffness.
- Reinforcement installed on the intrados strengthened and stiffened the arch by delaying the formation of hinges.

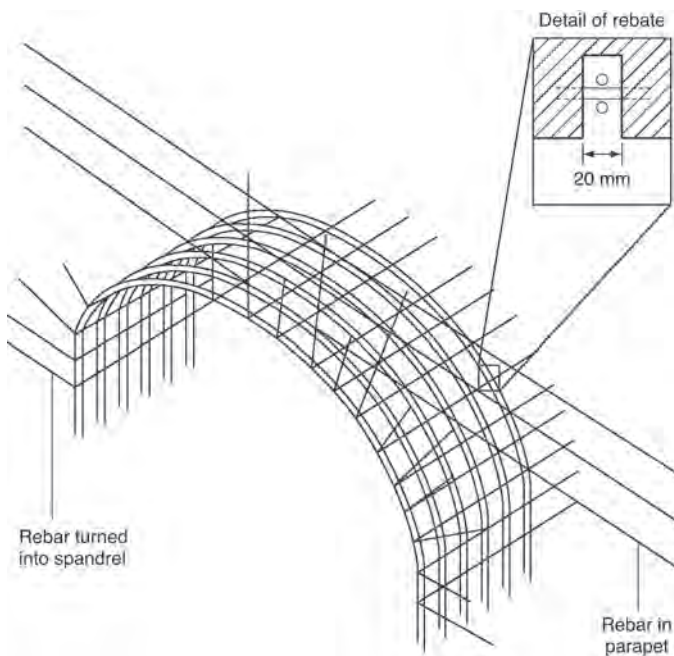


Figure 26.11 The MARS system of arch strengthening (Sumon, 1999)

- The small amounts of reinforcement used did not alter the fundamental behaviour of the model arch bridges.

Safety considerations associated with the design and installation of the temporary means of access and support required when carrying out any form of repair to a masonry arch bridge also apply with retro-reinforcement work. The reduction in structural integrity caused by the cutting of grooves was a major concern and it might be necessary to install the reinforcement in comparatively narrow bandwidths and to provide temporary support to the adjacent unreinforced masonry. The need to use high-strength grouts in order to develop the bond strengths required could result in the repaired zones being much stiffer than the surrounding unreinforced regions (Garrity, 1995b).

A number of commercial systems are now available. These systems generally consist of cutting rebates both longitudinally and transversely, then fabricating an in situ cradle (see Figures 26.11 and 26.12) of small-diameter stainless steel high yield rebar (Minnock, 1997). The bars are encapsulated with a low-modulus structural adhesive pumped under pressure to obtain high bond strength in the parent substrate and centralised reinforcement cradle. The formulation of adhesive used varies with each manufacturer. The exact method of installation of reinforcement also varies. One company has developed a way of installing the reinforcement by using circumferential drilling rather than by cutting longitudinal slots.



Figure 26.12 The MARS system under construction (courtesy of MARS)

Grouted anchors

An alternative method of retrofitting reinforcement in masonry arches is the Archtec system (see Figure 26.13). Developed jointly by Cintec Cavity Lock Systems, Gifford and Partners and Rockfield Software, the system was evaluated on the test rig at the Transport Research Laboratory (see later). The system was initially evaluated and is now designed using an advanced discrete element technique (Owen *et al.*, 1998), which had not previously been applied to bridges in an industrial environment. The efficacy of the system lies in accurate structural analysis using this method. On occasion, it has been shown that

