Structural Analysis and Design of Tall Buildings

Steel and Composite Construction

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Ph.D., P.E., S.E.



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This book is dedicated to my wife

Saroja

my dearest and best friend, whose considerable expertise and interest in all aspects of life has influenced my entire existence including my book-writing avocation.

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Foreword

Dr. Taranath has produced a serious state-of-the art book on tall steel and composite structures. As with his phenomenally successful text *Reinforced Concrete Design of Tall Buildings*, this new book should be titled "all you ever wanted to know about steel structures including composite." The good doctor does not cheat the reader by taking shortcuts. His text is truly a magnificent effort by a highly intelligent engineer with a purpose of teaching all he has accumulated in his illustrious career of designing and teaching. This book is not a thin publication or a die-fast text; it is truly a labor of love. Each of the chapters in this book clearly identifies the benefits, shortcomings, and range of applicability of alternate high-rise structural systems, along with the way these systems respond to changing functional and architectural demands.

The first two chapters (Chapters 1 and 2) are on lateral load resisting structures. Logically, the next two (Chapters 3 and 4) are on gravity load systems for steel and composite. The next five chapters (Chapters 5 through 9) are on the evolution of more and more complex concepts such as determination of along-wind and across-wind response, inelastic cyclic behavior of buildings during earthquakes, seismic rehabilitation, and preliminary analysis techniques. The final chapter (Chapter 10) deals with the nonautomatic phase of design—the art of connection design.

One of the underlying philosophies of the author is his humorous and yet serious concern about the brain damage inflicted on engineers who have a slavish adherence to computers. His writing attempts to create a balance between what we should know intuitively and the technical results perfected by the computer.

Structural steel is a complex building material which is the product of an advanced techno society. The infrastructure to produce steel and fabricate it is exceedingly complex. To produce it, you need iron ore, limestone, blast furnaces, pig iron, and open-hearth furnaces.

Steel in structures was initially utilized in America. Steel was responsible for the transition to high-rise structures with elegant members. The advent of the elevator made it possible to go higher and faster, creating the modern skyscraper. To this day, a valued mentor of the author, Dr. Fazlur Kahn, holds the record of the tallest steel structure in the world: the Sears Tower (now the Willis Tower), in Chicago, Illinois.

The steel structures of the Empire State Building, as well as the Humana Headquarters Building, shroud the steel in stone; nonetheless, the interior columns are space-saving and thin.

The first composite structure, believe it or not, was cast iron with masonry backup for stability. This cast iron façade became decorative art and was related to the robust cast iron columns on the interior. The use of composite structures today includes using steel deck composite with studded beams as well as the filling in or concrete encasement around steel members. The tallest composite structure in the world is the Taipei 101.

One of the challenges of using steel is the relative loss in strength with temperature increase. This challenge has been overcome with the modern use of spray or troweled fireproofing, as well as tnemec paints which expand in fire and protect the members.

Why steel? One has just to see the lightness of the Eiffel Tower filigree, suspension bridges, and the exquisite slenderness of modern curtain wall façades to appreciate a material which produces such lightness and grace.

Our field has had a generational crisis of sorts, between those who champion the elegant and intuitive solutions and those who solely rely on the lightning-speed output of computers. However, in order to conceive a structure and give it meaningful proportions, one must follow both the intuitive and the mathematical paths. Only then can we, structural engineers who are trained to be thrifty, fulfill our cherished goal of designing for "the most with the least."

That said, Taranath's abiding engagement and contribution to the structural engineering community has been to develop structural systems for tall buildings that are at once economical and elegant. Take, for example, the belt and outrigger system that has become the workhorse of lateral bracing for tall and ultra-tall buildings. Almost all tall buildings built around the world in the past two decades have used this system in one form or another. In 1974, while employed with the Chicago Office of Skidmore, Owings & Merrill, SOM, Taranath wrote a historic paper that offered for the first time an insight into the behavior of outrigger-braced tall buildings. Using a classic approach that could be literally performed on the back of an envelope, he provided general guidance in locating single and multiple outriggers in tall building structures. Since then, his recommendations have been used in the bracing of tall and super-tall buildings around the world.

Dr. Taranath's intuitive techniques presented in this book extend to several other practical aspects unique to the construction and design aspects of tall buildings, such as

- Structural concepts for super-tall buildings using concrete cores, megaframes, supercolumns, and double-outrigger and belt truss systems
- Response spectrum and design charts for determining seismic base shears in buildings located in selected cities in the United States
- Effect of panel zones in determining lateral stiffness of moment frames
- Differential shortenings of tall steel building columns and how to compensate for their effects
- Comparison of structural schemes for tall buildings including impact of column sizes on architectural layouts
- Unit quantity of materials for preliminary cost comparison
- Details for improving seismic safety of buildings

As an intellectual discipline and applied field, structural engineering's contributions to the lives of the individuals and society are indirect. We rarely directly interact with the generations of people who will inhabit the buildings we design, and yet our expertise and dedication are imprinted throughout.

Within this context, Taranath strives to zero in on the most economical system that simultaneously satisfies the owners' cost concerns and the architects' dreams. Additionally, to assist engineers in arriving at preliminary cost comparisons, he has developed graphs and charts that give unit quantities of materials such as structural steel and mild steel reinforcement in tall and ultra-tall steel and composite construction.

In addition to being an accomplished structural engineer, Taranath is also a gifted educator, passionate to share his enthusiasm with engineers entering the profession as well as seasoned specialists. Five of his books have been published thus far, and the sixth is on its way; all attest to his talent as a practitioner who is able to lucidly and engagingly explain the practice. The topic of each book varies, but the overreaching themes of all his publications bridge the gap between the structural intuition we humans have and the results obtained by using rigorous analytical techniques.

Few engineers have had a career both as a practicing engineer and educator as Dr. Taranath. Fewer, if any, have at the same time made such a prodigious contribution in sharing that knowledge with the profession at large, both in the United States and internationally.

It is indeed my distinct privilege and honor to write this foreword. This book is a must for any serious engineer who truly wants to understand the gestalt of steel and composite tall building design.

Vincent J. DeSimone
Chairman
DeSimone Consulting Engineers

ICC Foreword

It was a great honor when Dr. Taranath asked me to contribute the foreword to his latest work, *Structural Analysis and Design of Tall Buildings: Steel and Composite Construction*. His intentional reliance on intuitive concepts rather than a rigorous analytical approach helps to facilitate an understanding of the many complex ideas involved in tall building design. Although most modern structures eventually do become abstract computer models for purposes of analysis, all structures usually start out as simple concepts, sometimes even a rough sketch on a napkin in a coffee shop. As Dr. Taranath suggests in the preface, his approach is intended to bridge the gap between conceptual design and computer analysis. I was both impressed and amused by his comments regarding Chapter 5 on seismic design, "The primary emphasis is on visual and descriptive analysis. The engineering mechanics are kept to a basic level, and the mathematics to a slide rule accuracy. The highlight of this chapter, perhaps, is the presentation of structural dynamics in which differential equations and esoteric jargon are conspicuously and purposely absent." Basic conceptualization and intuition lies at the very foundation of structural engineering and design. Going back to basics and relying on fundamentals often leads to discoveries that can be masked by abstract computer analysis and reams of abstruse calculations. This is reminiscent of the late great Cal Tech physicist and Nobel Laureate Richard Feynman who has been called a "Babylonian Thinker" because he was an empiricist who favored intuitive conceptual thinking, and not a "Greek Thinker"—a rationalist who prefers rigorous, analytical thinking. We all recall the famous demonstration on television when Professor Feynman placed a small O-ring in a glass of ice water before a congressional committee investigating the Shuttle *Challenger* disaster and showed that the O-ring lost its elasticity at cold temperatures. This simple demonstration ultimately led to the conclusion that the failure of an O-ring seal was due to brittleness caused by cold weather prior to the launch. It was old-fashioned intuition that discovered the problem, not reams of abstract equations and computer analysis.

In this modern digital world of high-speed computers, we engineers need to remind ourselves that many of the world's greatest structures, such as the Golden Gate Bridge, were designed during the era of the slide rule. In the "good ole days" engineers only "estimated" the loads, forces, stresses, and strains that act on structures. The limited precision of the slide rule was a constant reminder that we only know the numbers to three significant figures, which kept us from confusing accuracy with precision. In fact, because the slide rule has no decimal point, the user had to determine where the decimal point belonged, based on mental estimation. This mental estimation forced us to keep track of the order of magnitude our numbers. We had to have a cognitive expectation of the answer we were seeking: would it be 5.5, 55, 550, or 5500? The German rocket scientist Wernher von Braun brought two Nestler slide rules with him when he came to the United States after World War II, and throughout his life he never used any other calculating devices. Slide rule accuracy was sufficient for this famous rocket scientist to *estimate* a host of rocket design parameters. In today's modern era of high-speed computers, it is easy to express numbers with great (misleading) precision, and with modern software such as spreadsheets, PowerPoint presentations, and laser printers, we can produce a dazzling array of beautifully formatted numbers, precise to 12 significant figures and yet be completely wrong!

I am sure the reader will agree that getting back to basics using intuition and conceptualization is the best way to present this subject matter, and Dr. Taranath has done an outstanding job of accomplishing that goal in this comprehensive tall building design guide. He has a perspicuous writing style with a flair for lucidity rather than obscurity, and on occasion even highlights it with some engineering humor. Saying that earthquake engineering is to engineering as psychiatry is to medicine clearly elucidated the point for me. Or saying that ductility is like money—you can never

have too much of it—is a perfect way to emphasize that seismic design is all about ductility, and bigger (stronger and stiffer) is not always better.

I am confident that all kinds of engineers, from students to young graduates, from seasoned practitioners to academics, will appreciate Dr. Taranath's conceptual approach to the subject with no need for rigor. The rigor comes later, after the conceptual ideas have coalesced. After all, one man's rigor is quite possibly another man's rigor mortis.

John R. Henry

Preface

Throughout my career as a practicing engineer, I have seen the ways that the field has shifted and transformed. Many changes certainly move us as practitioners toward greater accuracy, detail, and accountability. Other changes, in my view, seem to diminish our ability to identify key elements in structural design. The last 20 years, in particular, have witnessed tremendous evolution in how structural engineers work in design offices. Software navigating skills have come to the forefront of design concerns, and in so doing, have relegated the very art of structural conceptualization to the backseat. Engineering, of course, is the marriage of artistic and intuitive designs with mathematical accuracy and detail, and I see the latter overshadowing the former in the ways we teach the new generations. Training in the art of conceptualization is not offered in design offices except perhaps for teaching navigating skills through commercial software and in-house spreadsheets. Such training is deemed to be of low value to business balance sheets, yet would encourage us to preserve the conceptual knowledge that seems to belong to an older generation of practitioners.

In a world of high expectations, we seem to place less and less emphasis on learning the fundamentals of conceptual thinking. If we retained these skills as a profession, we engineers could be more adept at identifying what is critical for capturing the essential behavior of the structural system instead of addressing every component of design independently. Computer analysis, then, works to solidify and extend the creative idea or concept that might have started out as a sketch on the proverbial back of the envelope. Our unique gift as engineers is our critical thinking, and we risk shortchanging ourselves and our field, if we remain convinced that the output of voluminous calculations of every structural member is proof of good design.

Mine is not the only voice airing these concerns. A plethora of published journal articles address this ever-increasing gap between the conceptual approach and the computer analysis justified typically by endless tabulation of demand–capacity ratios for each and every structural member. Certainly, there is no doubt that software navigating skills are critical and necessary. My sense, however, is such skills are more useful if built upon solid foundation of engineering principles and conceptual knowledge. As these thoughts occur to me in my day-to-day engineering practice, and more so as I prepare this manuscript, I have set for myself a challenge to bridge the gap between these two approaches: the software operating skills and conceptual design. The very magnitude of this challenge begs for a communal effort on a national scale, and the work presented in this book is but a modest attempt by a single author to address these concerns.

Using conceptual thinking and basic strength of material concepts as foundations, the book attempts to show how to use imperfect information to estimate the answer to much larger and complex design problems. To do so requires a certain intuitive feel for numbers as well as an appreciation of the fact that the "right answer" in this context is only of an order of magnitude of a more precise computer solution, but good enough to put us on the right track. The whole idea is to break seemingly intractable problems down to more manageable pieces that can be quickly approximated. Thus, I attempt to base the entire text on that wonderful ability of intuition we humans have developed in visualizing and realizing economical structural systems. The emphasis in this book is on steel and composite building systems.

Structural steel, as we know today, has been with us for well over a hundred years. It was in the year 1894 that the first specification for structural steel was published, and an examination of test results of that era suggests that the properties of this early steel were not very different from the A36 steel used in the 1950s and 1960s. The first design specifications for steel buildings published by the American Institute of Steel Construction (AISC) in the 1920s firmly established steel as a building material, and ever since its growth has been phenomenal in the construction of buildings and bridges.

At first glance, composite construction may appear to be a new emerging technology, but in reality it has been with us also for over a hundred years. However, it is only recently that its use has been officially formalized by the AISC 341-05/10. Now we can, with equal assurance, design composite buildings in areas of high seismic risk.

The behavior and design of lateral load–resisting systems of structural steel and composite buildings is the subject of Chapters 1 and 2. Traditional as well as relatively recent bracing systems are discussed, including outrigger and belt truss systems that have become the bracing workhorse of super- and ultra-tall buildings. Also included are steel and composite concentric and eccentric braced frames, composite moment frames, and composite shear walls.

Chapters 3 and 4 are dedicated to gravity design of steel and composite floors and columns. In addition to traditional beam-and-girder framing systems, novel floor framing systems such as haunch and stub girders are also discussed.

In Chapter 5, we discuss methods of determining design wind loads using the provisions of ASCE 7-05 and 7-10. Wind-tunnel procedures are discussed, including analytical methods for determining along-wind and across-wind response.

The behavior of buildings and their components subject to inelastic cyclic deformation during large earthquakes is discussed in Chapter 6. The first part of this chapter explains the complex field of structural dynamics in a simple manner so as to make it comprehensible to anyone concerned with the seismic design of buildings. The primary emphasis is on visual and descriptive analysis. The engineering mechanics are kept to a basic level, and the mathematics to a slide rule accuracy. Design requirements of ASCE 7-05 and 7-10 that implicitly provide for acceptable performance beyond elastic range are discussed using equivalent static, dynamic, and time-history procedures. The highlight of this chapter, perhaps, is the presentation of structural dynamics in which differential equations and esoteric jargon are conspicuously and purposefully absent.

Chapter 7 focuses on the anatomy of seismic provisions for structural steel and composite buildings (ANSI/AISC 341-05 and 10). Also presented in this chapter are the designs of special moment frames, braced frames, shear walls, buckling-restrained brace frames, and special plate shear walls.

Chapter 8 is devoted to the structural rehabilitation of seismically vulnerable steel buildings. Design differences between a code-based approach and the concept of ductility trade-off for strength are discussed, including seismic deficiencies and common upgrade methods. The recently published standard, Seismic Rehabilitation of Buildings, ASCE/SEI 41-06, forms the basis of this chapter.

Chapter 9 deals with special topics and starts with a description of tall-building structural systems using case studies that range from the run-of-the-mill bracing techniques to systems that are more appropriate for ultra-tall buildings. I concentrate on actually built or proposed systems using plans, elevations, and three-dimensional schematics. Preliminary analysis techniques using examples are discussed as are graphical approaches for determining wind and seismic loads. The chapter continues with a discussion of human tolerance to wind-induced dynamic motions of tall buildings. Also explained in this chapter are the U.S. and International design guidelines for limiting building motion perception and the effect of structural damping in determining the serviceability limit state. Next, we consider the fundamental aspects of performance-based design (PBD), which has become the cutting-edge technology in building design. Owners like PBD buildings because they cost less, architects love it because it offers more design freedom, and engineers—being thrifty—go for it because it can result in higher-quality structures with the least amount of materials. The chapter concludes with a discussion of analytical methods for determining axial shortening of tall-building columns. Also presented are graphical aids for estimating unit-quantity of structural steel for purposes of conceptual estimates.

The final chapter, Chapter 10, deals with the nonquantifiable, nonautomatic phase of design—what engineers call their art, the art of connection design. Many of the special requirements for joints and connection materials as regulated by the AISC 360-05, AISC 358-05 and AISC 341-05/10, are discussed along with suggested typical details.

This book integrates the design aspects of steel and composite buildings within a single text. It is my hope that this book will serve as a comprehensive design guide and reference for practicing engineers and educators, and, more importantly, as a welcome mat for recent graduates entering the structural engineering profession by assuring them that they have discovered an exciting world of challenges and opportunities.

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Author



Dr. Bungale S. Taranath, PhD, PE, SE, is a corporate consultant to DeSimone Consulting Engineers, a consulting firm, with offices in New York, Miami, San Francisco, New Haven, Las Vegas, Hong Kong, and Abu Dhabi. He has extensive experience in the design of concrete, steel, and composite tall buildings and has served as principal-in-charge for many notable high-rise buildings. He has held positions as a senior project engineer in Chicago, Illinois, and as vice president and principal-in-charge with two consulting firms in Houston, Texas. He has also served as a senior project manager with a consulting firm in Los Angeles, California. Dr. Taranath is a member of the American Society of Civil Engineers and the Concrete Institute, and a registered structural and professional engineer in several states. He has conducted research into the behavior of tall buildings and shear wall structures and is the author of a number of published papers on torsion analysis and multistory construction projects. He has previously published four books: *Structural Analysis and Design of Tall Buildings*; *Steel, Concrete, and Composite Design of Tall Buildings*; *Wind and Earthquake Resistant Buildings: Structural Analysis and Design*; and *Reinforced Concrete Design of Tall Buildings.* Three of his books were translated into Chinese and Korean and are widely referenced throughout Asia. Dr. Taranath has conducted seminars on tall-building design in the United States, China, Hong Kong, Singapore, Mexico, India, and England. He was awarded a bronze medal in recognition of a paper presented in London, when he was a fellow of the Institution of Structural Engineers, London, England. Taranath's passion for tall buildings has never waned. Today, his greatest joy is sharing that enthusiasm with owners, architects, and fellow structural engineers to develop imaginative solutions for seemingly impossible structures.

1 Lateral Load Resisting Systems for Steel Buildings

PREVIEW

Structural systems for tall buildings have undergone a dramatic evolution throughout the previous two decades and into the 2010s. Developments in structural form have historically been realized as a response to emerging architectural trends in high-rise-building design. In the 1980s, international style and modernist high-rise designs, characterized by prismatic, repetitive vertical geometries and flat-topped roofs, were predominant. The development of the original tubular systems for tall buildings was indeed predicated upon an overall building form of constant or smoothly varying profile. The rigid discipline of the exterior tower form has since been replaced in many cases by the highly articulated vertical modulations of the building envelope characteristic of the present day high-rise expressions. This general discontinuity and erosion of the exterior façade has led to a new generation of tall building structural systems that respond to the more flexible and idiosyncratic requirements of an increasingly varied architectural esthetic. Innovative structural systems involving megaframes, interior super-diagonally braced frames, hybrid steel and high-strength concrete core and outrigger systems, artificially damped structures, and spine wall structures are among the compositions, which represent a step in the development of structural systems for high-rise buildings.

Tall buildings are not a product of the modern industrialized world. Inherent human desire to build skyward is nearly as old as human civilization. The ancient pyramids of Giza in Egypt, the Mayan temples in Tikal, Guatamala, and the Kutab Minar in India are just a few examples eternally bearing witness to this instinct. Skyscrapers in the modern sense began to appear over a century ago; however, it was only after World War II that rapid urbanization and population growth created the need for the construction of tall buildings.

It is only natural that the ordinary observer recognizes the tall building primarily with respect to its exterior architectural enclosure. The overall spatial form as well as the detailing of the cladding systems is crucial in defining the architectural expression and in placing the tower within the overall urban environment. The aim of this chapter, however, is to have a look under the outer covering of the building to reveal the structural skeleton.

As in the past, new achievements in material science, computer-aided design, and construction technology have opened paths toward more sophisticated and elegant structural systems for tall buildings. The structural system chosen for a particular project determines the fundamental properties of the overall building, the behavior under imposed loads, and often may have a dramatic impact on the architectural design. The intent of this chapter is to demonstrate the characteristic features of many outstanding system forms.

The structural systems for high-rise buildings are constantly evolving and at no time can be described as a completed whole. Nevertheless, it is worthwhile to review existing systems while being aware the progress in systems development is on going. The author believes that a narrative of prevalent lateral load resisting systems would be of interest and value to practicing engineers and architects as well as other tall building devotees.