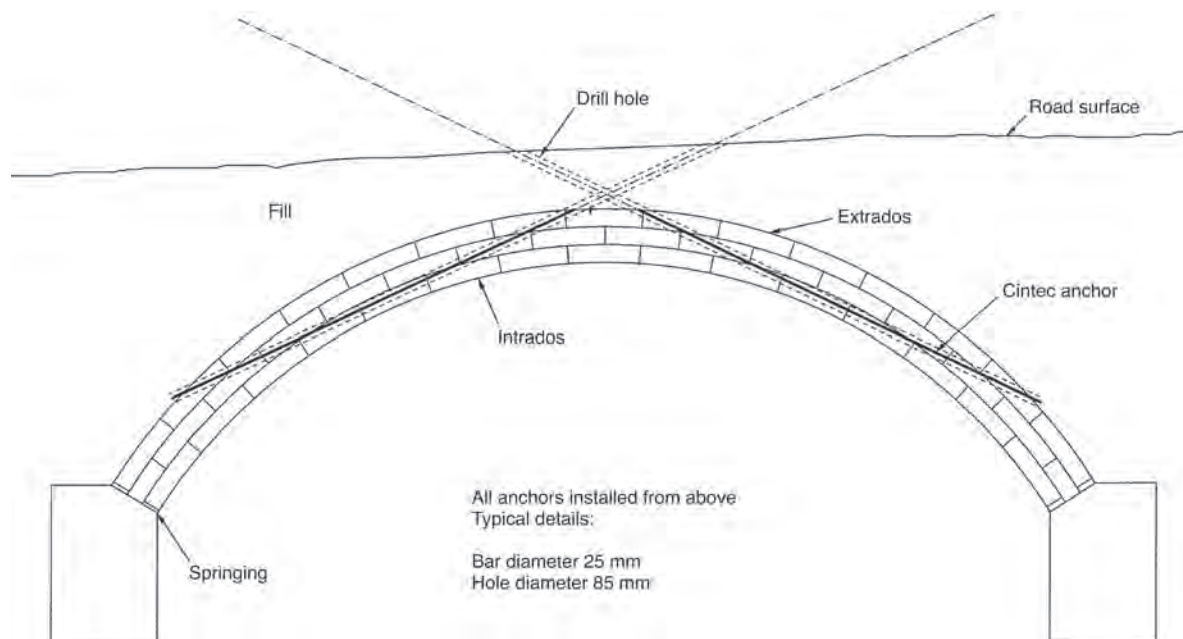


Figure 26.13 Elevation of typical Archtec arrangement for arch strengthening (courtesy Cintec Cavity Lock Systems)

there is unexpected hidden strength and strengthening is not required. More often, the analysis shows how reinforcement can be fitted to provide an optimum solution. The reinforcing components are standard Cintec polyester sock anchors. Very accurate rigs are used to diamond drill through the masonry so that anchors can be installed almost exactly along the thrust lines in the arch. The drilling can be undertaken from above or below the arch, depending on access, services and traffic management considerations.

In some cases, retro-reinforcement will only be appropriate if used in conjunction with other remedial work, such as pre-grouting of the masonry. It will not be possible to waterproof the arch using these techniques alone. Responsible and experienced contractors should be used and good supervision should be provided. Further details are described elsewhere (Ricketts, 2001). It has been recommended (McKibbins, 2006) that the following points, among others, should be addressed when procuring retro-reinforcement strengthening techniques:

- the basis of design and extent of verification undertaken, including the extent to which the design method takes account of the real condition of the specific arch and is not based on simple extrapolation of test results that might not be appropriate to the structure under consideration
- the adequacy of investigations and inspections undertaken to confirm the condition and structural dimensions, including confirmation that the degree of masonry weathering and condition of the arch does not make retro-reinforcement inappropriate
- the long-term durability of the reinforced bridge.

Asphalt overlay

An alternative approach to strengthening arches, by increasing the thickness of surfacing with a bituminous overlay, has been proposed (Fairfield and Ponniah, 1996). A series of numerical analyses was carried out at the University of Edinburgh on a 2 m span brick arch using a variety of techniques. A 0.1 m overlay was shown to increase arch load capacity by 61%, for that particular geometry only. The extra cover increases the capacity by allowing increased stress dispersal; this was verified in accordance with the codified method, Boussinesq's analysis and elastic finite-element analysis. The results of this research showed that this technique is cost-effective within certain constraints. If the arch shows signs of deterioration, as would be indicated by a reduced MEXE condition factor, other methods of repair should be used. If the structure is otherwise safe, Fairfield and Ponniah (1996) suggest that capacity increases could be achieved using the overlay method.

Grouting fill

Grouting of the contained ground above and behind an arch can be a useful measure. In suitably receptive grounds (not high in clay or silt), and in the absence of complications, such as services or drainage systems, the method is very effective and very economical. It increases the assessment fill factor to 0.9 and improves the arch ring condition factor by filling cracks and voids in the extrados. However, grout quantities can be hard to predict and considerable variation is to be expected (Welch, 1995).

Underpinning

This technique involves excavating material from beneath the foundations and replacing it with mass concrete (Page, 1993). Alternatively, a series of mini-piles could be used. A sequence of work is followed to ensure that the stability of the existing structure is not compromised.

Invert slab

This is a slab of concrete placed between the abutment walls or piers with its top surface at or below riverbed level (Page, 1993). It helps to prevent scour. If incorrectly installed, however, there is a risk of scour beneath the slab, particularly at its downstream end. This can be minimised by installing trench sheeting along the length of the invert slab. Where an existing invert, which acts as a strut between two supports, is being replaced, care should be taken to ensure that the stability of the bridge is not compromised.

Tie-bars and pattress plates

The introduction of tie-bars utilising pattress plates or similar devices (Figure 26.14) to distribute loading in bridges has been common practice for many years. These consist of a bar passing through the full width of the bridge, with pattress plates at each end, generally secured by a nut and washer, to provide the restraint to the spandrel wall. Traditional methods of installation involve the excavation of trenches in the fill material between spandrels and a percussive method of forming holes through the masonry for the installation of the bars. However, the excavation of trenches across bridges is often impractical. Tie-bars can corrode if they are not adequately protected. If the arch ring requires strengthening at the same time, a more common solution is to use a concrete saddle, which will also relieve the spandrel wall of outward forces (Page, 1993).



Figure 26.14 Arch strengthening with tie-bars

Spandrel wall reinforcement

The replacement of a small amount of fill adjacent to a spandrel wall with concrete on a wide bridge could be used to help stabilise outward movement of spandrel walls. When the whole of the fill is replaced, the technique is akin to saddling and is likely to be used to deal with arch and wall problems at the same time. Care must be taken to avoid damaging the spandrels. Trials have been reported of the use of non-metallic reinforced fill, which aims to reduce loading on the spandrel walls from the fill (Page, 1996). This technique can also be used to provide a drainage layer between the fill and the spandrel wall (Pearson and Cuninghame, 1998).