Although the use of steel in structures can be traced back to 1856 when Bessemer's steel-making process was first introduced, its application to tall buildings received stimulus only when the 984 ft (300 m) Eiffel Tower was constructed in 1889. After that, by the turn of the nineteenth century, several tall buildings beginning with the modest 286 ft (87 m) Flatiron Building in 1902 and continuing

with the 1046 ft (319 m) Chrysler Building in 1929 were constructed in Chicago and Manhattan, respectively. The height record was broken by the 1250 ft (381 m) Empire State Building in 1931, and then the now nonexistent twin towers of the World Trade Center buildings at 1350 ft (412 m) in 1972, followed almost immediately by the 1450 ft (442 m) Sears Tower in Chicago in 1974.

The role of steel members in earlier structures was principally to carry gravity loads. Gradually its function was enhanced to include wind and seismic resistance using systems ranging from modest portal frames to innovative systems such as outriggers, megaframes, and interior super-diagonals.

Today there exists a myriad of lateral bracing systems that may be grouped into distinct categories, each with an applicable height range. However, selection of lateral load resisting system typically includes one or a combination of the following systems. Also shown is the approximate number of stories favoring each system.

- Frames with semirigid connections: 10–15 stories
- Rigid frames: not exceeding 25 stories
- Braced frames: 25–30 stories
- Staggered truss: not exceeding 25 stories
- Eccentric bracing: 25–30 stories
- Interacting system of braced and rigid frames: not exceeding 40–50 stories
- Outrigger and belt truss system: 40–50 stories
- Framed tube: 50–60 stories
- Trussed tube: 60–70 stories
- Bundled tube: 80–100 stories
- High efficiency systems for ultra tall buildings: 100–150 stories

1.1 RIGID FRAMES

A rigid frame, also called a moment frame (see Figure 1.1), is a vertical support system that also serves as a lateral load-resisting system. It is constructed with beam and column elements either rigidly or semirigidly joined together with moment-resisting connections (Figures 1.2 and 1.3). These connections render the system indeterminate from an analysis standpoint, creating an interdependence between member forces and member deformation not found in determinate structures.

The rigid frame derives its unique strength to resist lateral loads from the moment interaction between beams and columns. The moment restraint at the ends leads to reduced positive bending moments for beams and reduced effective lengths for columns under gravity load.

The rigid frames were developed in the late 1800s as a viable system for multistory structures of iron and reinforced concrete. Features of the bearing wall concept remained, however, as these early structures generally depended on masonry walls to carry lateral loads. As buildings became taller and lighter, and as engineers developed better understanding of this new method of structural

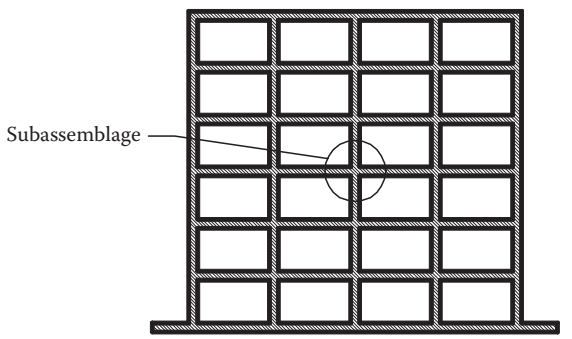


FIGURE 1.1 Multistory rigid frame.

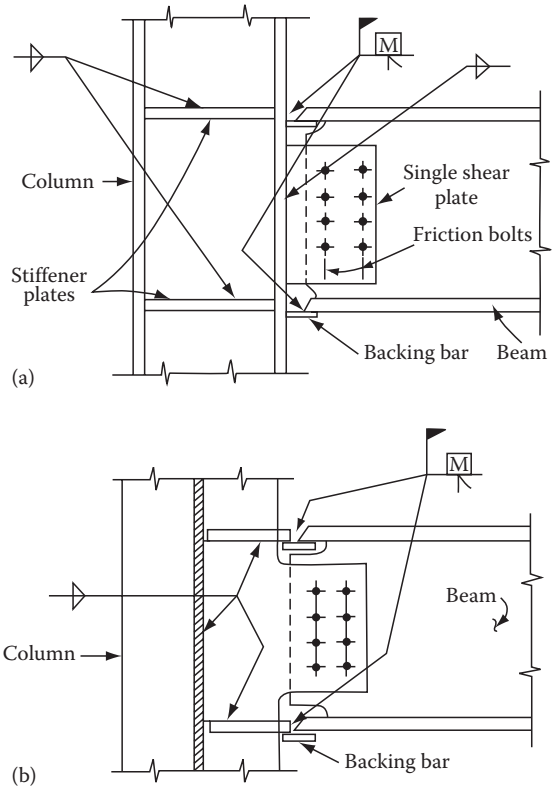


FIGURE 1.2 Beam-to-column rigid connection: (a) beam-to-column flange; (b) beam-to-column web.

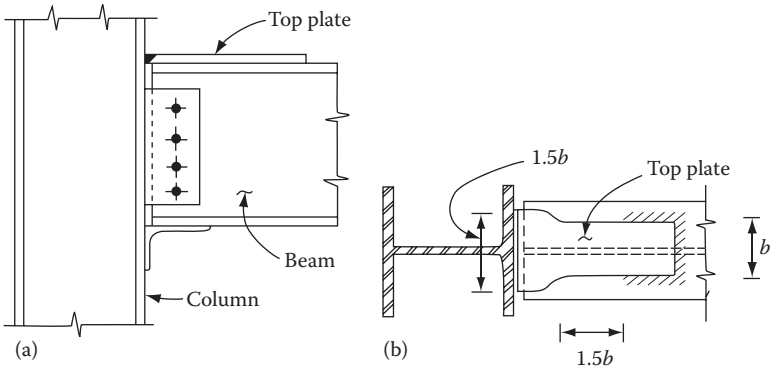


FIGURE 1.3 Beam-to-column semirigid connection: (a) elevation; (b) plan.

assembly, lateral loads were increasingly assumed to be resisted by the frame itself. Engineers such as Hardy Cross (1885–1959) provided the basis for our understanding of rigid frame behavior. It is a tribute to Hardy Cross that so many buildings were built successfully using his moment distribution method.

In the moment distribution method, the fixed-end moments in the framing members are gradually distributed to adjacent members in a number of steps such that the system eventually reaches its natural equilibrium configuration. However, the method was still an approximation but it could be solved to be very close to the actual solution. Today, the "moment distribution" method is no longer used because computers have changed the way engineers evaluate structures. Today's structural analysis software is based on the direct stiffness method.

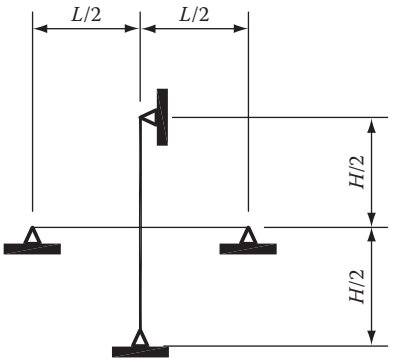


FIGURE 1.4 Beam–column subassemblage. *H*, floor to floor height; *L*, beam span.

The development of improved materials and connection methods (welding, bolting, etc.) paralleled analytical development, and rigid frame construction grew rapidly from the early 1900s to present day. The trend toward lighter construction and more slender elements was clearly established and continues today.

Rigid frame behavior can be studied by isolating a segment of a total frame, and observing the effects of axial load and bending moment on the individual members and their interaction with each other. Such a subassemblage is shown in Figure 1.4.

In the earlier AISC Steel Construction Manuals, the rigid and semirigid connections were referred to as type 1 and type 3 connections while type 2 was the designation used for unrestrained simple connections. Type 1 and Type 3 are now designated fully restrained (FR) and partially restrained (PR) connections.

If the subassemblage is restrained against sway, the effects of column axial load and bending moment applied at the joint are shown in Figure 1.5a. The bending moment is resisted by the four intersecting members in proportion to their relative stiffness (*I*/*L*). The axial load is resisted directly by the beam column but causes an additional column moment due to the $P\delta$ effects.

When lateral deformation of the frame, also referred to as sway, is not restrained, the moment magnification effects due to $P\Delta$ also come into play, where Δ is the sway as shown in Figure 1.5b.

Observe that both the effects of $P\delta$ and $P\Delta$ as discussed later in Chapter 3, must be considered in stability analysis of columns.

The ultimate strength of the beam–column subassemblage resulting in its failure may be due to material failure, joint instability, or frame instability. Material failure occurs when the combined stress from moment and axial load causes rupture of the material. Joint instability may occur when the joint reaches a certain point beyond which further rotation leads to a decreasing ability to carry moment. Frame instability occurs when Δ becomes large for small increments of lateral load resulting in buckling of columns.

The analysis of rigid frames requires calculation of the following loads, forces, and stiffness factors:

- Moments due to gravity loads
- Moments due to lateral loads
- Column axial loads
- Lateral drift (Δ)
- Stability check that includes $P\delta$ and $P\Delta$ effects

Since rigid frames are indeterminate, all of these factors (except perhaps column axial loads) depend on the member properties, which are not known initially. For preliminary analysis, however, we must make assumptions for analysis and later check their accuracy using computer analysis when individual members are sized.

The design loads for wind and seismic effects are applied to structures in accordance with the applicable codes. Next, the structure is analyzed to determine forces and moments for design of the

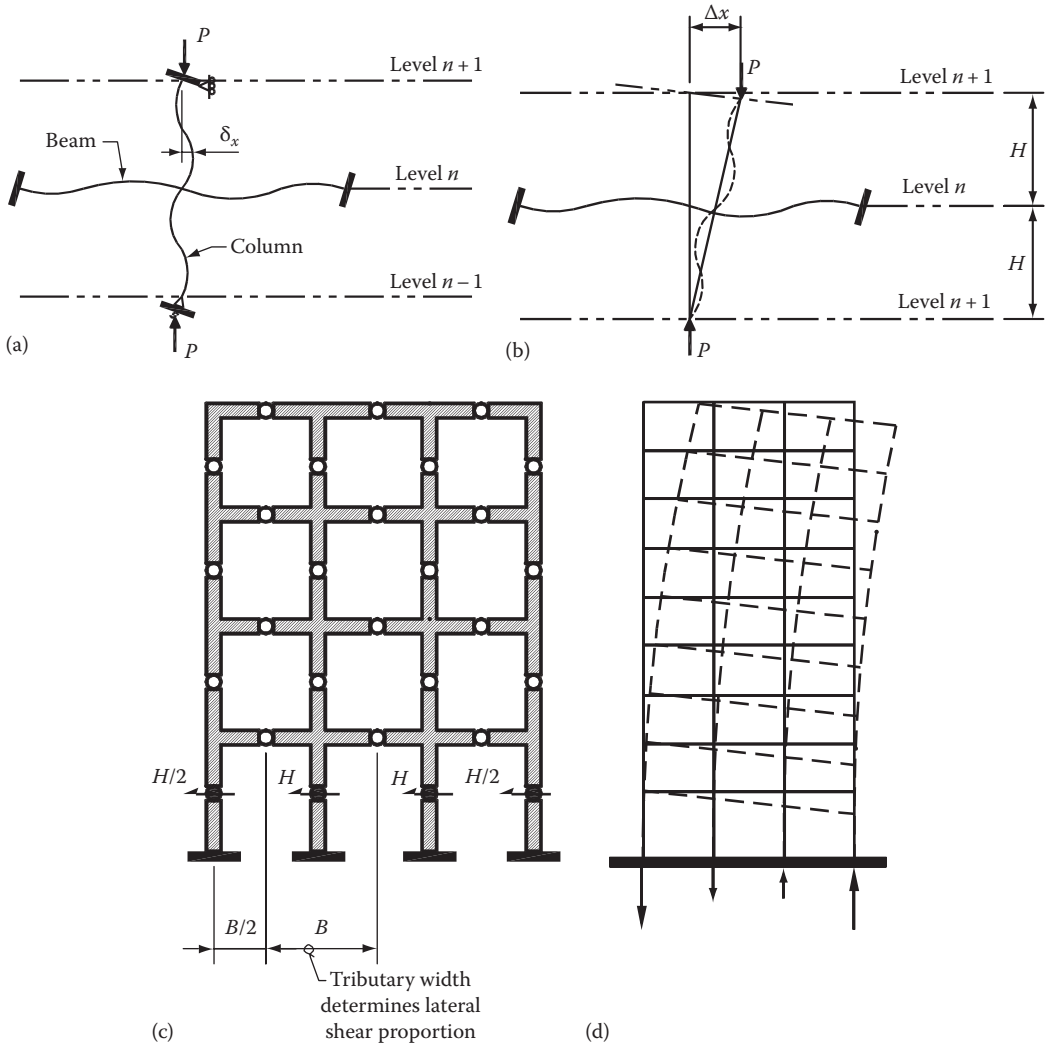


FIGURE 1.5 (a) Beam and column subassemblage restrained against sway, Δ_x. Slightest deviation of column from a straight configuration may have substantial effect on column stresses. When column reaches the critical load, the bending of column becomes a characteristic sudden buckling mode. (b) Beam and column subassemblage unrestrained against sway lateral deflection (Δ_x) causes additional moments in the subassemblage. (c) Portal method of lateral load analysis: Horizontal shear distribution in rigid frames of equal bays. (d) Cantilever method of lateral load analysis: Girders are assumed infinitely rigid, and axial forces in columns are assumed to be proportional to their distance from the frame centroid.

members and connections. Member and connection design proceeds quite normally for wind-load design after these internal forces are determined, but seismic design is also subject to the detailed ductility considerations described later in Chapters 6 through 8. Today, building frames are nearly always designed with the aid of a computer analysis to consider the frame stiffness, deformation, and distribution forces. However, approximate analysis methods are desirable for preliminary analysis and initial member sizing needed prior to development of computer models. Two such models for frames subject to lateral loads are the portal and cantilever methods.

The portal method is used for buildings of intermediate or shorter height. In this method, a bent is treated as if it were composed of a series of two-column rigid frames, or portals. Each portal shares one column with an adjoining portal. Thus, an interior column serves as both the windward

column of one portal and the leeward column of the adjoining portal. Horizontal shear in each story is distributed in equal amounts to interior columns, while each exterior column is assigned half the shear for an interior column, since exterior columns do not share the loads of adjacent portals. If the bays are unequal, shear may be apportioned to each column in proportion to the lengths of the girders it supports. When bays are equal, the axial load in interior columns due to lateral load is zero. Inflection points (point of zero moment) are placed at mid-height of the columns and mid-span beams. See Figure 1.5c. The location of the inflection points may be adjusted for special cases, such as fixed or pinned base columns, or roof beams and top-story columns. On the basis of the preceding assumptions, member forces and bending moments can be determined entirely from the equations of equilibrium.

The cantilever method is used for taller buildings. It is based on the recognition that axial deformation of the columns contributes to much of the lateral deflections of such buildings. In this method, the axial force in a column is assumed proportional to the distance of the column from the centroid of the frame. See Figure 1.5d. Inflection points are assumed to occur at mid-height of columns and mid-span of beams, as with portal method. The internal forces and moments are determined from equations of equilibrium. A complete discussion of the two methods is given in Chapter 9.

1.1.1 Frames with Partially Rigid Connections

Semirigid or partially rigid connections, as the name implies, are those with their rotational characteristics somewhere in-between a fully rigid and a simple connection. These connections offer a known rotational restraint at the beam-ends resulting in significant reduction in mid-span gravity moments. However, they are not sufficiently rigid to prevent the entire rotation between the beam and intersecting column.

1.1.2 Review of Connection Behavior

The 1969 *Specification* (AISC, 1969) classified "types of construction" as Type 1, 2, or 3, based on the nature of the connections of the beams to the columns. Type 1 construction comprised "rigid frames," also referred to as moment-resisting frames that had connections capable of transmitting the moments. Type 2 construction comprised "simple frames" with no appreciable moment transfer between beams and columns. Type 3 construction comprised "semirigid frames." Type 3 construction used PR connections and was allowed in building construction if a predictable and reliable amount of connection flexibility and moment transfer was demonstrable.

The designations have now changed from Type 1, 2, or 3 to the FR and PR. In these designations, the term "restraint" refers to the degree of moment transfer and the associated deformation in the connections. The term "simple framing" refers to structures with "simple connections," that is, connections with negligible moment transfer. In essence, FR is equivalent to Type 1, "simple framing" to Type 2, and PR to Type 3 construction.

Type 2 construction and "simple framing" of the earlier *Specifications* had additional provisions that allowed wind loads to be carried by moment resistance of selected joints of the frame provided that

1. The connections and connected members had capacity to resist the wind moments
2. The girders were adequate to carry the full gravity load as "simple beams"
3. The connections had adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loading

The justification of considering the so-called "wind connections" as both simple (for gravity loads) and moment resisting (for wind loads) is based on the argument that the connections actually have some moment resistance but that the strength is low enough that under wind loads the connections would sustain inelastic deformations. Under repeated wind loads, then, the connection response

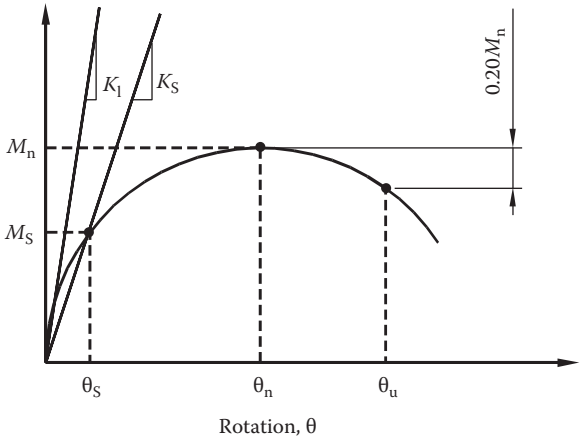


FIGURE 1.6 Moment–rotation M–θ curves for PR connections.

would "shake down" to a condition wherein the moments in the connections under gravity loads would be very small but the elastic resistance of the connections to wind moments would remain the same as the initial resistance. These additional provisions for Type 2 construction have been used successfully for many years. However, their use at the present time (2010) is not prevalent.

1.1.2.1 Connection Classification

In classifying connections, the most important behavioral characteristics can be modeled by a moment–rotation (M–θ) curve as shown in Figure 1.6. Implicit in the moment–rotation curve is the definition of the connection as being a region of the column and beam along with the connecting elements. The connection response is defined this way because the rotation of the member generally incorporates the contributions of not only the connecting elements, but also the ends of the members being connected and the column panel zone.

Because of the nonlinear behavior even at low-moment–rotation levels, the initial stiffness of the connection K_i, shown in Figure 1.6, does not adequately characterize connection response. Therefore, the secant stiffness K_S is taken as an index property of connection stiffness.

$K_S = M_S/\theta_S$, where M_S and θ_S are the moment and rotation, respectively, at service loads. L and EI are the length and bending rigidity, respectively, of the beam.

If $K_S L/EI = 20$, then it is acceptable to consider the connection to be FR. If $K_S L/EI \leq 2$, then the connection is considered to be simple. Connections with stiffness between these two limits are considered PR. Examples of connection response of FR, PR, and simple connections are shown in Figure 1.7.

1.1.2.2 Connection Strength

The strength of a connection is the maximum moment, M_n, that the connection is capable of carrying, as shown in Figure 1.7. M_n can be determined on the basis of an ultimate limit-state model, or from a test. If the moment–rotation response does not exhibit a peak load, then the strength can be taken as the moment at a rotation of 0.02 rad.

It is also useful to define a lower limit on the strength below which the connection may be treated as a simple connection. Connections that transmit less than 20% of the fully plastic moment of the beam at a rotation of 0.02 rad may be considered to have no flexural strength for design purposes.

1.1.2.3 Connection Ductility

If the connection strength substantially exceeds the fully plastic moment capacity of the beam, then the ductility of the structural system is controlled by the beam in which case the connection

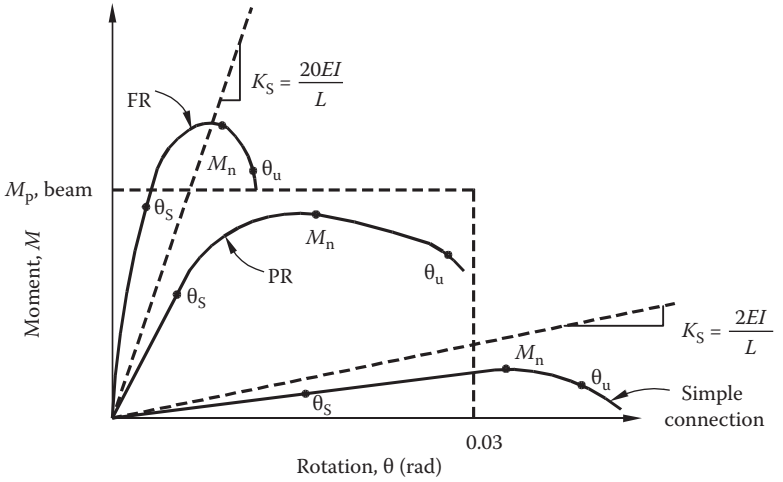


FIGURE 1.7 Classification of moment–rotation response of FR, PR, and simple connections.

can be considered elastic. If the connection strength only marginally exceeds the fully plastic moment capacity of the beam, then the connection may experience substantial inelastic deformation before the beam reaches its full strength. If the beam strength exceeds the connection strength, then deformations can concentrate in the connection. The ductility required of a connection will therefore depend upon the particular application. For example, the ductility requirement for a frame assigned to a lower Seismic Design Category (SDC) will generally be less than that required in a higher SDC.

1.1.2.4 Structural Analysis and Design

When a connection is classified as PR, the relevant response characteristics of the connection must be included in the analysis of the structure to determine the member and connection forces, displacements, and the frame stability. Therefore, PR construction requires that the moment–rotation characteristics be incorporated in the analysis and member design.

Typical moment–rotation curves for many PR connections are available from several databases. However, tabulated moment–rotation curves should not be extrapolated to sizes or conditions beyond those used to develop the database since other failure modes may control. When the connections to be modeled do not fall within the range of the databases, it may be possible, however impractical it may be, to determine the response characteristics from tests, simple component modeling, or finite element studies.

Three basic types of construction and associated assumptions are permissible under the AISC specifications, and each governs in a specific manner the size of members and the types and strength of their connections.

Connections in simple frames are typically designed to transfer vertical shear only. They are also designed for axial loads if they transfer chord and drag forces due to seismic loads. Connections in fully rigid frames on the other hand, develop resistance to both shear and bending moment. They are assumed to have sufficient rigidity to hold virtually unchanged the original angles between connecting members. Semirigid connection behavior is intermediate between the simple and rigid condition, its fixity varying anywhere from a low 5% to a high 90% of full-fixity.

Simple and rigid connection behaviors are ideal conditions. Practically, it is necessary to accept something less than ideal, since moment frames perform in the broad range between fully rigid and simple support condition. For example, consider the typical beam-to-column connection consisting of a double-angle web connection shown in Figure 1.8. The angles fastened to the beam web are

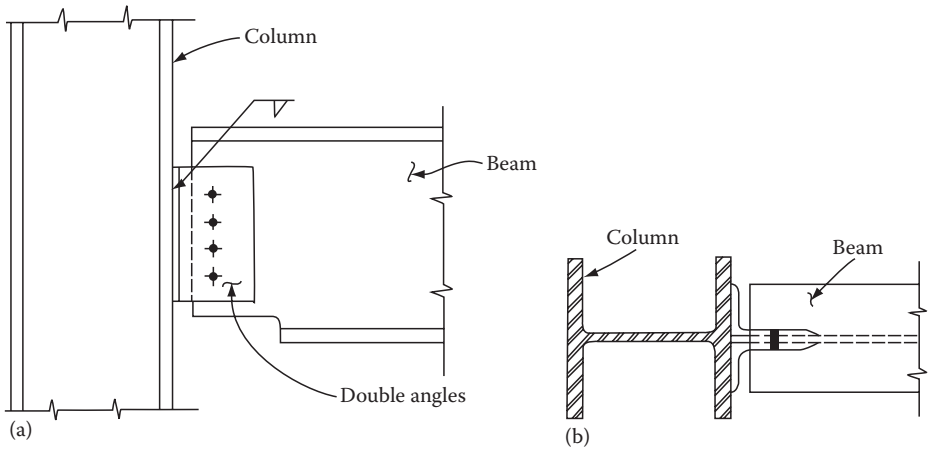


FIGURE 1.8 Beam-to-column field-bolted shear connection. (a) Elevation, (b) plan.

considered flexible and are assumed to carry only shear. Actually, they offer some restraint and thus oppose the rotation at the beam-end. The relationship between the applied moment and rotation is quite complex and can only be determined by tests because the rotation of the connection depends to a large extent on deformation of angles. Therefore, to minimize the rotational restraint, the angles are made as thin as possible in simple beam designs.

Unstiffened seated beam connection shown in Figure 1.9 is another example of a simple connection. The moment–rotation characteristics of a seat angle connection primarily depend on the beam depth, thickness of top angle, diameter of bolts, and thickness of column flanges. The connection is typically stiffer than web-angle connection but still considered a simple flexible connection. By adding top and bottom angles, it is possible to develop a connection that has greater moment resistance than either of the previously described connections.

The top and bottom angles are assumed to carry the moment, and web angles the shear. Although the load distribution may appear to be arbitrary, such a division of function produces adequately proportioned connections. Structural tees used in place of top and bottom angles increase the rotational restraint considerably. The increase occurs because the top tee is loaded in direct tension without significant eccentricity.

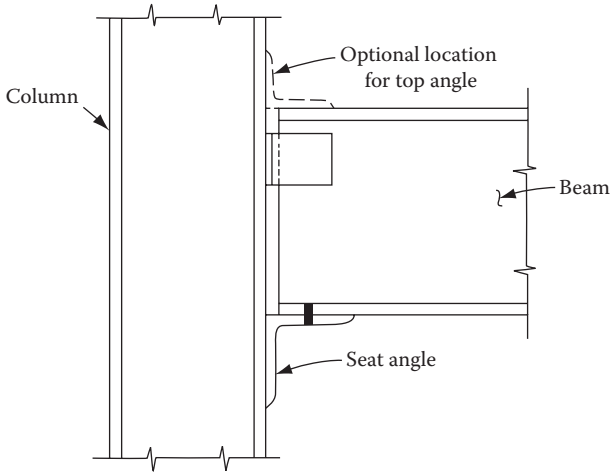


FIGURE 1.9 Unstiffened seated beam connection.

1.1.3 Beam Line Concept

The end moment M and the corresponding rotation θ of a uniformly loaded beam of length L, for a given end restraint, can be evaluated by studying a graph commonly referred to as the *beam line* plot. In this graph, shown in Figure 1.10, the end moment, M, is plotted along the vertical axis while the corresponding rotation, θ, in radians, is plotted along the horizontal axis.

Consider a prismatic beam of length, L, loaded with a uniformly distributed load $W = wL$. The relation between the end moment M and the corresponding rotation θ can be expressed by the following equation:

$$M = -\frac{2EI\theta}{L} - \frac{WL}{12}$$

This is a straight-line relationship that can be plotted by considering:

1. The rotation of a simply supported beam
2. The fixed end moment of a FR beam

Point a on the beam line is the end moment for a Fully Restrained, FR connection. Substituting $\theta = 0$, in the equation above, we get

$$M = -\frac{WL}{12}$$

Point b is the rotation at the end of beam when the beam has zero restraint at the ends. In other words, the beam behaves as a simply supported beam. Substituting $M = 0$ in equation above, we get

$$\theta = -\frac{WL^2}{24EI}$$

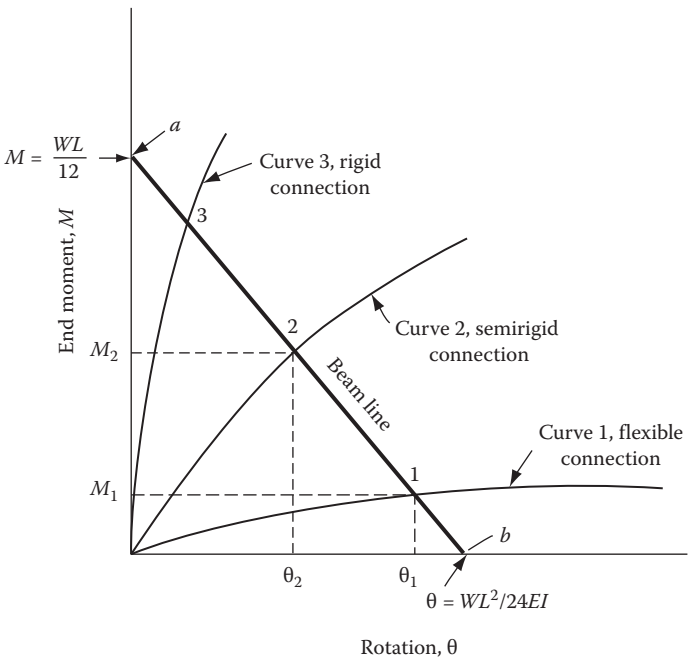


FIGURE 1.10 Beam line concept: Moment–rotation (M–θ) curves.

Line *ab* that connects the points *a* and *b* is the beam line. Curves 1, 2, and 3 are the moment–rotation relationships for three types of connections denoted as flexible, semirigid, and fully rigid. The point at which the beam line intersects the connection curve gives the resulting end moment and the corresponding rotation under the given load. These are shown as points 1, 2, and 3 in Figure 1.3.

The dependence of the beam behavior on the rigidity of the connection can be studied by using this diagram. In developing the M–θ relationship, it is assumed that the behavior of the two end connections is the same and that the beam is subjected to loads placed symmetrically on the beam. The behavior of all three types of connections, namely, the flexible, semirigid, and rigid connections, can be studied by using the beam line diagram. Curve 1 represents a flexible connection which is typical of a double-angle web connection. Under a uniform load W, the beam-ends rotate through an angle θ_1, which is very nearly equal to the rotation θ of a completely unrestrained beam. Corresponding to this rotation, a moment M_1 is generated at the ends signifying that even with the so-called flexible connection, some end moment is generated. Normally, the bending moment developed is about 5%–20% of the fully fixed moment.

Curve 2 represents a semirigid connection such as an end plate connection. A significant moment M_2 corresponding to the rotation θ_2 develops at the beam-ends. The restraint offered by this type of connection can vary anywhere from a low of 20% to a high of 90% of the full fixity. Thus, resulting end moments could be 20%–90% of the moment generated in a fully fixed beam.

Curve 3 represents the moment–rotation characteristics of a rigid connection such as a welded beam-to-column connection shown earlier in Figure 1.2. The beam develops end moments that are about 90%–95% of fully fixed condition, especially when column flange stiffeners are used.

The design of flexible and semirigid connections presents some very intriguing problems because some means of transferring these moments must be provided in these connections.

A concept popular some 30 years ago in the design of semirigid frames was to assume some inelastic but self-limiting deformation of the connection components without overstressing the fasteners. The *self-limiting deformation criteria* are imposed to prevent the use of semirigid connections for cantilever beams. This is because cantilever beam deflections are not limited by the rotation of the connections but may continue to progress, resulting in failure of the connection, and even the beam itself.

1.2 FRAMES WITH FULLY RESTRAINED CONNECTIONS

A frame is considered rigid when its beam-to-column connections have sufficient rigidity to hold virtually unchanged the original angles between the intersecting members. A rigid frame typically comprises of parallel or orthogonally arranged bents consisting of columns and girders with moment resistant joints. Resistance to horizontal loading is provided by the bending resistance of the columns and beams. The rigidity of the joint also assists in resisting gravity loading more efficiently by reducing the positive moments in the center span of beams.

Typical deformation of a moment-resisting frame under lateral loads is shown in Figure 1.11. The lateral deformations result in reverse bending of columns and beams with a point of contraflexure located near the mid-height of the columns and mid-span of the beams. The lateral deformation of a frame, as will be seen shortly, is due partly to frame racking, often called shear-sway component, and partly to column elongation and shortening, often referred to as cantilever or chord drift component. For low-rise buildings, say up to 20 stories tall, the shear-sway component constitutes approximately 60%–70% of the overall lateral deformation while the remainder results from the chord drift component.

Moment-resisting frames have certain advantages in building construction due to their flexibility in architectural planning. They may be placed on the exterior, or the interior of the building with minimal constraint on the architectural planning. The size of columns and beams in a moment-resisting frame, particularly in tall buildings, is invariably controlled by flexural stiffness rather than their strength. This is to limit the building drift under lateral loads.

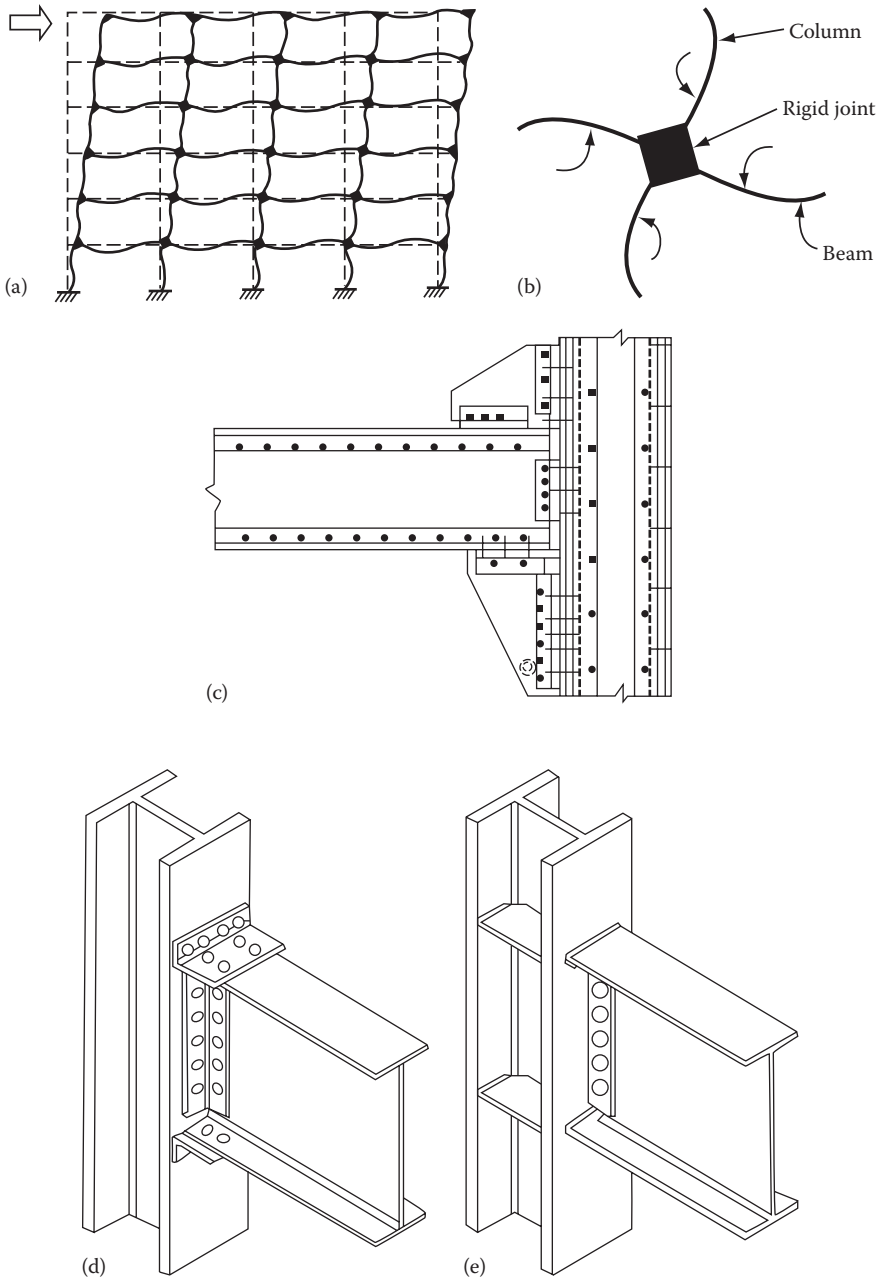


FIGURE 1.11 (a) Response of rigid frame to lateral loads. (b) Flexural deformations of beams and columns. (c) Typical early built-up and rivetted connection. (d) Rivetted, unstiffed seat angle connection. (e) Pre-Northridge moment connection (1970–1994): welded unreinforced flange-bolted web (WUF-B) moment connection.

In a typical building application, beam spans are 30 ft (9 m) to 40 ft (12 m) while story heights are usually between 12 ft (3.65 m) and 14 ft (4.27 m). Therefore, to arrive at an optimum system, the beam moment of inertia needs to be correspondingly larger than that of the column; ideally by a ratio of beam span to story height.

Moment-resisting frames are efficient for buildings up to about 20 stories in height. They are not efficient for taller buildings primarily because lateral loads are resisted through flexural and shear

strengths of beams and columns, and not through axial strengths as in braced frames. Resisting loads through axial deformation of members rather than by flexure almost always results in an efficient structure.

The behavior of connections is quite an important consideration in the design of moment frames because their rotation may increase the lateral sway of a frame significantly. Additionally, the strength and ductility of connections are of major importance, particularly in frames designed to resist seismic loads.

Steel moment–resisting frames with welded connections were regarded up until the Northridge earthquake, as one of the safest for having the required strength, ductility, and reliability. However, the Northridge, magnitude 6.7 earthquake of January 17, 1994, which caused damage to over 200 steel moment frame buildings and the Kobe earthquake of January 17, 1995, which resulted in collapse of a number of buildings of similar construction, have shaken engineers' confidence in its use for seismic design. Almost without exception, the connections that failed were of the type with full-penetration field weld of top and bottom flanges, and a high-strength-bolted shear tab connection. The majority of the damage consisted of fractures of the bottom flange weld between the column and girder flanges. There were also a large number of instances where top flange fractures occurred. Although many factors have contributed to the poor performance it is believed that the basic joint configuration is not conducive to ductile behavior.

1.2.1 Special Moment Frame, Historic Perspective

A seismic moment frame comprises of a rectangular system of rigidly connected columns and beams that resist moment and shear forces developed during earthquake ground shaking. Seismic standards typically consider the system extremely ductile and assign it the highest allowable response modification coefficient. It is one of only few systems permitted in SDCs D, E, and F for buildings exceeding 160 ft in height.

Because earthquake motions can induce multiple inelastic displacement cycles, special proportioning and qualification requirements are essential for robust moment frame performance. Although the steel special moment frame (SMF) is a relatively recent development in building codes, steel frames have been in use for more than 100 years, dating to the earliest use of structural steel in building construction. Framing in these early structures typically utilized "H" shapes built up from plate, "L" and "Z" sections. The connections typically incorporated large stiffened triangular gusset plates, joined to the beams and columns with angles and rivets as shown in Figure 1.11c. The basic construction style remained popular for high-rise construction through the 1930s. By the early 1900s, rolled "H" shape sections began to see increasing use in place of the built-up sections, in particular for lighter framing. Many tall structures including the Empire State Building in New York, for many years the world's tallest structure, are of this construction type.

Following World War II, engineers began to design connections using unstiffened angles or split tees to connect top and bottom beam flanges to columns (see Figure 1.11d). In the 1950s, as welding was introduced into building construction, the angles and split tees were replaced by flange top plates, shop welded to the column flanges, then riveted to the beam flanges. By the 1960s, riveting had become uneconomical and was replaced by high strength bolting. Finally, in the early 1970s, engineers began to use the connection type known today as the welded unreinforced flange-bolted web (WUF-B) (Figure 1.11e). Incorporating field-welded, complete joint penetration groove welds to join beam flanges to columns, and shop-welded, field-bolted shear plates joining beam webs to columns. Almost from their inception as a means of building construction, engineers began to observe that steel moment frames seemed to exhibit superior performance in earthquakes.

As a result of the apparent superior performance of these structures, building codes of the 1960s adopted preferential design criteria for steel moment frames. Under these codes, buildings having complete vertical load-carrying space frames as their lateral force resisting system could be designed for two thirds of the seismic forces specified for braced frames and half the forces specified for

bearing wall structures. Further, these codes required such frames in buildings exceeding 240 ft in height. In the 1960s and 1970s, researchers began to perform cyclic laboratory testing of steel moment frames. It was determined that some control on proportioning and detailing of these structures was necessary to obtain superior inelastic behavior in strong earthquakes. Throughout the 1970s and 1980s, the building codes began to adopt the recommendations of these researchers and required special design, configuration, and detailing of steel moment frames used for seismic resistance in regions of high seismic risk. Frames conforming to these design criteria were first designated as ductile moment–resisting space frames, and then finally in 1988, as special moment-resisting frames. The term "special" was adopted, both because special criteria applied to the design of these structures and also because special, superior behavior was anticipated of them in strong earthquakes.