If there is sufficient width, or a road closure is acceptable, one solution would be to excavate behind the wall and rebuild it conventionally. To back the wall with mass concrete is a possibility, but to do so creates a deep, stiff beam edge to the arch, inconsistent in structural action with that of the arch. A more harmonious structural action results from the incorporation of a reinforced earth system to support the fill. This prevents excessive pressure developing against the spandrel wall and the space between the reinforced earth and the back of the wall is filled with single-size drainage material (Welch, 1995).

Water management

As noted previously, it should be an objective of a repair and strengthening scheme to waterproof the arch and provide positive means of drainage. The following points should be considered.

- Waterproofing should be tucked at the surface and above the arch ring.
- The road surface should be kept intact.
- Replace grass verges with paved surfaces.
- Service ducts should be encased in concrete, with waterproofing above.
- Provide spare ducts.
- Seal cracks along the kerbline, next to parapets and around street furniture.
- Ensure adequate reinstatement of service trenches and excavations.
- Realign and reprofile carriageways to shed water.
- Waterproof surfaces in contact with fill.

It is important to tackle the causes as well as the symptoms of drainage problems (Powell, 1990). Positive drainage outlets should be provided at the springing points of multispan arches, with an outlet through the piers. Holes can be cored through the brickwork (75–100 mm diameter) and a uPVC liner grouted in. The liner should project 25 mm from the stone or masonry face and be flush with the internal concrete. Perforated weep pipes can be provided within granular fill to assist in distributing

water to drainage outlets. Weep holes should be provided in abutments and cut-off drains should be installed across the carriageway at the end of the bridge, where topography dictates (Pearson and Cuninghame, 1998).

Part reconstruction

When arch ring damage is extensive, the only real recourse is to rebuild to a major extent. Construction is traditional in that it is necessary to build off centring, although several variations of constructional form have been adopted. It is best to knit the reconstructed section into the remaining structure by removing alternate stones or bricks. However, where an arch has been historically widened, it is best to leave this section intact and create a butt joint. The two sections would be united with a saddle, which would also allow for the incorporation of a waterproofing membrane (Welch, 1995).

Evaluation of techniques

Seven full-size model arch bridges were tested to failure at the Transport Research Laboratory (Sumon, 1998, 1999, 2005) to investigate the effectiveness of a number of strengthening methods. A test frame was constructed to enable a series of 5 m span, 2 m wide, three-ring brick arches to be built and load tested to failure. Spandrel walls and road surfacing were omitted in the models to reduce the number of parameters being studied. The fill was retained by a steel box, which was designed not to restrain movement of the arch ring. The seven types of arch tested, together with their reported failure loads, were

- an unstrengthened arch (242 kN)
- an arch strengthened with sprayed concrete (900 kN)
- an arch strengthened with a concrete saddle (701 kN)
- an arch with built-in ring separation (200 kN)
- a ring-separated arch strengthened using stainless steel mesh reinforcement (276 and 345 kN)
- a ring-separated arch strengthened using helical stainless steel mesh reinforcement (212 and 365 kN)
- a ring-separated arch strengthened using grouted anchors (410 kN).

The objectives of the test programme were (Sumon, 1998):

- to examine experimentally the increase in capacity of the arch ring provided by a variety of repair strengthening methods
- to use the experimental results to develop and calibrate analytical models that could be used as part of the design process for repair and strengthening schemes
- if possible, to develop new methods of repairing and strengthening that are economic and overcome the problems of existing methods.

Reconstruction

Highways England encourages the construction of new arch bridges and has prepared a new design standard (HE, 2020b). The Brick Development Association has published a design guide (Cox and Halsall, 1996) to give practical advice on brick arch bridge design, based on the experience of rebuilding Kimbolton Butts Bridge, in Cambridgeshire, in the UK. The behaviour of this bridge was extensively monitored and reported (Mann and Gunn, 1995; Prentice and Ponniah, 1995).

In some cases, it may be preferable to entirely reconstruct an existing arch in masonry, rather than consider a strengthening method. One example of such an application is Mytchett Place Canal Bridge, where it was necessary to widen an existing bridge in a conservation area; this is reported by Sains (1996). The choice of brick bonding is important, as ring separation can be a problem in header type arch rings. This can be eliminated by the use of English bond. This type of bonding provides a shear key between the rings (Halsall and Cox, 1994).

If the mortar is too strong in the reconstruction, it will encourage single cracks without distribution; if too weak, it may fail in compression. It is not sufficient to specify only a recipe mix or strength; the only satisfactory specification method is to specify a nominal mix and maximum and minimum strength, the mix design being established on-site, with a flexible mix being preferable. It is inherent in structural brickwork construction that strict control will be kept on the materials and that the batching will be closely supervised. Brick strengths are seldom a problem but brick sizes can be. It is important for detailers to remember that they are dealing with a natural material and that it might not be possible to achieve close dimensional specification. There are distinct differences between 'metric' and 'imperial' brick sizes.

It is important to check that the construction of centring does not permit undue deflection as the vaulting proceeds. Due allowance must also be taken for moisture movement in the forms, and that they can be readily eased and removed (Halsall and Cox, 1994). The ideal way to proceed with the construction is to remove the centring, backfill and complete the bridge before the pointing. There is a risk, however, that, during placing of the fill, distortion of the arch might take place. The centring should, therefore, be gradually eased rather than instantaneously removed. If 10 mm of sand is used in place of mortar adjacent to the centring, the production of quality pointing is considerably eased.

A number of points should be taken into account when considering altering existing bridges.

- The refurbished bridge should be as good or better than the existing, aesthetically, environmentally and structurally (Wallsgrave, 1996).
- Structures should not be cleaned more than is functionally necessary, that is, grime should only be removed if it is damaging the fabric, or if architectural detail is no longer visible.

- Use the latest research to avoid unnecessary intervention.
- Avoid short-term solutions that cause ongoing liabilities.
- Work is to be to the satisfaction of English Heritage (for listed or scheduled structures).

It is very important to consult the planning authorities when formulating works proposals to bridges that are listed buildings, scheduled ancient monuments or in conservation areas. A detailed before-and-after record survey of the bridge might be required. Certain restrictions might be imposed, which could, for example, prevent the removal of the existing fill – an alternative solution would then need to be devised. If at all possible, an aesthetically pleasing solution should be found.

A novel method, called FlexiArch, has been developed to enable fast and economic construction of a durable arch (Long *et al.*, 2013) and is now often used for the replacement of masonry arches. Although not a masonry arch in a strict traditional sense, the FlexiArch system comprises precast concrete voussoirs connected with polymeric reinforcement (hence avoiding potential corrosion problems) and can be masonry-faced if required. Spans of almost 20m have been constructed (longer-span applications are also expected in the future), and skew spans can also be achieved.

Many masonry arch bridge reconstruction projects in the UK are related to electrification of the railway, invariably requiring improved headroom above the railway. A new bridge jacking system specifically for masonry arches, which was developed and successfully trialled in 2016, gives an option that avoids reconstruction (Anon, 2016).

Conclusion

Masonry arches have served the nation well over many years. The causes of defects should be determined before selecting appropriate repair and strengthening techniques. Many new and innovative methods allow effective intervention without causing widespread disruption to the users of the transport system. An appropriate use of materials will allow the aesthetic appeal of the masonry arch to be retained and enhanced. Careful and sensitive maintenance, including the effective management of water, will allow this satisfactory state of affairs to continue for many more years to come.

Replacement of structures

Introduction

The general trend towards increased traffic loading requirements, and the consequent bridge strength assessment programme, has dramatically increased the number of bridges known to be below capacity for unrestricted traffic. Many of these structures are beyond economic strengthening; others are reaching the end of their useful life because of durability problems. There is often no alternative to replacing the bridge.

The design of replacement structures is essentially the same as for new works, except that there are considerably more restraints on the type of structure that may be used. The demands of the existing network, whether road, rail or waterway, must be considered; these will normally dictate the type of construction. Consideration must also be given to how the many services, such as water and gas pipes and electric and communication cables, will be handled; often these have to remain live throughout the replacement work. The disruption costs associated with bridge replacements can easily exceed the actual cost of the work. Minimising the construction time is of prime importance in reducing the disruption. This situation has led to the development of new techniques and innovative methods to replace bridge decks.

The health and safety implications of working in close proximity to live traffic also require particular care, for both the network user and the contractor. In many cases, these requirements also dictate the design chosen.

Legislation in the UK (the Traffic Management Act 2004) has placed a duty on all highway authorities to minimise disruption; this has increasingly led to replacement solutions that maintain the full capacity of the highway throughout the work.

The design and construction of any replacement bridge should make full use of industry guidance, particularly with respect to improving durability and minimising maintenance. Careful choice of the structural form and material provides an opportunity to avoid future costly maintenance and disruption; whole-life cost and carbon footprint will only continue to become much more important factors over the lifetime of the new bridge.

General requirements and restraints on closure

The same structural types are open to the designer of replacement structures as for new works. However, in most cases, the type of structure chosen will depend on different factors from new construction. What the bridge carries, the time available for replacement, site restrictions and health and safety, among others, will dictate the type of structure chosen.

Constraints imposed by the existing road, rail and waterway networks

The structure being replaced will be on one or more existing networks, whether highway, railway or waterway, and these will inevitably be affected by the construction of a replacement bridge.

The degree of disruption to the highway network depends on the traffic flow and suitability of alternative routes. Diversions and road or lane closures cause delays to the users, which have a cost in time or extra mileage; a notional value can be put on these delays to aid in comparison of different solutions. Traffic delay costs are evaluated using a computer program,

such as QUADRO (Highways Agency, 1997b). This takes into account the way vehicles divert onto alternative routes when subject to delays on the main route by determining the cost of the additional time spent queuing. Delay and access costs are very variable, according to the route and local circumstances.