Initially, the special design criteria were limited to a requirement that connections be capable of developing the strength of the connected members, with the WUF-B connection identified as a deemed-to-comply standard. Later, requirements were introduced to provide for strong-column weak-beam behavior, balance of the shear strength of panel zones with beam flexural capacity, and addition of section compactness and lateral bracing criteria. Building codes of this era required the use of ductile moment–resisting space frames in all structures exceeding 240 ft in height zones of high risk of experiencing strong ground motion. As a result, nearly every tall building constructed in the western United States in this era was of steel moment frame construction. Such structures designed in the 1960s and 1970s tended to employ moment-resisting connections at every beam–column joint, providing high redundancy and alternate load paths for distribution of lateral force resistance. However, by the 1980s engineers had begun to economize their designs and minimize expensive field welding by using fewer bays of moment-resisting framing that employed heavier beams and columns, resulting in less redundant structures with more concentrated lateral force resistance. In extreme cases, some tall structures were provided with only a single bay of moment-resisting framing on each side of the building.

Following the 1994 Northridge earthquake, engineers were surprised to discover that a number of modern special moment-resisting frame structures had experienced unanticipated brittle fracturing of their welded beam–column connections. Similar damage and even collapse of some buildings occurred 1 year later, in the 1995 Kobe, Japan earthquake. Following these discoveries, a consortium of professional associations and researchers, known as the SAC Joint Venture, engaged in a federally funded, multiyear program of research and development to determine the causes of this unanticipated behavior and to develop recommendations for more robust moment frame construction. It was determined the fractures were a result of the basic connection geometry, lack of control of base materials, the use of weld filler metals with inherent low toughness, uncontrolled deposition rates, inadequate quality control and other factors. The resulting research is the basis for the current steel SMF code design requirements.

1.2.1.1 Deflection Characteristics

The lateral deflection components of a rigid frame can be thought of as being caused by two components similar to the deflection components of a prismatic cantilever beam. One component can be likened to the bending deflection and the other to the shear deflection of the cantilever. Normally for prismatic members when the span-to-depth ratio is greater than 10 or so, the bending deflection is by far the more predominant component. Shear deflections contribute a small portion to the overall deflection and, therefore, are generally neglected in calculating deflections. The deflection characteristics of a rigid frame, on the other hand, are just the opposite; the component analogous to the shear deflection particularly for the low-rise buildings, often dominates the deflection picture and may amount to a large percentage of the total deflection. The split may be 70%–30%, the 70% coming from shear while the remaining 30% from the bending component. The bending and the shear components of deflection are often referred to as the cantilever bending and frame racking, each with its own distinct deflection mode.

1.2.2 Cantilever Bending Component

Let us consider the lateral load effects of wind acting on the vertical face of the building. This load causes overall bending of the building with the bending moment reaching the maximum value at the base. The bending moment causes the building to rotate resulting in axial deformation of columns. The corresponding forces are compressive in the leeward columns and tension in the windward columns. Thus, the columns lengthen on the windward face of the building and shorten on the leeward face. The axial deformations of columns causes the building to rotate about the base resulting in lateral deflection, as shown in Figure 1.12. Because of the cumulative rotation up the height, the story drift due to overall bending increases with height, while that due to shear racking tends to remain the same or increase slightly down the height.

1.2.3 Shear Racking Component

This phenomenon is analogous to shear deflection and occurs as a result of bending of individual beams and columns. The accumulated horizontal shear above a given story is resisted by shear in the columns of that story as shown in Figure 1.13. The shear causes the story-height columns to bend in double curvature with points of contraflexure at approximately mid-story-heights. The moments

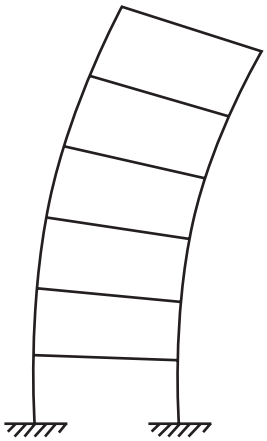


FIGURE 1.12 Cantilever bending of rigid frame.

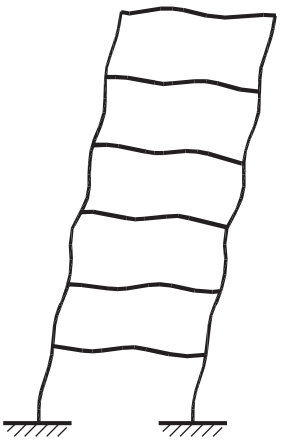


FIGURE 1.13 Shear racking of rigid frame.

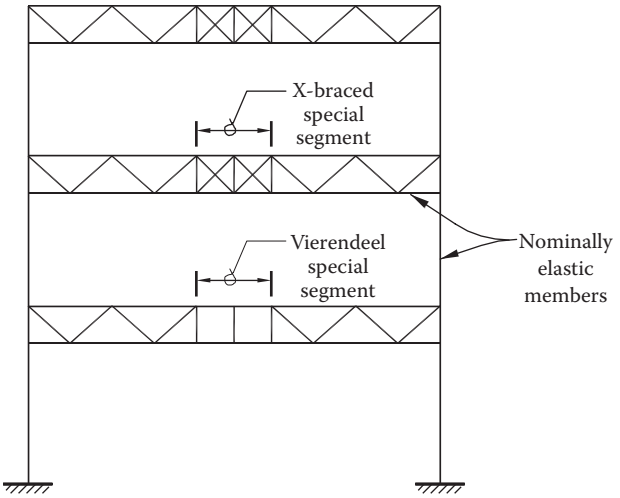


FIGURE 1.14 Special truss moment frame.

applied to a joint from the columns above and below a particular level are resisted by the attached girders at that level, which also bend in double curvature, with points of contraflexure at approximately at their mid-spans.

The double curvature bending deformations of columns and girders results in shear racking of the frame with a consequent increase in the horizontal deflection in each story. The overall deflected shape of a rigid frame due to racking has a shear configuration with concavity upwind, a maximum inclination near the base, and a minimum inclination at the top, as shown in Figure 1.14. This shear mode of deformation accounts for the majority of the total sway and in a typical rigid frame with column spacing at about 35–40 ft (10.6–12.2 m) and a story height of 12–13 ft (3.65–4.0 m), the beam flexure contributes about 50%–65% of the shear racking. The bending of columns, on the other hand, contributes about 10%–20%. This is because, in most moment frames, the ratio of column stiffness to girder stiffness is high resulting in a larger rotation of the girders. So, generally speaking, when it is desired to reduce the deflection of a moment frame, the place to start adding stiffness is to the girders. However, in nontypical frames, such as those that occur in framed tubes with column spacing approaching or even less than the floor-to-floor height, it is necessary to study the relative girder and column stiffness before making adjustments to member properties.

1.2.4 Methods of Analysis

Invariably, for the past two decades, the analysis and design of buildings is performed using computer programs. Hand calculations are rarely undertaken except for very preliminary purposes.

Among the better known approximate methods of analysis, the cantilever and portal methods are perhaps the most popular and considered not too long ago to be sufficiently accurate for use in the final analysis of buildings. The portal method is considered valid for buildings less than 25 stories, while the cantilever method is assumed good for buildings in the 25–40 story range. A description of each method is given in Chapter 9 of this book.

1.2.5 Drift Calculations

Calculation of drift due to lateral loads is a major task in the analysis of tall building frames. Although it is convenient to consider the lateral displacements to be composed of two distinct components, whether or not the cantilever or the racking component dominates the deflection is dependent on factors such as height-to-width ratio of the building and the relative rigidity of the

columns, girders, and their connections. As stated previously, for a very tall or very slender building, it is usually the cantilever bending component that dominates the deflection picture. A simple method for determining approximate deflection of a tall building is to assume that the entire structure behaves as a vertical cantilever in which the axial stress in each column is proportional to its distance from the centroidal axis to the building. This approach assumes that the frame is infinitely stiff with respect to shear deformations and hence underestimates the deflection but will serve as a starting point for preliminary designs.

1.2.6 Truss Moment Frames

Truss moment frames consist of horizontal trusses rigidly connected to columns. The resistance to lateral displacement is by traditional frame action. It should be noted, however, that certain limits are imposed on the span and depth of truss used in buildings assigned to SDCs C or higher. In these buildings, a special segment of the truss is designed to provide a yield mechanism, while the truss elements outside of the special segment are designed and detailed to remain nominally elastic. The horizontal truss elements are designed and constructed with the special segment placed near midspan of the truss in order to mitigate the gravity shear effects. The special segment can be designed and detailed using either X-braced panels or Vierendeel panels, as shown in Figure 1.14. A properly designed and detailed special segment is expected to withstand large inelastic deformations. The energy dissipation is through flexural or axial yielding of web members, depending on the particular type of special segment used.

1.3 CONCENTRIC BRACED FRAMES

Generally speaking, rigid frame systems are not efficient for buildings taller than about 20 stories because the shear racking component of deflection due to bending of columns and girders causes the drift to be too large. A braced frame improves upon the efficiency of a rigid frame by virtually eliminating the bending of columns and girders. This is because by adding web members such as diagonals or chevron braces, the horizontal shear is resisted by the web. The webs carry the lateral shear predominantly by axial forces in the braces thus minimizing bending of beams and columns.

1.3.1 Behavior

In simple terms, braced frames may be considered as vertical trusses resisting lateral loads primarily through the axial stiffness of columns and braces. The columns act as the chords in resisting the overturning moment, with tension in the windward column and compression in the leeward column. The diagonals work as web members resisting the horizontal shear in axial compression or tension. Because the lateral load is reversible, braces are subjected to both compression and tension; consequently, they are most often designed for the more stringent requirements of compression.

When a brace frame is subjected to lateral loads, the resulting axial deformation of the columns tend to cause a flexural deformation of the frame with concavity downwind and maximum slope at the top as shown in Figure 1.15a. On the other hand, the axial deformation of the diagonals tend to cause a “shear” mode deformation with concavity upwind, a maximum slope at the base, and a zero slope at the top (see Figure 1.15b). The resulting deflected shape shown in Figure 1.15c is thus a combination of flexural and shear modes with the flexural deflection most often dominating the deflection scene.

The role of web members in resisting the horizontal shear can be demonstrated by following the load path down the braced bent. Consider the braced frames, shown in Figure 1.16, subjected to an external shear force at the top. In Figure 1.16a, the diagonal in each story is in compression, causing

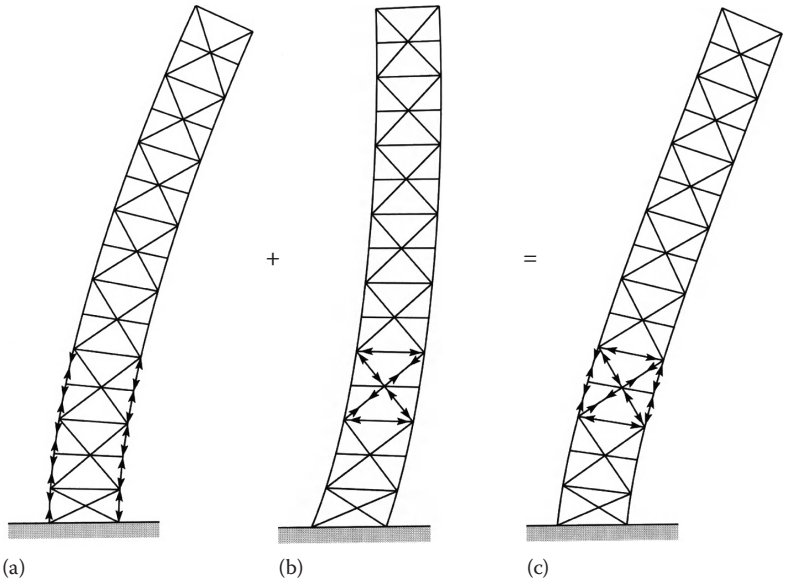


FIGURE 1.15 Braced frame deformation: (a) flexural deformation; (b) shear deformation; (c) combined configuration.

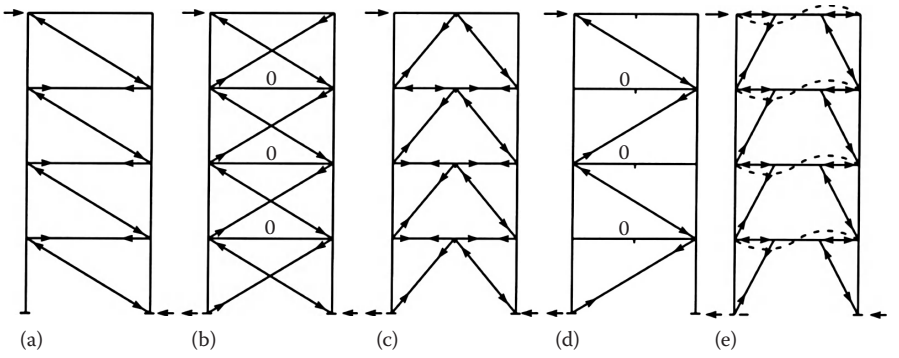


FIGURE 1.16 Load path for horizontal shear through web numbers: (a) single diagonal bracing; (b) X-bracing; (c) chevron bracing; (d) single-diagonal alternate direction bracing; (e) knee bracing.

the beams to be in axial tension; therefore, the shortening of the diagonal and extension of the beams gives rise to the shear deformation of the bent. In Figure 1.16b, the forces in the braces connecting to each beam-end are in equilibrium horizontally with the beam carrying insignificant axial load.

In Figure 1.16c, half of each beam is in compression while the other half is in tension. In Figure 1.16d, the braces are alternately in compression and tension while the beams remain basically unstressed. And finally in Figure 1.16e, the end parts of the beam are in compression and tension with the entire beam subjected to bending in double curvature as shown by the dotted lines. Observe that with a reversal in the direction of horizontal load, all actions and deformations in each member will also be reversed.

The principal function of web members is to resist horizontal shear forces. However, depending upon their configuration, the web members may pick up substantial compressive forces as the columns shorten vertically under gravity loads. Consider, for example, the bracing configuration shown in Figure 1.17. As the columns shown in Figure 1.17a and b shorten, the diagonals are subjected to compression forces because the beams at each end of the braces are effective in resisting the

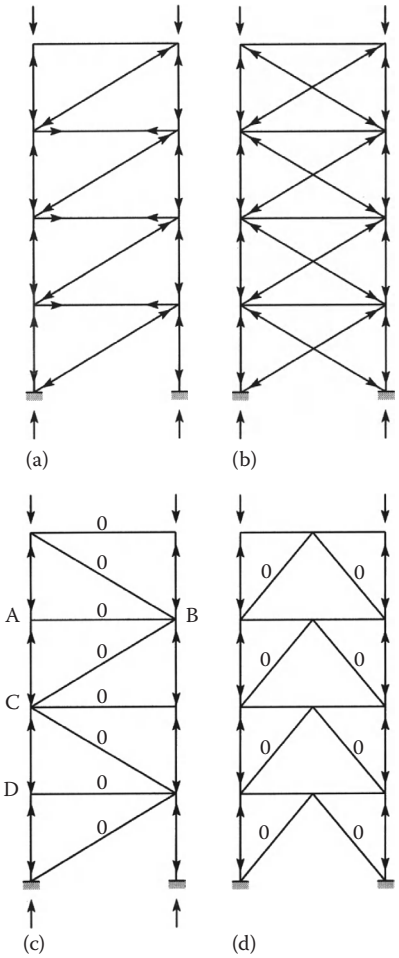


FIGURE 1.17 Gravity load path in braced frames: (a) single diagonal single direction bracing; (b) X-bracing; (c) single diagonal alternate direction bracing; (d) chevron bracing.

horizontal component of the compressive forces in the diagonal. At a first glance, this may appear to be the case for the frame shown in Figure 1.17c. However, the diagonals shown therein, will not attract significant gravity forces because there is no triangulation at the ends of beams where the diagonals are not connected as, for example, at nodes A and D. The only horizontal restraint at the beam-end is by the bending resistance of columns, which usually is of minor significance in the overall behavior. Similarly, in Figure 1.17d, the vertical restraint from the bending stiffness of the beam is too large; therefore, as in the previous case, the diagonals experience only negligible gravity forces.

1.3.2 Types of Concentric Braces

Braced frames may be grouped into two categories as either concentric braced frames (CBF) or eccentric braced frames (EBFs). In CBFs, the axes of all members, that is, columns, beams, and braces intersect at a common point such that the member forces are axial without significant moments. On the other hand, EBFs, utilize axis offsets to deliberately introduce flexure and shear in preselected beam segments to increase ductility.

Braced frames can take on many configurations some of which are shown in Figure 1.18. The diagonal member often consists of double angles, channels, tees, tubes or wide flange shapes, their selection frequently based on the ease of connection details.

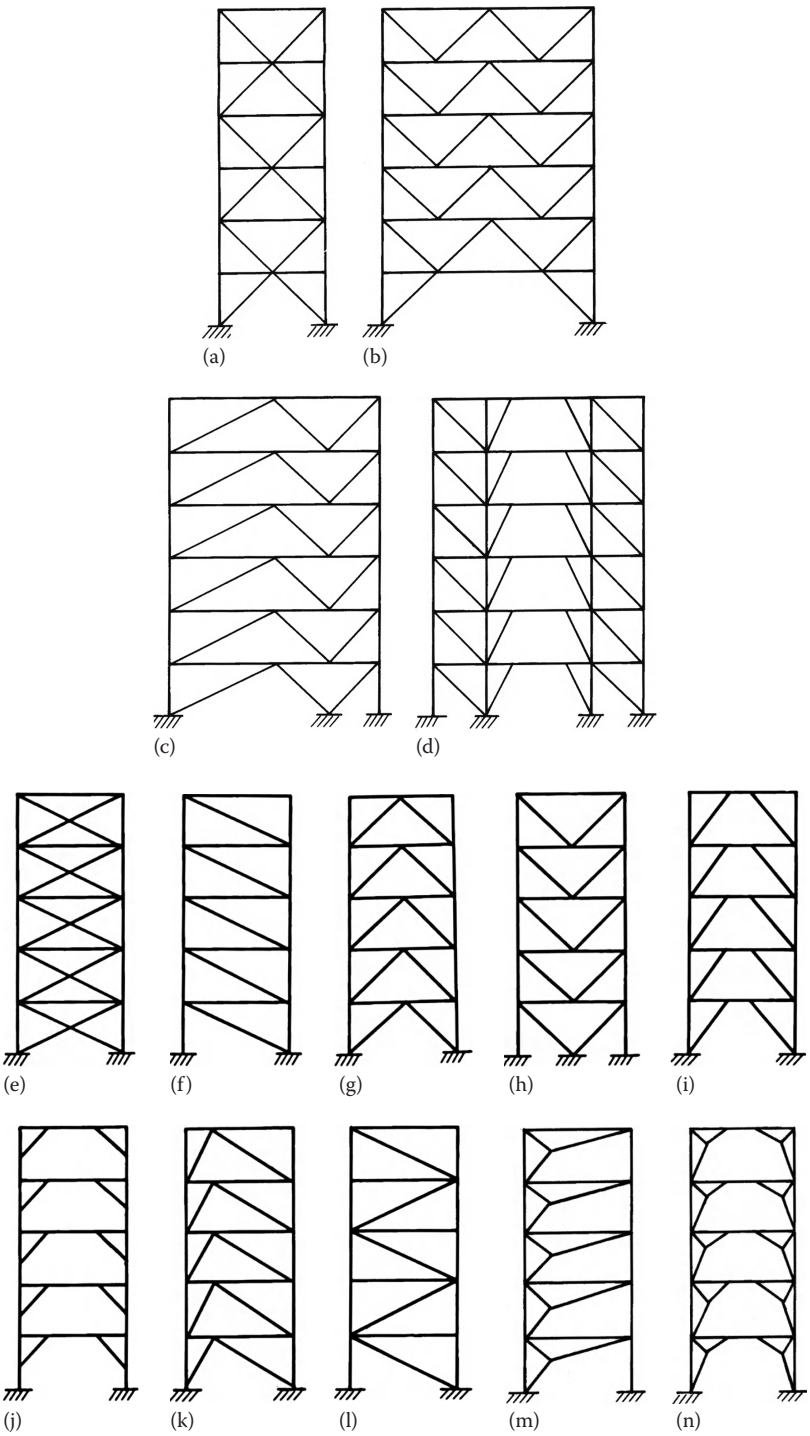


FIGURE 1.18 Bracing configurations.