The cost of disrupting a railway line is more easily quantifiable in terms of lost income, loss of use of the line and the provision of alternative services. Rail closures longer than 24 h will only be available at times when traffic is limited, such as over the Christmas period in the UK. Such closures need to be planned and booked with the railway authorities many months, or even years, in advance. Very short closures, typically up to 6 h long, are available more easily at nights and weekends. The railway will normally provide their own staff to manage the possession, the cost of which needs to be added to the overall possession cost. Any construction works during railway closures will require meticulous planning and recorded decision points to minimise the risk and severe consequences of any over-run into the operational railway timetable.

Waterways, such as canals and navigable rivers, usually have a closed season, typically from November to March. During this time, it may be possible to close the canal or river and replace the structure; outside these times, navigation may be restricted to certain hours by arrangement. At other times, protection of the canal traffic may be required, for example by temporary scaffolding or decking, to allow canal traffic during maintenance work without impinging on the minimum required headroom.

The key to successful timing of structure replacements is consultation with the network operators at an early stage to determine what requirements they might have. This information will dictate the type of structure required.

Public utilities and other services

Highway bridges carry a large number of underground and overhead utilities services belonging to, for example, the water, electric, gas and telecommunication authorities. These can be found located in service ducts on more modern bridges, but on the older structures they are just as likely to be built into the deck or suspended from the side or soffit of the bridge.

At an early stage, inquiries should be made with the utility companies to determine what services cross the bridge and whereabouts they are located. This information should be supplemented with hand-dug trial holes in or adjacent to the bridge to confirm the records and locate any other services not otherwise recorded. At this stage, the condition of the pipes or ducts needs to be investigated to enable decisions to be made about how to accommodate the service or whether a diversion is required.

During the first half of the twentieth century, cast iron was used extensively for pipework and can be very brittle and easily damaged; modern steel or plastic ducts are more robust and easier to handle and support.

The cost of diverting typical water or gas mains and optical fibre routes can be considerable. It is usually more economic,

wherever possible, to support the service and work round it, although there are then additional safety hazards associated with working adjacent to live services. Special temporary support and methods to provide protection for services will usually be required.

Utilities that cannot be diverted might need to be built into the new structure, limiting the possible design solutions. If possible, utilities should be placed on the new structure to allow simple maintenance access and minimise disruption (e.g. moving utilities from under the carriageway into a footpath verge).

Early consultation is essential; in the UK, this is normally carried out as part of procedures under the New Roads and Street Works Act 1991. Much of the work to accommodate services during bridge replacements may be undertaken in advance of the main contract, possibly causing additional disruption.

Safety considerations

Working adjacent to live traffic is extremely hazardous; setting up a safe system of work is particularly important. The primary objective must be to maximise the safety of the workforce and the public, with a secondary objective of keeping traffic flowing as freely as possible. It is vital that risk assessments are carried out at all stages of the development of the project, bearing in mind the potential hazards to the workforce and the public. For highway work in the UK, the Department of Transport has strict standards governing lane widths, road speeds and safety protection zones (DfT, 2009). Minimum lane width is 3.0 m for all traffic, reducing to 2.5 m for cars and light vehicles only. Safety zones are dependent on the road speed; 0.5 m for speeds up to 40 miles per hour and 1.2 m for speeds over 40 miles per hour. Working within the safety zone is not permitted. These restrictions will often dictate the extent of any partial reconstruction and need to be considered at an early stage. Temporary speed limits or road closures can take up to 3 months to arrange, so need to be considered at an early stage.

The needs of other road users, such as pedestrians and cyclists, must be considered and adequate provision made for them to cross the site safely. Where temporary bridges are used for this purpose, they must be designed for use by the disabled (Disability Discrimination Act 2005).

Railway authorities have strict safety policies that dictate the method of working to ensure safety of the operatives and the line. The provision of railway safety staff, and network planning and risk review, can be expensive and should be considered in estimating the costs.

The normal requirement of providing a workboat, life jackets, and so on, for working over waterways will apply. Navigable waterways will also require appropriate signage and might also require changes to the navigation; notice of such changes will be required some months in advance. Waterway authorities have strict guidelines for the control of pollution, which should be considered at all stages, especially during the demolition phase.

It might be more economic to consider a temporary working platform below the structure being replaced to allow work to proceed when the road, railway or waterway below is still in use. Over rivers, adequate clearance must be left below the platform to allow for debris to pass in times of flood. Platforms used over railways must be designed to allow for the suction caused by the passage of high-speed trains.

Demolition of existing decks

Demolition of the existing bridge decks needs particular care and should be considered at the same time as the design of the replacement deck. When considering phased construction, where part of the bridge will remain under traffic, the structural integrity of the old construction after part has been demolished must be maintained and should be checked at the design stage.

Working alongside live traffic and around live services, such as gas mains and electric cables, can be extremely dangerous; demolition work needs to be planned for safety. In the UK, guidance has been published by the Health and Safety Executive (1984–1988) and the BSI (2011). The utility companies have their own rules about the use of plant in close proximity to their apparatus. Similarly, river authorities and waterway companies will require protection to be installed to prevent water pollution. Disposal of the bridge, once demolished, will need to be considered in the context of recycling and reuse of materials. Bridges constructed during the mid-twentieth century might contain asbestos; special measures must be taken when working near and disposing of asbestos (The Control of Asbestos Regulations 2006).