From architectural considerations, the least objectionable locations for braces are around service cores and elevators, where frame diagonals may be enclosed within permanent walls. The braces can be joined together to form a closed or partially closed three-dimensional cell for effectively resisting torsional loads. Any reasonable configuration with single or multiple braced bays, as shown in Figure 1.18, may be designed for resisting lateral loads. However, availability of proper depth for bracing is often an overriding consideration. As a preliminary guide, a height-to-width ratio of 8–10 is considered proper for a reasonably efficient bracing system.

Braced-frame systems tend to be more economical than moment-resisting frames when material, fabrication, and erection costs are considered. These efficiencies are often offset by reduced flexibility in floor plan layout, space planning, and electrical and mechanical routing, encountered as a result of the space requirements for the brace members. Braced frames are, therefore, typically located in walls that stack vertically between floor levels. In a typical office building, these walls generally occur in the "core" area around stair and elevator shafts, central restrooms, and mechanical and electrical rooms. This generally allows for greater architectural flexibility in articulating vertical modulations of the building envelope. Depending on the plan location and the size of the core area of the building, the torsional resistance offered by the braced frames may become a controlling design parameter. Differential drift between stories at the building perimeter must be considered with this type of layout, as rotational displacements of the floor diaphragms may impose deformation demands on the cladding system and other nonstructural elements of the building.

1.4 ECCENTRIC BRACED FRAMES

CBFs are quite efficient from strength and stiffness considerations and are used widely in lower SDCs A and B.

However, they are of questionable value in seismic regions because of their poor inelastic behavior. Although moment-resistant frames exhibit considerable energy dissipation characteristics, they are relatively flexible when sized from strength considerations alone. Eccentric bracing is a unique structural system that attempts to combine the strength and stiffness of a braced frame with the inelastic energy dissipation characteristics of a moment frame. The system is called *eccentric* because deliberate eccentricities are designed with in the system, typically with in the beam segment.

The eccentric beam element designed as a fuse limits large forces from entering the braces. The eccentric segment of the beam, called the link, undergoes flexural or shear yielding prior to buckling of compression members. Thus, the system maintains stability even under large inelastic deformations. The required stiffness during wind storms or minor earthquakes is maintained because no plastic hinges are formed under these loads and all behavior is elastic. Although the deformation is larger than in a concentrically braced frame because of bending deformation of the "fuse," its contribution to deflection is not significant because of the relatively small length of the fuse. Thus, the elastic stiffness of an eccentrically braced frame can be considered the same as the concentrically braced frame for all practical purposes.

Shown in Figure 1.19 are some common EBF configurations with links at one end of brace.

1.4.1 Behavior

Eccentrically braced frames can be configured in various forms as long as the brace is connected to at least one link. The underlying principle is to prevent buckling of the brace from large overloads that may occur during major earthquakes. This is achieved by designing the link to yield either in shear or in bending.

The shear yielding of beams is a relatively well-defined phenomenon; the corresponding shear yielding load of a beam of given dimensions can be calculated quite accurately. The resulting axial load and moments in columns and braces connected to the link can also be assessed accurately.

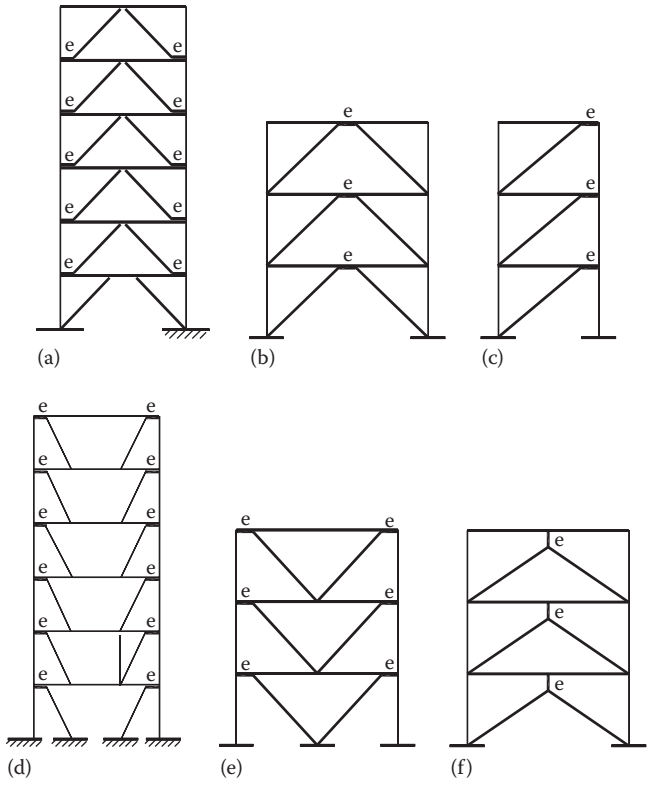


FIGURE 1.19 Eccentric bracing system: (a–f) Links at one end of brace.

Using certain overload factors, explained shortly, the braces and columns are designed conservatively to assure that in the event of a large earthquake, it is the link that behaves as a fuse without seriously overloading the columns or braces.

Whether the link develops plastic hinges or yields in shear is a function of its length. Links longer than twice the depth tend to develop plastic hinges while shorter links tend to yield in shear. Accordingly, the links can be identified either as short or long. Short link experiences moderate rotation, while the longer, a relatively larger rotation.

The cyclic shear yielding is an excellent energy dissipation mechanism because large cyclic deflections can take place without failure or deterioration in the hysteretic behavior. This is because yielding occurs over a large segment of the beam web and is followed by a cyclic diagonal field. The web buckles after yielding in shear, but the tension field action takes over the load-carrying mechanism to prevent failure, resulting in a robust hysteric loop representative of good energy dissipation characteristics.

1.4.2 Deflection Characteristics

The lateral deflection of an eccentrically braced frame can be estimated as the sun of three components: (1) deflection due to axial shortening or elongation of the brace; (2) deflection due to axial strain in the columns; and (3) the deflection due to bending deformation of the link. Because the braces and columns are designed to remain elastic in a design earthquake, their deflection contributions are very nearly the same even after the shear yielding of the link. The beams in EBFs are much heavier than in a concentrically braced frame. Therefore, the bending deformation of the link is likely to contribute very little to the overall deflection of the frame. Hence, an eccentrically braced frame is not an unreasonably flexible system as compared to a concentric frame.

1.4.3 Seismic Design Considerations

It is now well-recognized that eccentrically braced frames can provide two distinct benefits: (1) an elastic stiffness that is comparable to that of special concentrically braced frames (OCBF), particularly when short link lengths are used and (2) excellent ductility and energy dissipation capacity in the inelastic range, comparable to that of SMFs, provided that the links are not too short.

The distinguishing characteristic of an EBF is that at least one end of every brace is connected so that the brace force is transmitted through shear and bending of a short beam segment called the link. Inelastic action in EBF under seismic loading is restricted primarily to the links. Cyclic yielding in the links can, therefore, occur in a stable manner while the diagonal braces, columns, and portions of the beam outside of the link remain essentially elastic under the forces that can be developed by fully yielded and strain-hardened links.

In some bracing arrangements, as shown in Figure 1.20, with links at each end of the brace, links may not be fully effective. If the upper link has a significantly lower design shear strength than that for the link in the story below, the upper link will deform inelastically and limit the force that can be developed in the brace and to the lower link. When this condition occurs, the upper link is termed an inactive link. The presence of potentially inactive links in an EBF increases the difficulty of analysis. Thus, an EBF configuration that ensures that all links will be active, such as those illustrated in Figure 1.19 is recommended.

In EBF systems, lateral forces are resisted by a combination of the flexure, shear, and axial forces in the framing members. Eccentrically braced frames are essentially a hybrid system, offering lateral stiffness approaching that of a concentrically braced frame and ductility approaching that of a moment frame system. The general concept is to have the centerline of a brace member coincide with the intersection of a beam-to-column joint at one end, while at the other end, it intersects the beam away from a column. The section of the beam transferring the vertical shear component of the eccentrically connected brace to the adjacent column or brace is called the "link." The link becomes the focal point in the design and detailing of an EBF system, as it is intended to be the primary location for the inelastic behavior in the frame. The remainder of the members and connections in the EBF are intended to remain essentially elastic and are required to have sufficient strength to withstand forces corresponding to the link yield forces.

While EBF systems combine many concepts of both concentrically braced frames and moment frames, the technology of eccentrically braced frames is relatively new. The system was first developed in Japan in the early 1970s. Research and development in the United States followed later that decade, continuing through the 1980s, with the first codified design procedure in the 1988 Uniform Building Code (UBC).

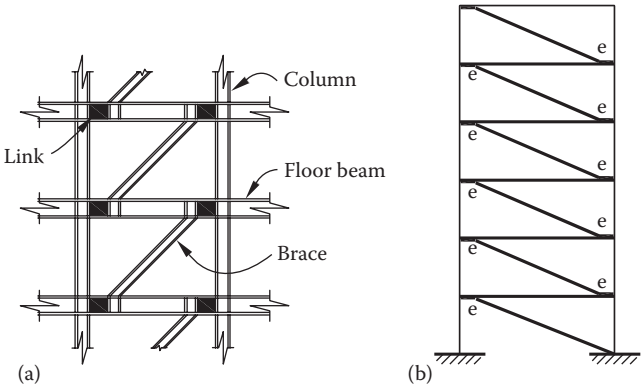


FIGURE 1.20 Eccentric bracing configurations: (a and b) Links at each end of brace.

1.4.3.1 Link Beam Design

The focal points in the design of an EBF link are as follows:

1. Limit the web width-to-thickness ration h/t_w based on the ratio $P_u/\varphi_b F_y$. See AISC Seismic, Section 15 for details.
2. Limit specified yield stress of link web to 50 ksi.
3. No double plates or penetrations permitted in the link zone.
4. The design shear strength, φV_n is equal to lesser of

$$\varphi V_n = 0.90 \times 0.6 F_y A_w$$

$$\varphi V_n = 0.90 \times 2 M_{p/L}$$

5. If P_u, the axial strength in the link exceeds $0.15P_y$, additional requirements apply. See AISC, seismic, Section 15.2b for details.
6. Limit the length, e of the link to a target value of $1.6M_p/V_p$. This will keep the link length short enough to be controlled by shear. Observe shear yielding is much more reliable than flexural yielding.
7. When the link is directly adjacent to a column:
   a. The connection is required to meet the provisions for SMRF, unless the connection is reinforced to force beam yielding to a location away from the column.
   b. Laterally brace both top and bottom flanges of the link beams at the ends of link. A composite floor system may be assumed to provide lateral bracing for the top flange but not for the bottom flange of the link. Provide independent bracing for the bottom flange. In all cases, lateral restraint against out-of-plane displacement and twist is required at the ends of link to ensure stable inelastic behavior of the link.
   c. For EBF configuration shown in Figure 1.19f, provide lateral bracing at the intersection of the diagonal braces and the vertical link.
8. Limit the link rotation angle V_p to 0.08 rad for shear yielding links ($e \leq 1.6M_p/V_p$) and 0.02 rad for flexural yielding links ($e \leq 2.6M_p/V_p$). Use linear interpolation for links in the combined shear and flexural yielding range. See Figure 1.21 for the definition of link rotation angle for a link located at the center of a bay. Refer to AISC Seismic Commentary for other conditions.

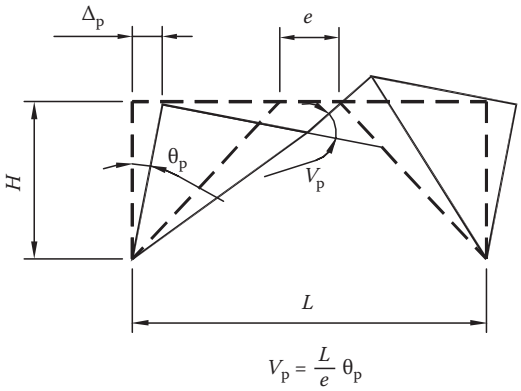


FIGURE 1.21 Link rotation angle λ_p: This is related to the plastic story drift Δ_p, and is strongly influenced by the link length. The limits of V_p are 0.08 and 0.02 radians for shear yielding and flexural yielding links. L = Bay width, H = Story height, Δ_p = Plastic story drift (conservatively, may be taken equal to the design story drift), θ_p = Plastic story drift angle = Δ_p/H, V_p = Link rotation angle.

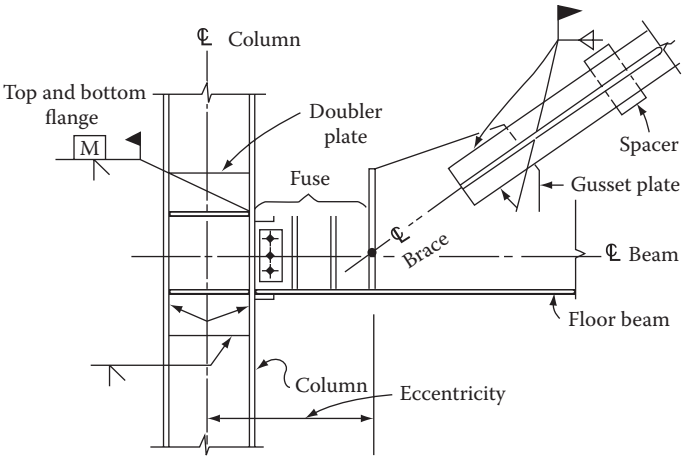


FIGURE 1.22 Schematics of link-to-column connection: note that beam-to-column connections which qualify for use in an SMF may not necessarily perform adequately when used as a link-to-column connection in an EBF. Link-to-column connections must therefore be tested in a manner that properly simulates the forces and inelastic deformations expected in an EBF.

1.4.3.2 Link-to-Column Connections

Schematics of a link-to-column connection are shown in Figure 1.22. It should be understood, however, that because of the problems observed in moment frame connections after the 1994 Northridge earthquake, the AISC Seismic Provisions require that the performance of link-to-column connections be verified by testing, or by the use of prequalified link-to-column connections. The load and deformation demands at a link-to-column connection in an EBF are substantially different from those at a beam-to-column connection in an SMF. Link-to-column connections must therefore be tested in a manner that properly simulates the forces and inelastic deformations expected in an EBF.

The AISC Seismic Provisions permit the use of link-to-column connections without the need for qualification testing for shear yielding links when the connection is reinforced with haunches or other suitable reinforcement designed to preclude inelastic action in the reinforced zone adjacent to the column. An example of such a connection is shown in Figure 1.23.

1.4.3.3 Diagonal Brace and Beam outside of Links

The intent in seismic design is to ensure that yielding and energy dissipation in an EBF occur primarily in the links. Consequently, the diagonal brace and beam segment outside of the link must be designed to resist the loads developed by the fully yielded and strain-hardened link.

In most EBF configurations, the diagonal brace and the beam are subject to large axial loads combined with significant bending moments. Consequently, both the diagonal brace and the beam should be designed as beam–columns.

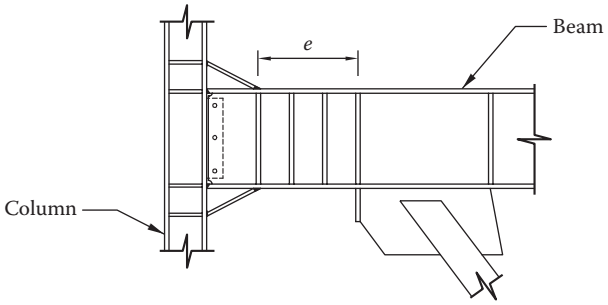


FIGURE 1.23 Example of reinforced link-to-column connection.

A diagonal brace in a concentrically braced frame is subject to cyclic buckling and is the primary source of energy dissipation in such a frame. A properly designed diagonal brace in an EBF, on the other hand, should not buckle, regardless of the intensity of the earthquake ground motion. As long as the brace is designed to be stronger than the link, then the link will serve as a fuse to limit the maximum load transferred to the brace, thereby precluding the possibility of brace buckling. Consequently, many of the design provisions for braces in OCBF and SCBF systems intended to permit stable cyclic buckling of braces are not needed in EBF. Similarly, the link also limits the loads transferred to the beam beyond the link, thereby precluding failure of this portion of the beam if it is stronger than the link.

1.4.3.4 Link Stiffness

A properly detailed and restrained link web can provide stable, ductile, and predictable behavior under severe cyclic loading. Full-depth stiffeners are required at the ends of all links and to restrain the link web against buckling.

The maximum spacing of link intermediate web stiffeners in shear yielding links is dependent upon the size of the link rotation angle with a closer spacing required as the rotation angle increases. Intermediate web stiffeners in shear yielding links are provided to delay the onset of inelastic shear buckling of the web. Flexural yielding links are required to have an intermediate stiffener at a distance from the link end equal to 1.5 times the beam flange width. This is to limit strength degradation due to flange local buckling and lateral–torsional buckling. Links of a length that are between the shear and flexural limits are required to meet the stiffener requirements for both shear and flexural yielding links. When the link length exceeds $5M_p/V_p$, link intermediate web stiffeners are not required. Link intermediate web stiffeners are required to extend full depth to effectively resist shear buckling of the web and to effectively limit strength degradation due to flange local buckling and lateral–torsional buckling. Link intermediate web stiffeners are required on both sides of the web for links 25 in. in depth or greater. For links that are less than 25 in. deep, the stiffener need be on one side only. All link stiffeners are required to be fillet welded to the link web and flanges and should be detailed to avoid welding in the *k*-area of the link.

Shown in Figures 1.24 and 1.25 are schematics of EBFs with W-shape and HSS bracing.

1.4.3.5 Columns

Similar to the diagonal brace and beam segment outside of the link, the columns of an EBF should be designed to resist the maximum forces developed by the fully yielded and strain-hardened links. The maximum shear force developed by a fully yielded and strain-hardened link can be estimated as $1.25R_y$ times the link nominal shear strength V_n, where the 1.25 factor accounts for strain hardening. It is permitted however to reduce the strain hardening factor to 1.1. This relaxation reflects the view that all links above the level of the column under consideration particularly in tall buildings, will not likely reach their maximum shear strength simultaneously. Consequently, applying 1.25

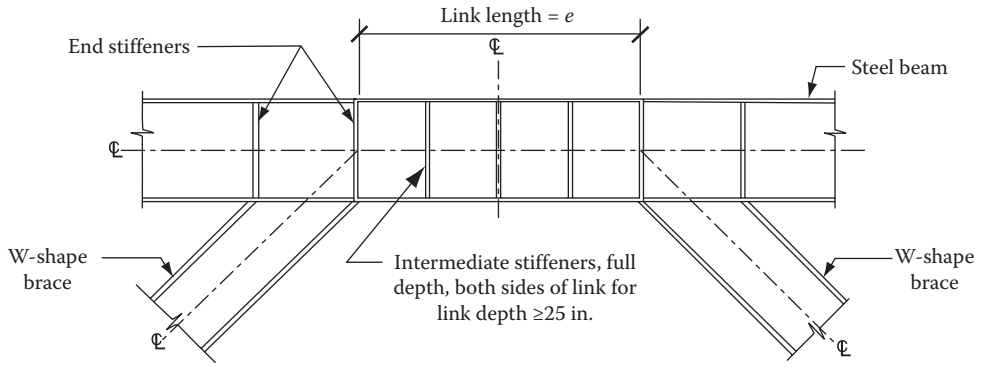


FIGURE 1.24 EBF with W-shape bracing.