The initial decision to be made is whether the bridge is to be totally demolished in situ or cut into transportable sections and removed to a more convenient location for breaking up. The time available and physical constraints might limit the choice. The number of methods of demolition available to the engineer has increased with improvements in water jetting and diamond cutting techniques. Increases in the capacity of cranes and jacking have also made the removal of large sections a practical proposition.

Structural considerations

Before demolition of the deck, or part of the deck, begins, the bridge engineer should have a good understanding of how the existing structure carries its load and the effects of demolition of part of the structure. Most bridges known to be weak have some hidden reserves of strength, but this should not be relied on after removal of part of the deck.

Masonry arches rely on the fill behind the abutments to resist the horizontal thrust; consequently, the removal of fill either on or behind the arch should be undertaken with care. The sequence of demolition of multispan arches is important for similar reasons, the adjacent arch often resisting the horizontal thrust. It might be necessary to prop multi-arches at springing level or, alternatively, plan for demolition of the complete structure.

It can be extremely dangerous to demolish prestressed concrete because of the large inbuilt prestressing forces. With prestressed beams, wherever possible, it is safest to remove the beams to an area where they can be broken up safely. With more complex post-tensioned structures, where this is not possible, careful planning is essential, with reference to the original stressing sequences and details, if available (Lindsell, 1994). A demolition contractor with experience of this type of work should be used. Jack arch structures, much like multi-arch bridges, rely on the adjacent jack arch to resist the horizontal thrust. Removal of half the deck might leave the remaining section unstable. An alternative means of restraining the horizontal forces is with tie-bars. On many structures, tie-bars are continuous across the full width of the deck; cutting them in half can reduce their effectiveness, leaving a potentially unstable deck. Slab-type bridge decks often act as props between abutments, thus ensuring the stability of the abutments. The extent of this propping action should be assessed and suitable props introduced before demolition begins. Partial and sequential removal of a bridge deck, with remaining parts acting as propping, can also be used (Clapham *et al.*, 2015).

Methods of demolition

The methods used to demolish a structure prior to replacement will depend on the time available, the proximity of services, traffic and other buildings, and the amount of the structure being demolished. With the traditional method of breaking up using a hydraulic breaker, control of noise, dust, fumes and vibration is difficult; there is also the likelihood of overbreak onto the parts of the structure remaining. Special care and smaller breakers will be required in the vicinity of services.

Water jetting and hydrodemolition are suitable for use on small areas but unlikely to be economic for use on complete decks. There is also the problem of dealing with the large amounts of water and slurry produced by this method, in particular over rivers.

Explosives are quick, once the initial setting up has been completed, and have been the best option for the rapid removal of bridges over railways and motorways, where possession times are short. Placing, and identifying optimum locations for, controlled explosives is a very specialist operation if success is to be guaranteed at the first attempt. Protection will be required for the railway or road onto which the bridge is dropped.

A number of techniques are available when a structure needs to be cut into manageable portions. Diamond saw cutting, with either a disc or a wire, can be used to allow parts of the bridge deck to be lifted out. This method was used for the reconstruction of Moss Canal Bridge in Rochdale, in the UK (Clapham *et al.*, 2015), where removal and replacement of the existing concrete bridge was undertaken in sequence to allow adequate propping and stability of the abutments at each stage.

On a small number of occasions, the removal of a complete bridge deck has been undertaken using self-propelled modular transporters; the removal of Ingst Bridge over the M4 in Avonmouth, in the UK, was one example (Anon, 1992).

Selection of superstructures

Selection of the construction type for replacement superstructures is determined by

- how much of the road can be closed
- the length of time available
- the site area adjacent to the bridge being reconstructed
- the overall dimensions of the bridge, in particular, the span
- the cost of disruption compared with the cost of the work
- fixed clearances under the bridge, limiting the available construction depth.

Table 26.10 illustrates some of the factors to be taken into account when selecting the type of superstructure to use.

Aesthetics of the replacement bridge

The appearance of bridges is emotive and subjective, with many different opinions being expressed; nevertheless, a number of books have been published with guidelines on appearance (Highways Agency, 1998a). In designing a replacement, the engineer needs to be aware of the importance of the structural form and how it fits in its setting.

Bridge decks can be replaced by copies of the original, but this will not always be possible or desirable. Higher loads must be carried, requiring deeper or stronger sections. Such materials as cast iron cannot be economically copied in modern materials, while arches might be too labour-intensive to reproduce quickly and economically. It might be that the original bridge was inelegant or badly proportioned, in which case, the opportunity should be taken to improve on the original.

When trying to match a replacement deck and parapet with

existing wing walls in brick, remember that metric bricks are a recent introduction. Imperial bricks, or even a local non-standard size, will have been used. Special bricks are still available for copings and string courses and can be made to order.

When replacing simply supported multispan decks, the decks should be made continuous whenever possible, with the advantages of reduced construction depth and fewer joints to leak, improving the water management and structural durability. Similarly, making the deck integral with the substructure will reduce future maintenance.

Wherever possible, bridges with interesting original features should be strengthened and not replaced, retaining as many of the features as possible. However, in many cases, it is not possible to retain, for example, an original wrought iron lattice truss or cast iron beam, because it does not have the structural capacity.

New materials are available for the bridge engineer to use; consider FRP cladding to improve the appearance of fascias. Soffit claddings and bridge enclosures can also hide and protect a steel beam deck (HE, 2019b).

Replacement of road bridges within an extended closure

If a bridge must be replaced, undertaking this in one operation within an extended closure is often the cheapest option, based on construction costs alone. Complete closure of a major road for an extended period would accrue considerable delay and disruption costs and would not normally be acceptable or economic. But where traffic is light and alternative routes short, this method allows the contractor a site free from traffic together with easier construction methods. Health and safety requirements might dictate full closure for bridges on narrow roads when there is not the width available to keep a lane open to traffic while working on the other lane. Whenever road

Structural type	Disadvantages	Advantages
1. Steel beam with in situ concrete deck	Speed of construction of deck; curing of concrete; waterproofing	Easily craned into place; suitable for roll-in; light dead weight allows original substructure to be used
2. Steel beam with precast concrete deck	Difficult to obtain full composite action between deck and beam	As (1); bolt together sections allow rapid construction
3. Steel or FRP beam with FRP deck	As (2); initial cost; sections need to be fabricated under controlled conditions	As (1); lightweight easily craned units provide a safe working platform; durable deck easily maintained
4. Precast concrete beams or arch units	Normally require an in situ concrete deck, requiring time for curing and waterproofing	Minimum disruption to road, rail or water under bridge; can be installed in a single possession; if used with precast deck, can offer a fast construction method
5. In situ concrete	Slow construction period; extensive falsework required	
6. Precast concrete box	Units for large spans or clearances can be heavy	Suitable when substructure requires replacement on smaller structures; easily adaptable for jacking; rapid installation
7. Corrugated steel	Time-consuming backfilling procedure; overwide excavation required to ensure integrity of the backfill	Lightweight sections easily craned and assembled
8. Masonry arch	Extensive falsework required; slow construction method	Aesthetically pleasing

Table 26.10 Summary of deck types, with advantages and disadvantages

closures are used, speed of construction is important to minimise delays and disruption.

Complete closure is rarely acceptable; where access is still needed for cyclists and pedestrians, this can be provided by a small scaffold footbridge alongside the works.

For some types of bridge, such as through-girders, replacement of the whole structure may be the only option available, either within an extended closure or short-term closure.

The design of a replacement bridge to be constructed during an extended closure is similar to that of new construction. The whole of the site is available to the contractor. The deck can be constructed as one, without the need to ensure stability of a partly constructed deck under traffic.

One major difference from new construction is that of working around services and utilities. Support gantries will be required to support services that have not been diverted; telecommunications ducts and the like can be supported by relatively simple scaffold structures. However, gas and water mains will need more substantial support gantries for spans longer than about 5 m. Water mains also often have substantial thrust blocks immediately behind the abutments, further limiting access. On older concrete bridges, service ducts are often built into the deck and, in many cases, will need to remain at the same line and level, limiting the type of deck to in situ concrete in the area of the ducts.

Replacement of road bridges with short-term closures

When the high costs of delay and disruption make extended closures uneconomic, short-term closures take advantage of the lower traffic flows during the night or at weekends to reduce disruption. This type of contract needs thorough planning to maximise the use of often short closures, typically 8 h for an overnight closure, up to 48 h for a weekend. Good public relations, with plenty of advance notice for closures, are a prerequisite for minimising disruption and maximising the amount of useful work. Such contracts are probably best undertaken in the summer months, taking advantage of the warmer and shorter nights.

The choice of deck is limited, because of the need to ensure that the carriageway is reinstated for use by the next morning. The choice has to be made between lifting or sliding in whole deck sections or lifting in smaller sections and stitching the sections together in situ.

The development of large prefabricated lightweight FRP decks provides another method for quick replacement of an entire bridge using small cranes (including craneage from the rail) in a short timeframe, as used at the Standen Hey highway bridge (Dawson and Farmer, 2009) and two footbridges (Canning and Halpin, 2019) in Lancashire, in the UK.

Time must be allowed for demolishing and removing the existing deck. Materials should be chosen for speed of construction; bolted joints where possible, rapid-setting concrete, and so on.

Minimising the weight of replacement deck sections allows for easier craneage or the replacement of larger sections of deck. New materials currently under development that give significant weight advantages decks are aluminium and FRP decking systems.

Replacement of road bridges in stages, keeping the road open

To minimise delay and disruption, the road should remain fully open. A temporary bridge can be constructed either alongside, if land is available, or over the structure being replaced. Adequate working space must be provided under a temporary bridge crossing over and this will also preclude the use of cranes, again limiting the structure type for the replacement deck to smaller units. A temporary bridge will cause some disruption, owing to the substandard alignment. There will also be disruption during the installation of the temporary bridge.

Replacement in two or more parts allows the road to remain open, with the traffic reduced to a single lane under traffic lights or as a single carriageway using a contraflow on dual carriageways. The traffic delay costs are reduced but the difficulties of construction adjacent to live traffic and in parts add to the overall cost of the contract. Continuous temporary barriers must be used to separate the work site from the live traffic lanes, to protect the workforce and road user.