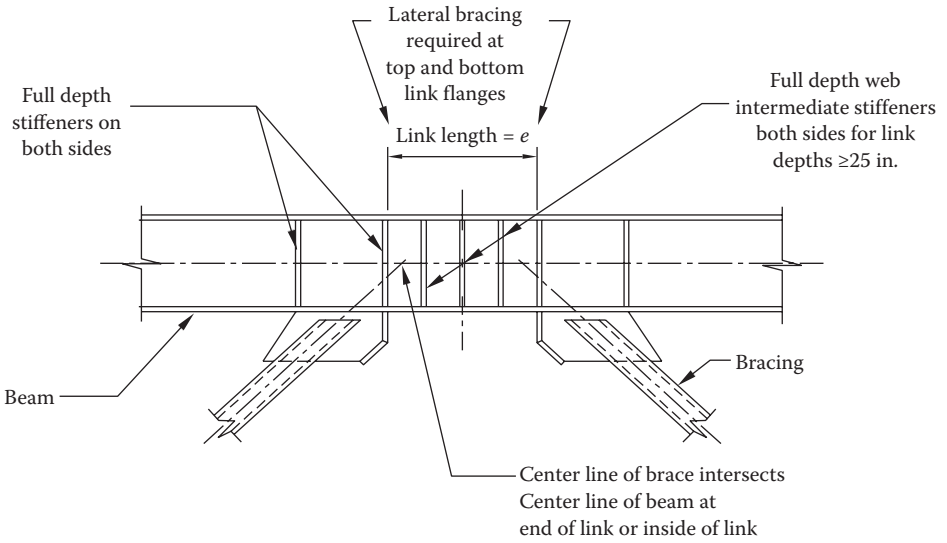


FIGURE 1.25 EBF with HSS bracing.

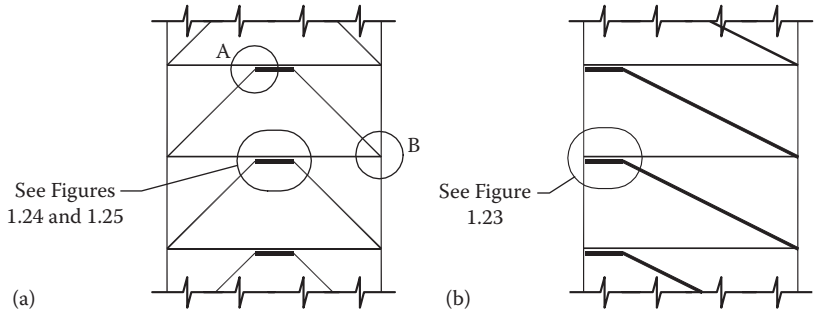


FIGURE 1.26 (a and b) Key elevations of EBF.

strain hardening factor to all links above the level of the column under consideration is likely too conservative for a multistory EBF. However, for a low-rise EBF with only a few stories, designers should consider using strain hardening factor of 1.25 for capacity design of the columns, since there is greater likelihood that all links may simultaneously reach their maximum shear strength.

1.4.3.6 Schematic Details

Shown in Figures 1.26 through 1.28 are key connection details for a typical EBF.

1.5 BUCKLING-RESTRAINED BRACE FRAME

Buckling-restrained brace frames (BRBFs), shown in Figure 1.29, have a high degree of ductility (energy absorbing capability) and good lateral stiffness, and are relatively simple to repair if need be, after a major earthquake. Unbonded brace frames, which may be considered a special class of BRBFs, consist of a steel core installed within an outer shell with mortar infill between the plate and the shell. An unbonding agent is applied to the core plate to prevent it from transmitting axial load to the buckling-restraining mechanism. The unbonded brace element, typically a diagonal member, consists of a restrained yielding segment, nonyielding restrained steel segments, and nonyielding unrestrained segments. The yielding segment commonly referred to as the core typically consists of a steel plate. The nonyielding segments are typically of cruciform shape. The entire assembly is

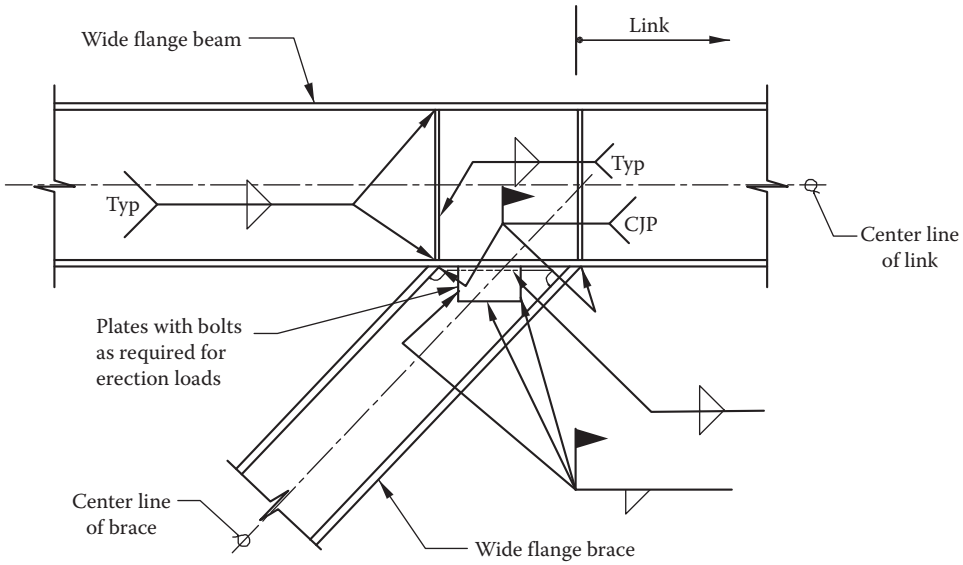


FIGURE 1.27 Detail A.

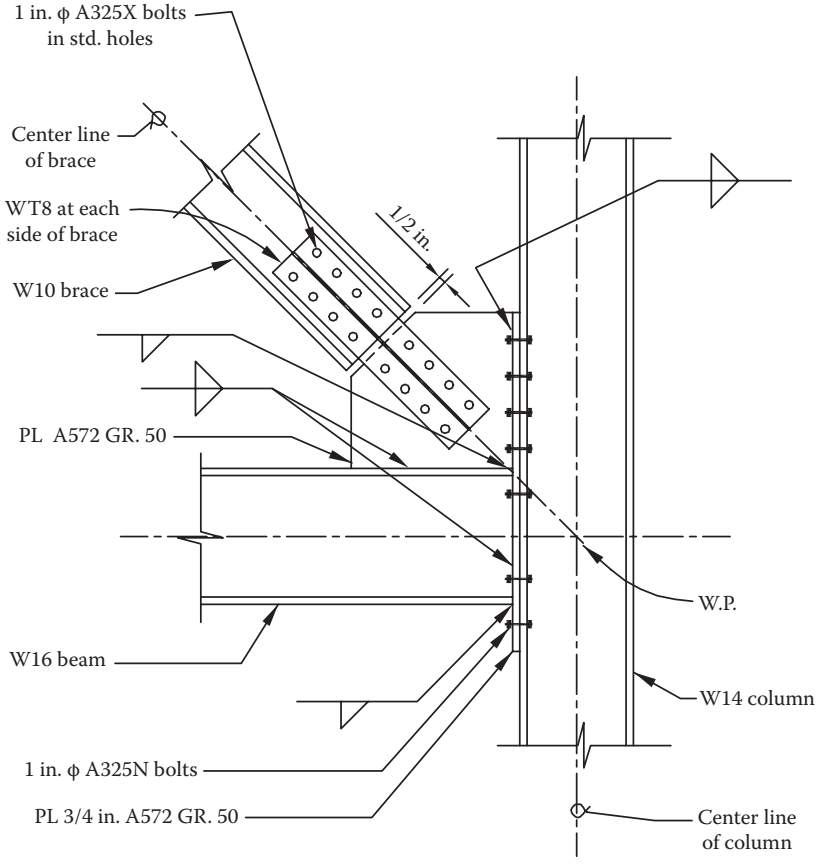


FIGURE 1.28 Detail B.

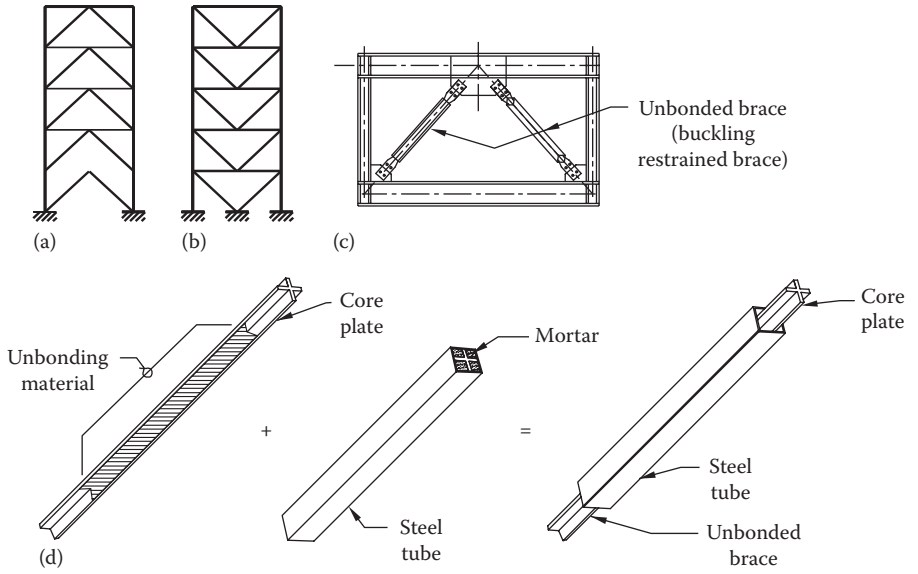


FIGURE 1.29 Main components of buckling restrained brace frames: (a and b) elevations; (c) BRBF Chevron braces; (d) bracing components. *Note*: K and X-braces are not viable options for BRBF.

procured as a preassembled unit manufactured to meet the performance objectives specified by the engineer. See Figures 1.30 and 1.31 for schematic components of two types of bracing. Figure 1.30 shows a circular sleeve while Figures 1.29 and 1.31 show a rectangular HSS.

Because the braces are able to yield without buckling in compression, well-defined, stable, and symmetric hysteric loops are generated when the braces are subjected to reverse cyclic loading. This results in excellent energy dissipating characteristics.

The use of a BRBF as a seismic-lateral-resisting system is relatively new in the United States. However, it is similar to a special concentrically braced frame in that it also has a triangulated vertical framework of members that resist lateral loads through axial tension and compression. The main difference is that the buckling-restrained braces achieve significantly higher ductility and energy dissipation characteristics by effectively eliminating buckling and the poor hysteric performance associated with it. Because of this, the tension and compression behaviors of the brace are very similar.

In buckling-restrained braces, a fairly long segment can yield in compression as well as in tension. The yielding segment is part of an axial-force-resisting steel core. Although its effective slenderness is extremely low, buckling is not an issue due to the lateral restraint provided by surrounding casing of steel infilled with mortar. For buckling to be precluded, this casing must be kept free from axial forces. Several methods of confining the axial force to the steel core are in use in the United States. Most of these are developed around proprietary specifications, and some are patented.

As buckling-restrained braces are typically a specification item, the required brace strengths are generally specified by the design engineer. Customarily, the manufacturer designs the braces to comply with the given requirements using the material and grade specified for the element. Since the material grade has a significant effect on the brace stiffness, the lower the yield stress, the greater the required area of steel, resulting in a stiffer brace. Decreasing the yield length concentrates the inelastic strain, reducing the cumulative energy dissipation capacity.

Because the tension and compression strengths of buckling-restrained braces are similar, a chevron configuration (see Figures 1.29 through 1.31) does not penalize the design of the beam connected to the chevron braces.

With certain simplifications, gusset plates at the connections are designed similar to those for SCBF. However, BRBF gussets are not required to accommodate buckling of the brace; hinge zones are therefore not required, nor are the gussets required to have flexural strength in excess of the brace.

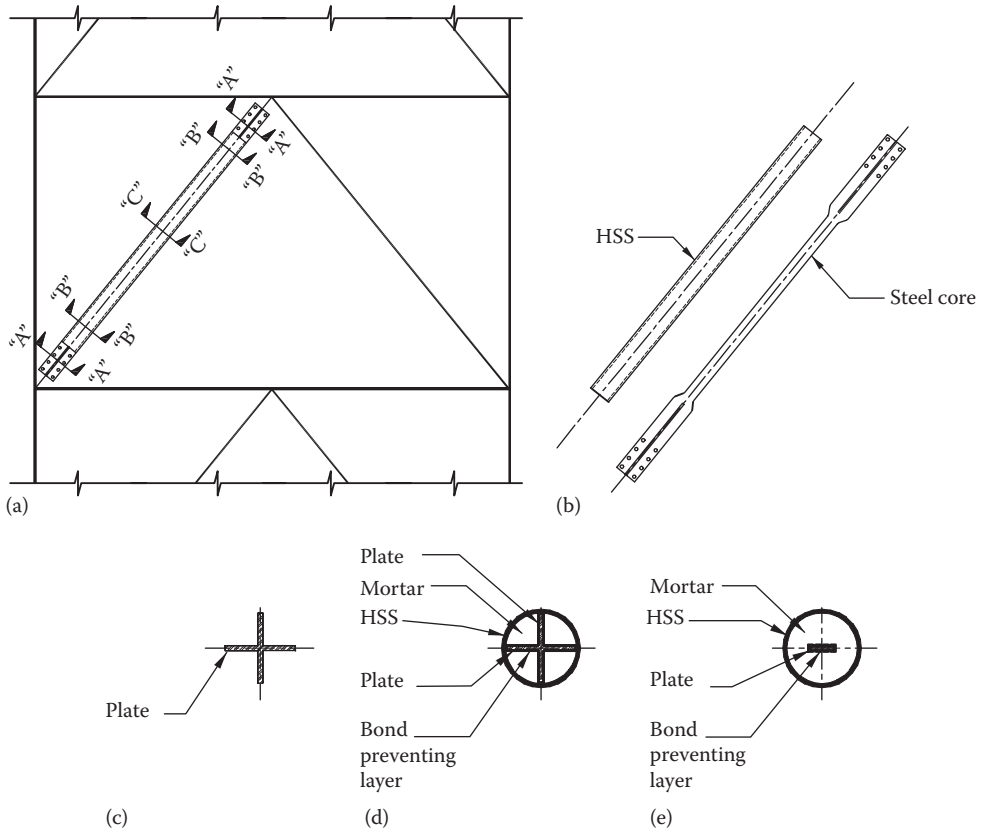


FIGURE 1.30 Details of buckling restrained brace: (a) partial elevation of BRBF; (b) HSS encasement and steel core; (c) section AA; (d) section BB; (e) section CC.

Small eccentricities may be permissionable in the connection design if the resulting brace rotations are still within the tested limits.

The maximum connection force is calculated using the brace strength and overstrength factors β and ω determined from testing. The factor β represents the overstrength in compression (buckling-restrained braces tend to be somewhat stronger in compression than in tension), and ω represents strain-hardening within the expected deformation range. The factor R_y, representing expected yield strength as compared to nominal yield strength, is assumed not to be applicable in sizing of the braces because the final cross-sectional size of a buckling-restrained brace is typically determined considering the material yield strength as measured from coupon tests. The brace yield strength can thus be calculated without guesswork from the required strength and resistance factor.

Column design forces are determined using the special seismic load combinations specified in the Seismic Provisions. Although this can be done in a manner similar to that for SCBF, BRBF tends to have much lower overstrength. Therefore, an explicit consideration of brace capacity can usually result in lower column design forces, resulting in savings in the columns and foundations.

As mentioned previously, buckling-restrained braced frames are a special class of concentrically braced frames. As in the case of special concentrically braced frames (SCBF), the centerlines of BRBF members intersect at a point to form a complete vertical-truss system. BRBF systems have more ductility and energy absorption capacity than SCBF systems because overall brace buckling, and its associated strength degradation, is precluded at forces and deformations corresponding to the design story drift. Buckling-restrained braced frames are composed of columns, beams and bracing elements, all of which are subjected primarily to axial forces. Braces of buckling-restrained

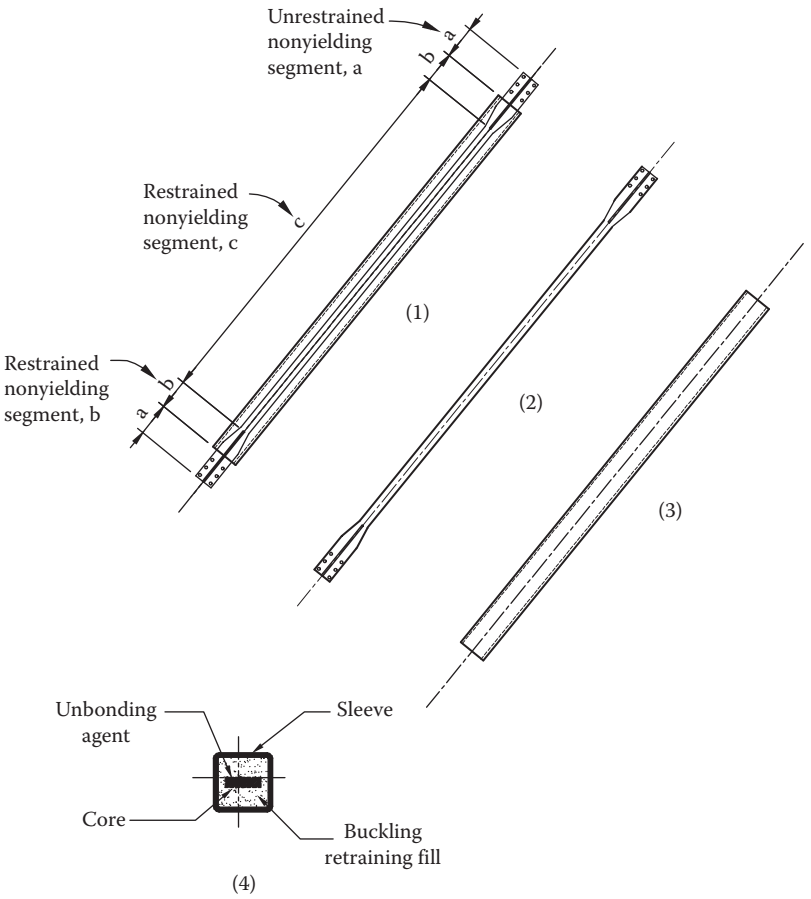


FIGURE 1.31 Components of buckling restrained brace: (1) buckling restrained brace; (2) core; (3) sleeve; (4) section. The yielding of core plates in compression without buckling results in a stable hysteric loop with excellent energy-dissipating characteristics.

braced frames are composed of a steel core and restraining system encasing the steel core that prevents buckling.

The steel core within the bracing element is intended to be the primary source of energy dissipation. During a moderate to severe earthquake the steel core is expected to undergo significant inelastic deformations.

BRBF systems can provide elastic stiffness that is comparable to that of an EBF or a SCBF. The ductility and energy dissipation capability of BRBF is expected to be comparable to that of an EBF or SMF and greater than that of SCBF. This high ductility is attained by limiting buckling of the steel core.

1.6 STEEL PLATE SHEAR WALL

Steel plate shear walls consist of columns, beams, and steel plate elements. The plate infills the boundaries created by the columns and beams. The behavior of special steel plate shear wall is analogous to a vertically cantilevered steel plate girder with the columns acting as the plate girder flanges, the beams acting as the intermediate stiffeners, and the steel plate acting as the web of the plate girder. Figure 1.32 shows an elevation of a steel plate shear wall that can be used to configure independent planar walls, coupled walls linking two planar walls with coupling beams, or interconnected walls to create a three-dimensional system. Special plate shear walls (SPSW) described in this section can also be used as part of a dual system, utilizing moment frames designed to carry a

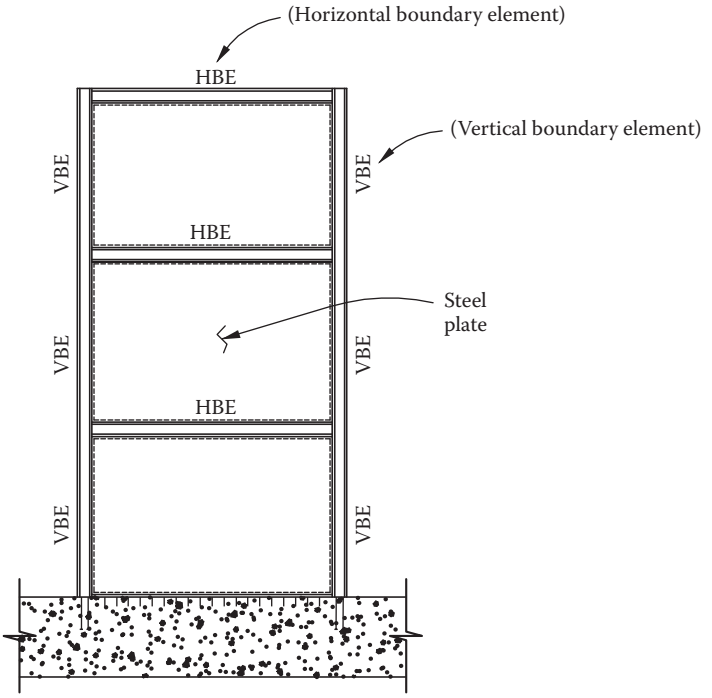


FIGURE 1.32 Steel plate shear wall.

portion of the prescribed seismic forces. The steel plate provides the primary shear resistance, and may be stiffened or unstiffened. When the plate is designed as stiffened, it is designed to prevent buckling of the plate prior to shear yielding, and when designed as unstiffened, it is expected to buckle in order to utilize tension-field action. Test results have indicated that SPSWs provide a very stiff and ductile seismic load resisting system, capable of dissipating significant amounts of energy. Hence, their use is on the increase in the west coast both for seismic retrofit and new buildings.