The existing structure will often dictate the location of the join, in particular on structures constructed with discrete beams and longitudinal jack arches. Similarly, the new construction should be detailed so that reinforcement can be connected at the joint using couplers or that deck joints occur at the correct locations. Each part must be capable of taking full loading and should be designed so that the join between sections can be made under traffic. This might necessitate propping under the newly constructed section.

The choice of deck construction is only limited by the space available for handling new sections of deck, considering the physical restraints of working adjacent to live traffic and around services. Steel beam and concrete slabs, prestressed precast concrete beam in-fill decks and in situ reinforced concrete have all been used in Oxfordshire, in the UK.

Railway bridges

For railway bridges (or bridges over the railway), the need to work within very short rail possessions has led to more innovative solutions being developed.

- Roll- or slide-in decks can be constructed alongside the bridge to be replaced and slid in during a possession. Extensive temporary works are normally required, adding to the cost of this method.
- The installation of complete deck sections is practical but requires large cranes, remembering that crane costs increase dramatically with size. Special attention is required to the location of the crane relative to the structure and the offloading position to minimise the reach and hence crane capacity.

- It is worth considering incorporating completed parapets with the deck sections to provide ready-made protection over the railway.

Selection of substructures

In many situations, the substructure may be adequate to accommodate the replacement deck. In assessing its suitability for reuse, the following considerations should be made.

- Is the existing substructure in good condition? Abutments and piers might show stability problems, such as settlement or slippage.

Structural problems, such as cracking, poor brickwork or deteriorating concrete, might also be apparent. Damage to reinforced concrete caused by de-icing salts can be a problem under leaking deck joints.

- If the self-weight of the replacement deck is greater than the original deck, the imposed load on the substructure will clearly increase. Similarly, increases in vehicle loading will increase the loads imposed. In this situation, the substructure needs to be assessed for its suitability to take the increased loading. The use of lightweight bridge deck materials in particular, such as FRP and aluminium, can overcome this challenge and avoid the need for geotechnical investigations or substructure works, both of which are often expensive.

The design and assessment requirements for collision forces on piers and abutments have increased dramatically in recent years. The pier should be assessed under collision loads.

Slab-type bridge decks often act as props between abutments, thus ensuring the stability of the abutments. The extent of this propping action should be assessed and replacement decks designed to accommodate it.

It is unlikely that the original bridge deck was designed to accommodate the current code requirements for longitudinal forces imposed by braking and skidding. The design of the bearings will need to allow these forces to be taken into the substructure. Alternatively, these forces can be taken off the structure into the fill behind the abutment by continuing the deck beyond the substructure and anchoring the deck.

In the majority of situations, it is economic to retain the original abutments and piers, repairing as required to accommodate the new deck. Suitable methods for restoring the stability of abutments include the use of ground anchors and mini-piles.

When the replacement bridge deck is to be craned in, the substructure will need to be able to support the load imposed by the crane or the crane must be moved back from the abutment and the size increased accordingly.

Where the substructure needs to be replaced but deep excavations are not desirable, owing to the proximity of traffic, the use of a contiguous bored pile or sheet pile wall should be considered.

Types of contract

Design and build contracts are commonly used, capitalising on the contractors' knowledge at an early stage in the design process and reducing the likelihood of problems during the construction phase. This teamwork or partnership approach allows for better planning of traffic management and programming of rail closures, using timescales agreed by the contractor and client. It also allows more innovative methods to be used, sharing the risk between the contractor and client.

Public relations and planning

The key difference in new and replacement works is in the level of consultation and planning required. It is important that, for key structures on the network, consultation with the network owners, public utilities and the users takes place at an early stage, sometimes years ahead of the work. This type of work cannot be considered in isolation from the rest of a network and should be part of an overall network management plan. It may be possible to make significant savings by combining contracts; sensible planning of other work on diversion routes can also reduce delays. To close the road, a legal order is required; this will take about 3 months to process because of the large amount of consultation that must occur between the various interested bodies. The Traffic Management Act 2004 now places a duty on highway authorities to coordinate their work properly with other activities using the highway.

Operational needs of the railways and waterways will often dictate that firm dates are set months in advance, with severe cost penalties for late changes.

Nearer the time of the works, the media should be used to publicise the closure and give alternative means of travelling. Clear information boards should be displayed at and before the site, giving clear information on diversions.

Consider the following means of getting the information to the people that will be affected by the disruption:

- radio announcements in traffic bulletins, but make sure that they are accurate
- setting up a website on the internet and the use of social media to give details of the project, road closures, and so on
- press releases to the local papers and advertisements of closures
- automatically generated text messages sent to subscribers' mobile phones
- consultation with the Road Haulage Association, bus companies, and so on
- public meetings
- liaison with local authorities at county, district and parish levels

- leaflets as a mail drop or available at libraries, garages, shops, and so on
- display boards on the roadside, well in advance of the work.

Good publicity can assist in the smooth running of a project. Road users will appreciate being told the reasons for the disruption and having the opportunity to avoid the area where possible. The detailed planning required to make accurate forecasts of disruption can also result in minimised delays.

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