1.6.1 Low-Seismic Design

The term low-seismic refers to structural systems that (1) are not expected to undergo significant inelastic deformations, (2) are not designed to meet the requirements of the Seismic Provisions, and (3) have an R factor equal to or less than 3. The general term used for the system with steel boundary elements and web plates is the steel plate shear wall (SPW). Low-seismic system is not proportioned to fully yield the web plate. Instead, the forces determined from the analysis may be used directly for sizing the web plate, the horizontal boundary element, HBE, and the vertical boundary element, VBE.

1.6.2 High-Seismic Design

Akin to composite construction of structural steel and reinforced concrete, the concept of shear steel plate shear wall may seem like a new idea. It is not. The idea has been around for decades, and has been used in a significant number of buildings. The main focus of the recently published AISC Seismic Provisions AISC 341-10, like its predecessor 341-05, is not on the concept as such, but on the design requirements and recommendations for SPSW system, also referred to as high-seismic system. The term high-seismic, refers to structural systems that are expected to undergo significant inelastic deformations, designed to meet the requirements of the Seismic Provisions, and have an R factor greater than 3.

In steel plate shear wall terminology, the plate connected to the columns and beams is referred to as the web plate. The columns are referred to as VBEs and the beams, HBEs.

1.6.2.1 Behavior

Web plates in steel plate shear walls are categorized into two graphs according to their ability to resist buckling: (1) stiffened and (2) unstiffened. They are called stiffened if they are sufficiently stiffened to preclude buckling and allow development of full shear strength. It should be noted, however, that while stiffening increases the effectiveness of web plates, they are not as economical as "unstiffened" web plates in which buckling of the web plates is permitted.

In typical designs, the webs are unstiffened and slender. They are therefore capable of resisting large tension forces, but little or no compression. As lateral loads are imposed on the system, shear stresses develop in the web until the principal compression stresses, oriented typically at 45° angle to the shear stress, exceed the compression strength of the plate. At this point, the web plate buckles and forms diagonal fold lines. The lateral loads are transferred by principal tension stresses oriented at an angle α, which, for conceptual purposes, may be considered equal to 45°. For optimum design, the web should be designed to reach full tension yield at each level simultaneously.

1.6.2.2 AISC 341-05 Requirements for Special Plate Shear Walls

In general terms, the AISC requirements are based on the following principles:

1. The web plates are assumed to reach full tension yielding at an angle α. This angle is based on the wall geometry and the properties of the boundary elements.
2. The webs are designed to satisfy the shear demand determined from the applied loads.
3. To ensure that the webs can reach their full tensile strength, the required strengths of the connections to the boundary elements are based on the fully yielded strength of the web, using the expected tension yield stress, $R_y F_y$.
4. The boundary elements are designed to remain essentially elastic (with the exception of the anticipated plastic hinging at the ends of the HBE) when the web reaches its expected tensile strength at angle α. Because the webs are assumed to fully yield in tension, the required strengths of the boundary elements and their connections are based on strength of web and the plastic moment strength of HBE, combined with gravity loads.
5. The VBE–HBE moment ratio must meet the requirements specified in AISC Seismic for SMFs. This requirement is included to provide columns that are strong enough to force flexural yielding in beams in multiple levels of the frame, thereby achieving a higher level of energy dissipation.
6. The width–thickness ratios of the boundary elements must meet the same requirement as SMF. This requirement recognizes the fact that significant frame action occurs in the system and ensures that the moments are compact enough to undergo significant inelastic deformation.
7. For the same reason, HBE have lateral bracing requirements similar to the beams in SMF.
8. The connections of the HBE to the VBE are expected to form plastic hinges, but they are not the intended source of energy dissipation. The SPSW is not expected to undergo as much drift as an SMF; therefore, the requirements of an SMF moment connection are not necessary. Instead, the expected performance is that of an ordinary moment frame connection.
9. The stiffness of the VBE is critical for achieving uniform tensile yielding in the entire web. Therefore, the VBE is required to have a minimum flexural stiffness.
10. The panel zone requirements for the VBE at the top and base HBE of the SPSW are the same as those for SMF.

1.6.2.3 Modeling for Analysis

Two modeling techniques are in vogue using: (1) strip model and (2) membrane model. In the *strip model*, the web plate is modeled as a series of diagonal strips that resist only tension forces. The strips are aligned at an angle, α, approximately equal to 45°.

In the *membrane model*, the web plate is modeled using orthotropic membrane elements as opposed to isotropic elements. The reason is to model the differing compression and tension resistance of the web plate. The local axes of the elements are set to match the calculated angle of tension stress, α. The material properties in the axis aligned with α represent the tensile behavior of the membrane. The stiffness in the orthogonal direction representing compression capacity of the web is set to zero.

1.6.2.4 Capacity Design Methods

Once the shear force in the web plates is determined from an analysis, a capacity design is used to determine the forces in the boundary elements and their connections. The reader is referred to the Commentary Section of AISC 361-10 for further details.

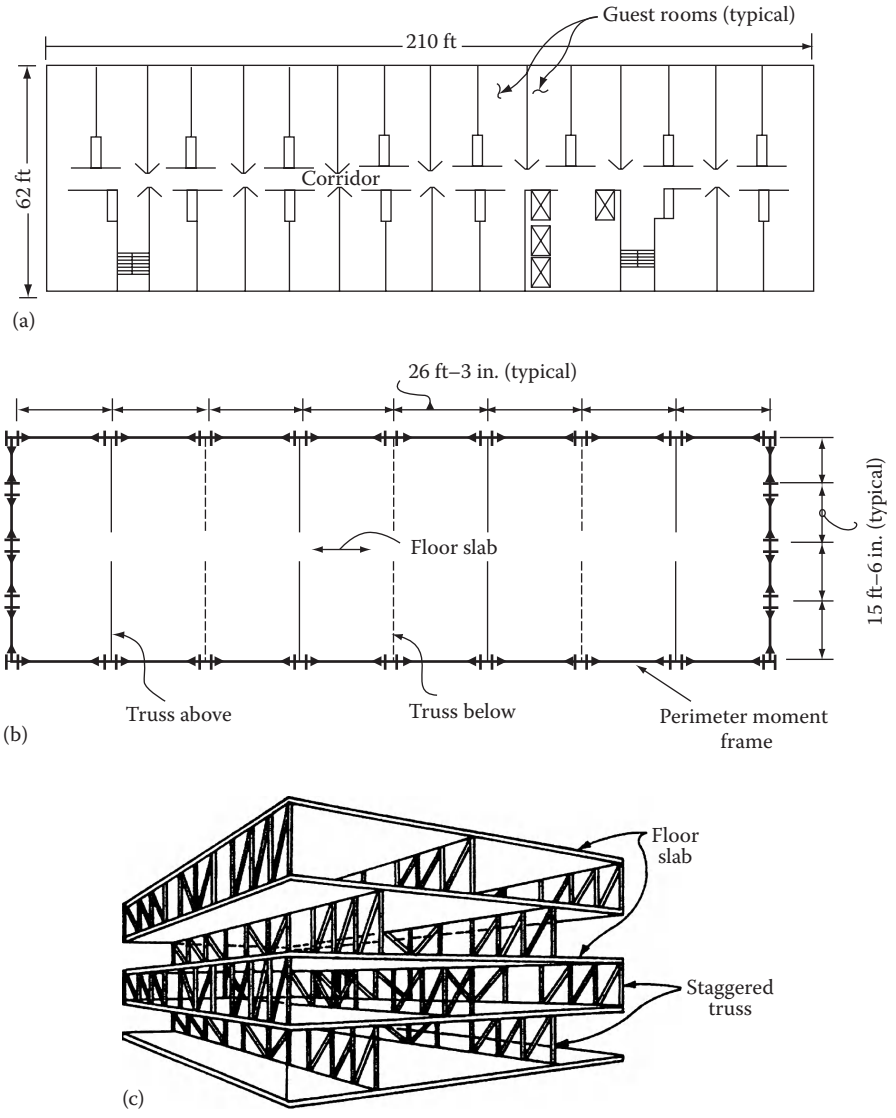


FIGURE 1.33 Staggered truss system: (a) architectural floor plan; (b) arrangement of staggered truss, plan; (c) schematics.

1.7 STAGGERED TRUSS

Residential buildings such as apartments, condominiums, and hotels are generally 60 ft in width with a length varying anywhere from about 100–200 ft or even more. To take advantage of this unique plan layout, a study was conducted in Massachusetts Institute of Technology (MIT) to investigate the possibility of developing a new system for this plan. A system commonly referred to as *staggered truss system* evolved as an outgrowth of the study. In this system story-high trusses span the entire width of the building in the transverse direction between exterior columns. The trusses are arranged in a staggered pattern at alternate floors, as shown schematically in Figures 1.33 and 1.34. The floor acts as a diaphragm transferring the lateral loads to the trusses. The columns, therefore, receive no bending moments by frame action. To allow for an uninterrupted corridor, the truss diagonals are eliminated at the location of corridors, typically at the center of the truss. See Figure 1.34. Since the diagonals are eliminated, the shear is carried by Vierendeel action of the truss vertical members and the top and bottom chords.

Vierendeel openings other than these required for corridors should be avoided in the interest of economy. Truss design is based on continuous chord and pinned web members. Generally,

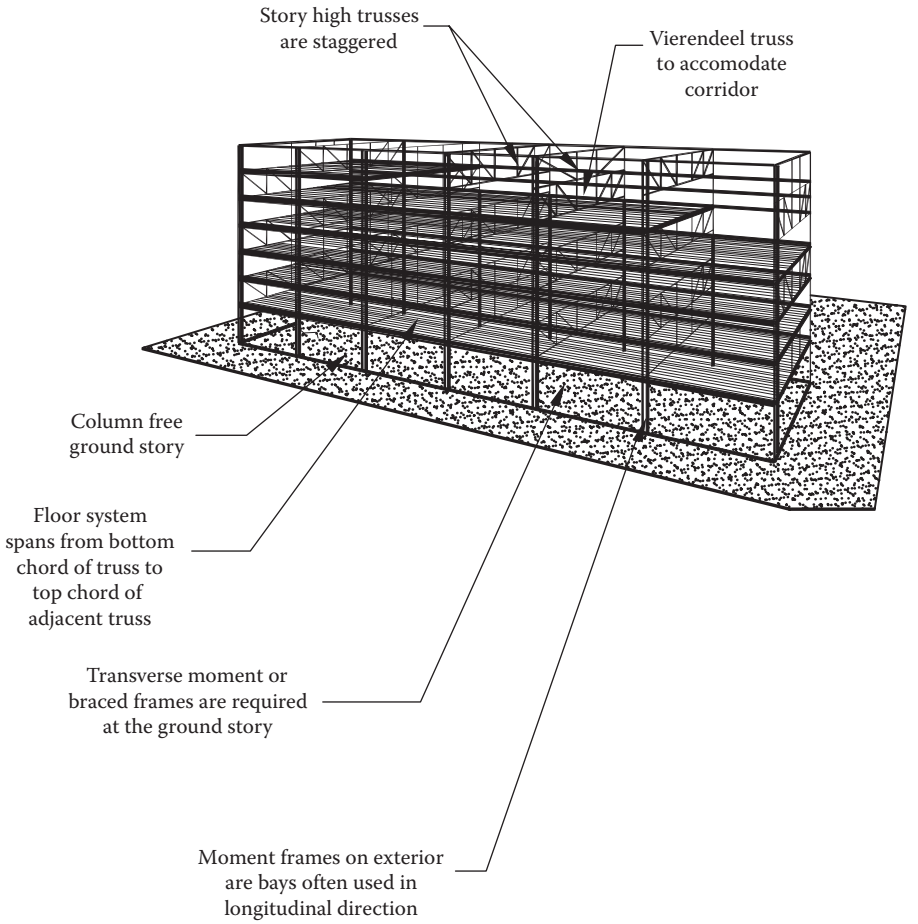


FIGURE 1.34 Staggered truss bracing for a rectangular building.

wide flange shapes are selected for the chord members since they are efficient in resisting the secondary bending between panel points.

An added advantage of this system is that it allows for column-free interior space on the lower level. The staggered trusses are typically spaced uniformly across the length between single residential units. However, one-, two-, or three-bedrooms can be arranged on a single floor by varying the column spacings. The system is not limited to simple rectangular plans. It can be used in curvilinear and serpentine plans as shown in Figure 1.35.

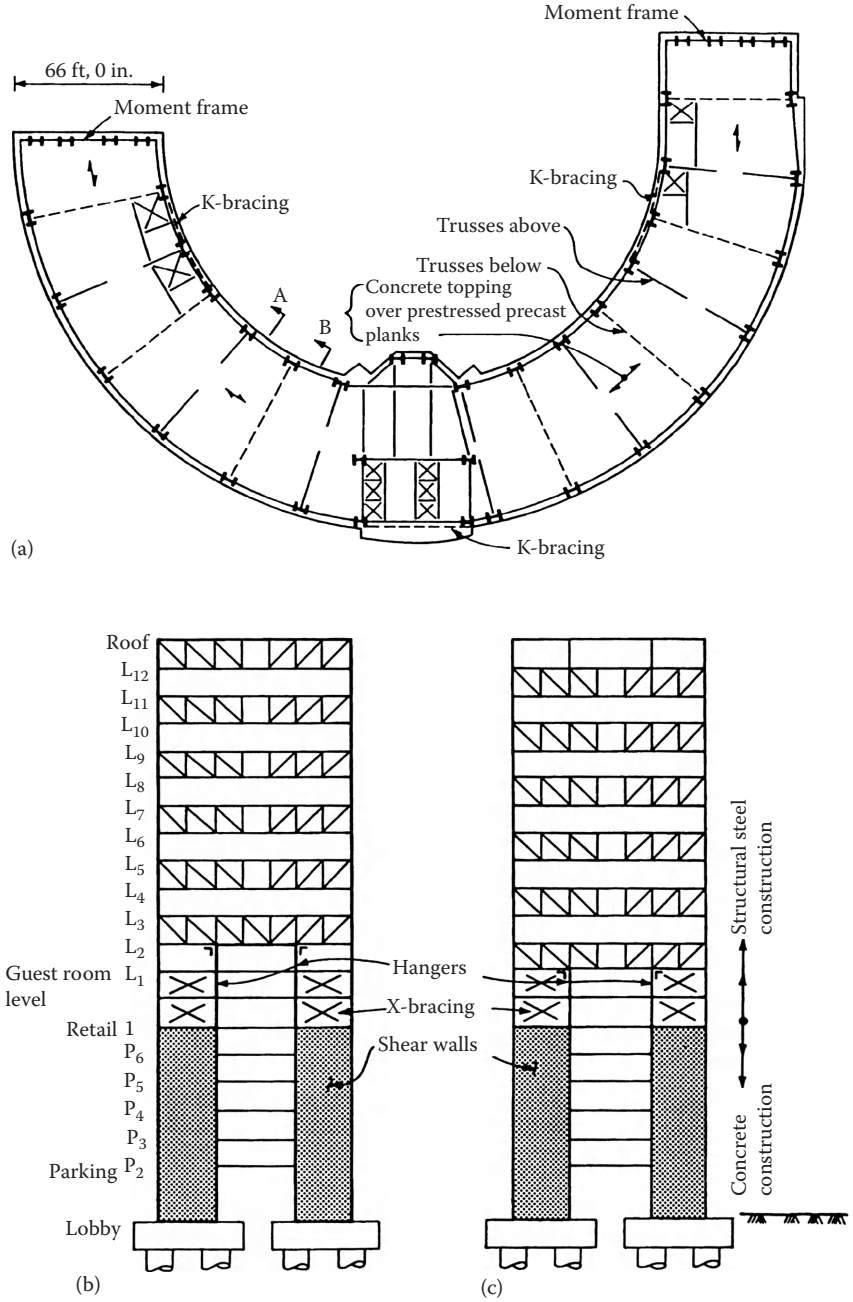


FIGURE 1.35 Staggered truss system for a circular building: (a) plan; (b) section A; (c) section B.

1.7.1 Behavior

Consider the three-dimensional interaction between vertical bracings of a building interconnected through floor diaphragms, as shown in Figure 1.36. Assume for architectural reasons, we are required to eliminate the bracings at A and B below level 2, but it is permissionable to add a bracing at the center of the building as shown in Figure 1.37. This arrangement permits transfer of shear

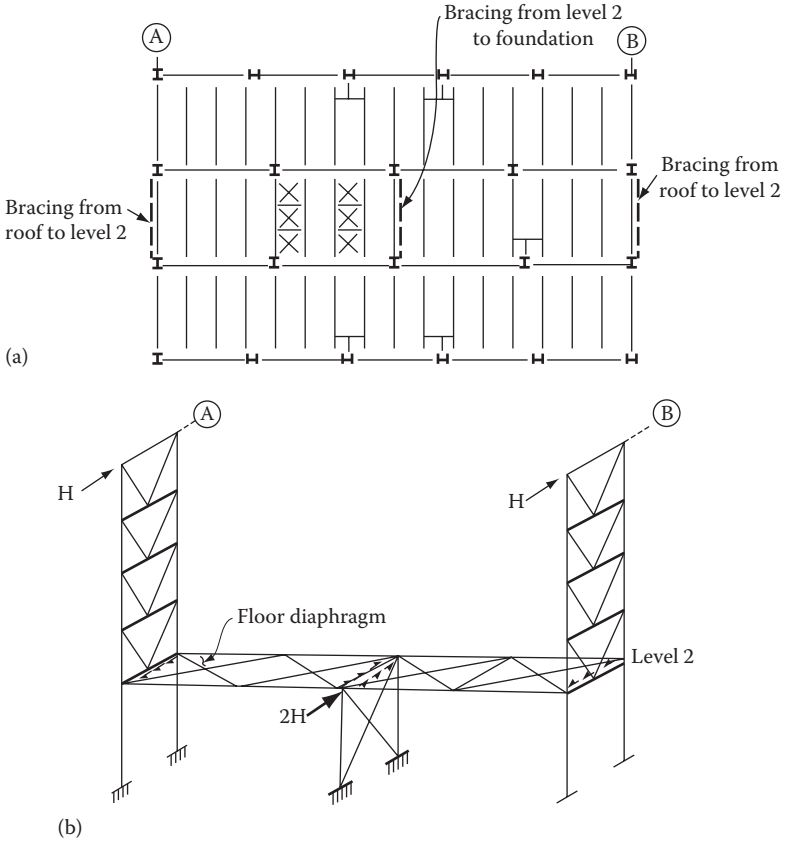


FIGURE 1.36 Conceptual model for staggered truss system: (a) building plan; (b) lateral load transfer through diaphragm action.

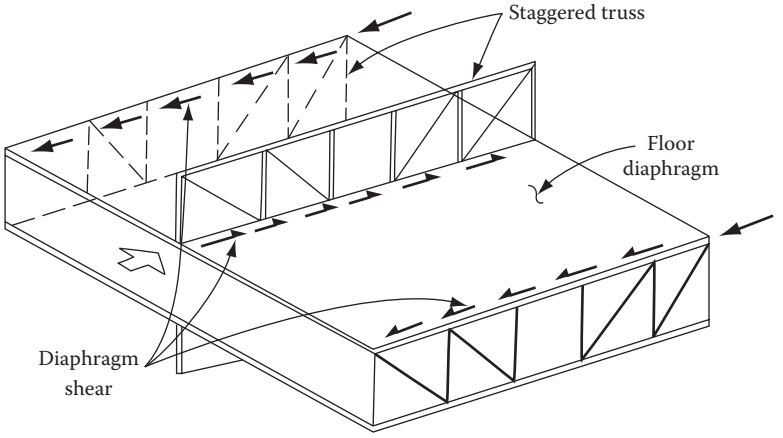


FIGURE 1.37 Load path in staggered truss system.

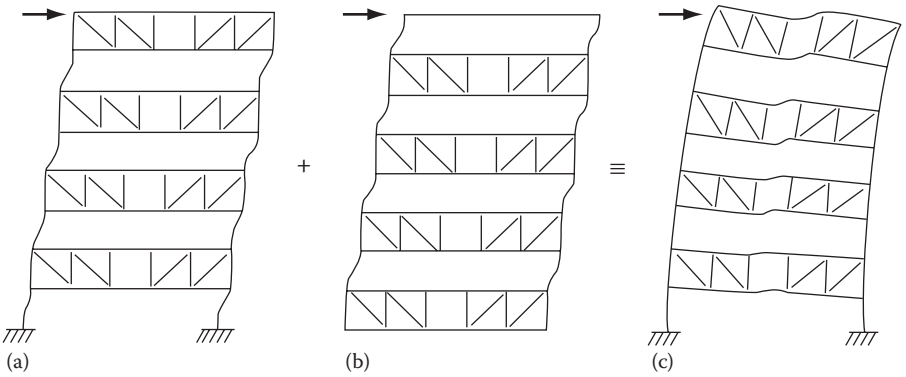


FIGURE 1.38 Conceptual two-dimensional model for staggered truss system: (a and b) lateral deformation of adjacent bays; (c) overall behavior. *Note*: the absence of local bending of columns.

from the exterior braces to the interior brace through diaphragm action of the floor system. The columns below the exterior braces are therefore subjected to axial stresses only due to overturning moments, while the entire shear is resisted by the interior bracing below level 2. This in essence is the structural action in a staggered truss system. The lateral force in a given truss is transmitted across the floor to the adjacent truss below and continues down across to the next floor as shown schematically in Figure 1.37.

The columns in-between the floors, as shown in Figure 1.38, receive no bending moments due to frame action. It is as though the entire building has a vertical truss for the entire height, resulting in an efficient structure. Since the trusses are placed at alternate levels on adjacent column lines, two-bay-wide column-free interior floor space is created in the longitudinal direction.

1.7.2 Design Considerations

1.7.2.1 Floor Systems

As in other systems, the floor in a staggered truss building needs to fulfill two primary requirements: (1) collect and transmit gravity loads to the vertical members and (2) resist lateral loads as a diaphragm to provide a continuous load path for transferring the lateral loads from the bottom chord of one truss to the top chord of the adjacent truss down through the structure. Thus, one could use precast concrete planks, long-span composite steel decks, open-web joist system, or any other system consistent with the structural and architectural requirements. Precast planks and flat-bottom steel decks are often used as exposed ceilings with minimum of finish. For spans up to 30 ft (9.15 m), 8 in. (203 mm) thick planks are required, while for spans less that 24 ft (7.3 m), 6 in. (152 mm) thick planks are adequate. In a composite steel deck system, a 7½ in. (190 mm) deep deck is required for spans up to 30 ft (9.15 m), and for spans up to 24 ft (7.3 m), 6 in. (152 mm) deep steel deck is adequate. When precast planks are used, shear transfer is achieved by using steel plates cast in the planks and by welding the same to the truss chords.

In the case of a metal deck system, generally adequate shear transfer is obtained by welding of the steel deck to the trusses. The choice of the floor system depends on the geographical location, contractor's preferences, as well as local practice.

Since the trusses are staggered at alternate floors, the equivalent lateral load resisted by each truss is equal to the load on two bays. Hence, floor panels on each side of the truss must transmit half the load to adjacent truss below.

1.7.2.2 Columns

In the staggered truss system, the lateral loads are resisted by the trusses acting as vertical braces. Therefore, lateral loads are transferred into direct tension and compression in the columns. No shear

exists in the columns to create bending in the transverse direction of the building. Thus, the columns can be designed as braced members to resist minor axis buckling, and the strong direction of the column can be used to resist lateral loads in the longitudinal direction. Another aspect of column design that should be considered is the effect of truss deflections in causing excessive weak-axis bending in the columns. The axial compression of the top chord and elongation of the bottom chord introduce bending in columns. This problem can be solved by introducing a camber in the truss by deliberately making the truss bottom chord smaller than the top chord. As an alternate, the connection between the bottom chord and the column can be designed to slip under load conditions. Torquing the bolts after application of dead loads to the truss will limit the bending moments in the columns.

As a second alternative, the connection of the bottom chord can be designed to remain loose, increasing the effective length of the column in the weak axis to 2-stories between the top chords of the trusses at two levels. If none of the above procedures is applicable, then these moments should be provided for in the design of the columns.

1.7.2.3 Trusses

The design of the staggered truss is quite conventional. Because the trusses support relatively large floor areas, the design allows for maximum live load reduction. Additional bending that occurs in the truss chords at corridor openings should be accounted for in the design.

The span-to-depth ratio of the truss is usually in the range of 6:1, resulting in adequate depth for efficient design of top and bottom chords. Usually the panel width of trusses is not a governing criterion. Larger panel lengths with fewer web members decrease fabrication costs and may work out to be more economical. For maximum efficiency, it is preferable to maintain a uniform spacing of trusses. This allows for repetition of typical truss units thus reducing fabrication costs. The staggered truss system is typically considered for buildings from 6 to about 25 stories tall. Its benefits are most apparent in regular buildings with rectangular floor plans.

1.7.3 Seismic Design of Staggered Truss System

Under transverse seismic loads, the Vierendeel panel would be subject to high deformations (much like the similar panel in a special truss moment frame) and would therefore have to be designed to dissipate energy through flexural yielding. The trusses resist transverse shear, overturning forces, and interstory drift; and the floor diaphragm acts as a load path provider between adjacent trusses. The longitudinal lateral force-resisting system is typically a perimeter moment frame or braced frame.

Most staggered truss systems built to date (2010) are in areas of low seismic hazard. Additionally, it should be noted that staggered truss system is not a recognized seismic force-resisting system in ASCE 7-05 Table 12.2-1. It is equivalent to an "undefined structural system."

Therefore, its use requires review of substantiating cyclic test data and analytical studies, or project specific peer review in combination with analytical studies. The seismology committee of Structural Engineers Association of California (SEAOC) recommends against the use of staggered truss system as a seismic force-resisting system for buildings assigned to SDCs C through F.

1.7.3.1 Response of Staggered Truss System to Seismic Loads

Acceptable earthquake performance of a staggered truss system will be limited by the following attributes:

- The lateral and gravity force-resisting systems of the building are one and the same. Every gravity-resisting truss and column is also integral to the transverse lateral force resisting system.
- Long transverse spans limit the ability of the system to redistribute gravity loads in the event of a column failure.

- The ground story is usually much more flexible than the floors above. Customarily, moment frames replace the staggered truss elements at the ground story.
- Diaphragms are critical to the lateral load path of this structural system, transferring relatively high forces between vertical elements. This is especially true at lower stories, where the diaphragm-to-truss connections must transfer nearly the entire base shear from one story to the next.

This last point about the diaphragms might be unique to the staggered truss system. The floor diaphragms are required to participate in the lateral system as fully as the trusses and columns.

Therefore, the use of seismic design factors such as R, C_d, and Ω_0 representative of moment-resisting frames, braced frames, or special truss moment frames, for a staggered truss system would be inappropriate.

In the opinion of the Seismology Committee, adequate testing and/or analysis will be required to adequately address at least the following design and performance issues before using the staggered system for buildings assigned to high SDCs.

- Identification of predictable inelastic mechanisms
- Design forces and deformations in yielding Vierendeel panels and adjacent truss members
- Design forces related to diaphragm–truss interaction, considering expected strength, stiffness, and ductility
- Force distribution and elasticity in precast diaphragms and topping slabs under high in-plane forces
- Force distribution and inelasticity in diaphragms under vertical displacements related to truss deflections and link deformation
- Design of diaphragm-to-truss connections, considering cyclic loading and diaphragm or truss overstrength
- Column design forces and ductility demands, considering dynamic truss–column interaction and sharing of columns by lateral and transverse systems
- Vulnerability of the gravity system to failure of seismic-force-resisting members
- Effects of openings and discontinuities in highly loaded diaphragms
- Disproportionate effects of atypical and irregular building configuration
- Axial and flexural interaction in truss chords, diagonals, and connectors

In addition, because the system load path involves an out-of-plane offset at every floor level, testing must consider the interaction of yielding (and possibly degrading) diaphragms, trusses, and connections as opposed to just the behavior of individual components. Even the testing of an entire truss frame would not capture the essential aspect of shear transfer between adjacent frames.

1.8 INTERACTING SYSTEM OF BRACED AND RIGID FRAMES

Even for relatively short buildings, say in the range of 10–15 stories, unreasonably heavy columns may result if the lateral bracing is confined only to the building service core. This is because the available depth for bracing is usually limited. In addition, the high uplift forces that may occur at the bottom of braced frame columns can present foundation design problems. In such instances, an economical structural solution can be configured by using rigid frames in conjunction with braced frames. Although deep girders are required for rigid frames they are perhaps less objectionable than additional columns from space planning considerations. Often times, architecturally, it may be permissible to use deep spandrels and relatively closely spaced columns at the building perimeter because these columns will not interfere with the space planning and the depth of spandrels need not be shallow for passage of air conditioning ducts. A schematic floor plan of a building using this concept is shown in Figure 1.39.

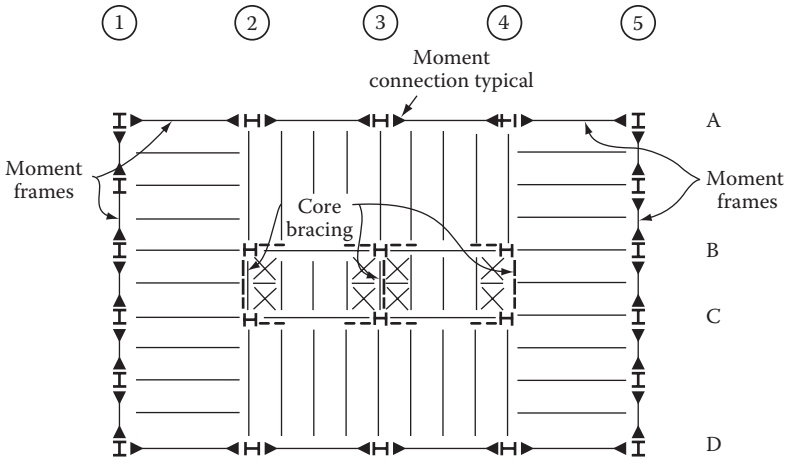


FIGURE 1.39 Building plan showing core braces and perimeter rigid frames.

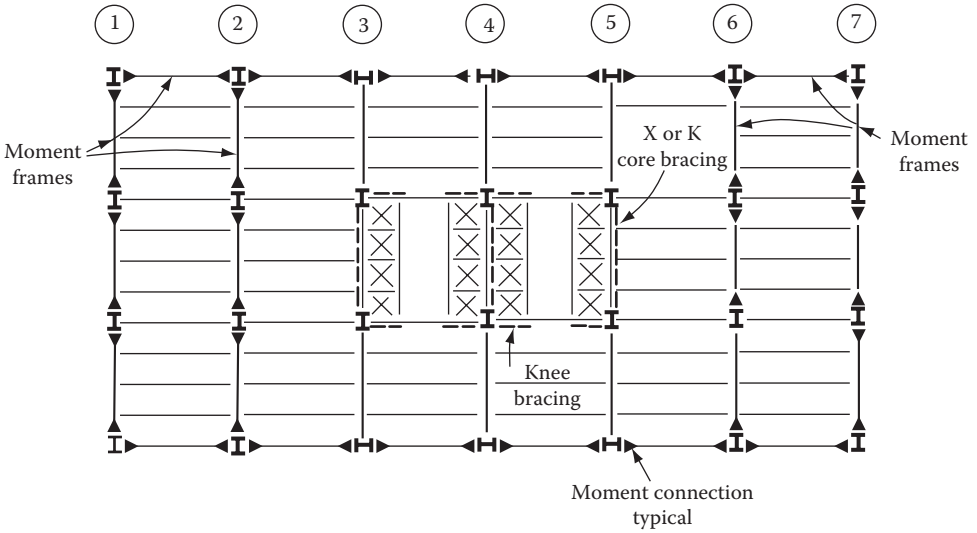


FIGURE 1.40 Building plan showing braced core and exterior/interior rigid frames.

In this scheme, the lateral loads in the North-South direction are resisted by the core bracings on grids 2, 3, and 4, and by the moment frames on grids 1 and 5. For loads in the East-West direction, the lateral resistance is provided by the braced core on grids B and C, working in conjunction with the exterior frames on grids A and D.

As an alternative to exterior frames, a set of interior frames can be used with the core bracing. Such an arrangement is shown in Figure 1.40 in which frames on grid lines 1, 2, 6, and 7 interact with core bracing located on lines 3, 4, and 5. Yet another option is to moment-connect the girders between the braced core and the exterior columns as shown in Figure 1.41. In this case the girders act as outriggers resulting in an overall reduction of bending in the braced frames.

For slender buildings with height-to-width ratios in excess of 6, an interacting system of moment and braced frames becomes uneconomical if braces are confined only to the building core. In such cases, a good structural solution is to spread the bracing to the entire width of the building as shown in Figure 1.42. Depending on architectural preference, the full depth bracing may occur at the

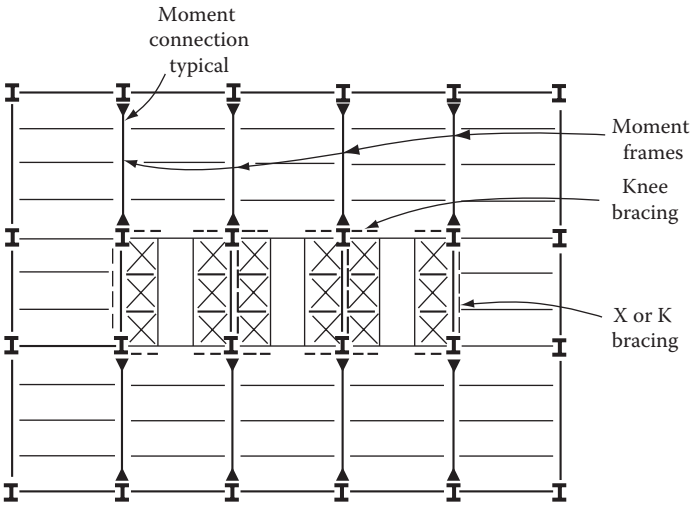


FIGURE 1.41 Building plan showing core braces and outriggers.

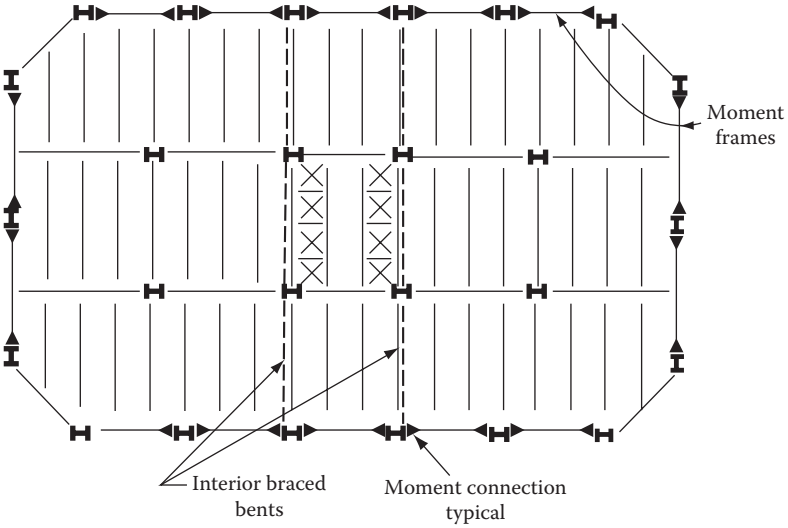


FIGURE 1.42 Building plan showing full-depth interior K braces and exterior moment frames.

building exterior or at the interior. In either case, the braces are stretched out for the full width of the building in the form of a giant brace. The overturning and shear forces in this system are resisted by developing predominantly axial forces in the truss members. A schematic transverse cross section of a building using this system is shown in Figure 1.43. Also shown is a secondary brace the purpose of which is to transfer the lateral loads to the panel points of the giant brace. The diagonals of the K braces running through the interior of the building result in sloping columns whose presence has to be acknowledged architecturally as a trade-off for structural efficiency.

The systems described above are but a sample of moment and braced frame combinations. Any number of their variations can be used either singly or in combination with each other. The extent of interaction between the two can be controlled by varying their stiffness to achieve an economical structural system.

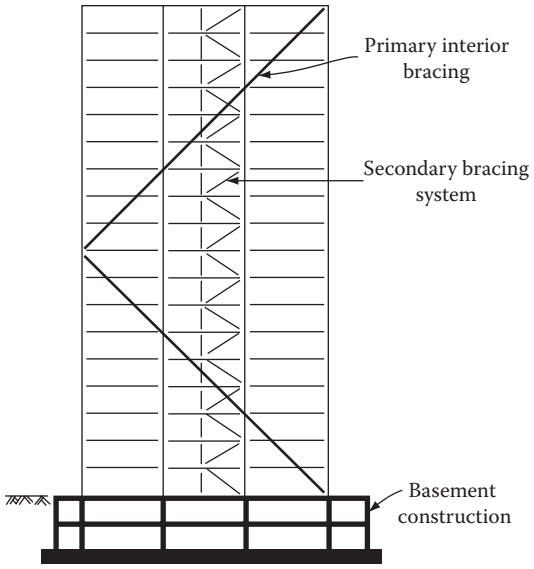


FIGURE 1.43 Schematic section showing primary K brace and secondary core brace.

1.8.1 Behavior

If the lateral deflection patterns of braced and moment frames were similar it would have been reasonable to distribute the lateral loads between the two systems according to their relative stiffness. This is not so because unbraced and braced frames deform with their own characteristic shapes (see Figure 1.44).

Insofar as the lateral load resistance is concerned, rigid and braced frames can be considered as two distinct units. The basis of clarification is the mode of deformation of the unit when subjected to lateral loading. The deflection characteristics of a braced frame are similar to those of a cantilever beam. Near the bottom, the vertical truss is stiff; and therefore, the floor-to-floor deflections will be less than half the values near the top. Near the top, the floor-to-floor deflections increase rapidly mainly due to cumulative effect of chord drift. The column strains at the bottom of the building produce a deflection at the top; and since this same effect occurs at every floor, the resulting drift at the top is cumulative. This type of deflection is often referred to chord drift.

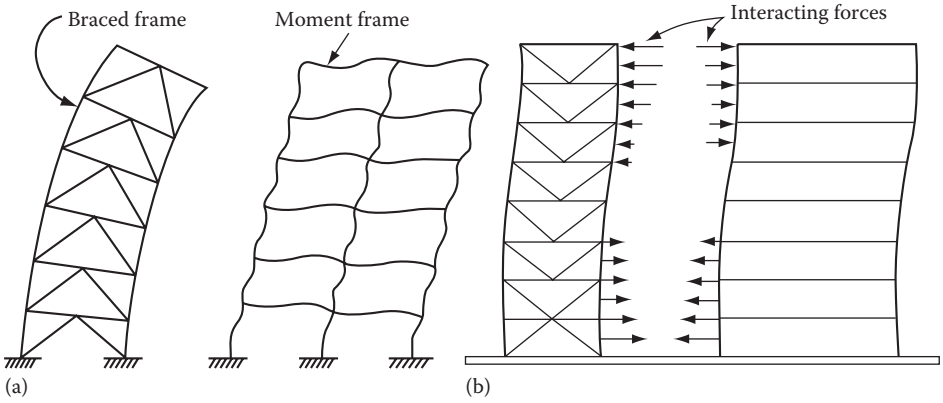


FIGURE 1.44 Interaction between braced and moment frames: (a) characteristic deformation shapes; (b) variation of shear forces resulting from interaction.

Rigid frames deform predominantly in a shear mode. The relative story deflections depend primarily on the magnitude of shear applied at each story level. Although near the bottom, the shear deflections are larger, and near the top smaller as compared to the braced frames, the floor-to-floor deflections can be considered more nearly uniform for the entire height. When the two systems, the braced and rigid frames are connected by rigid floor diaphragms, a nonuniform shear force develops between the two. The resulting interaction helps in extending the range of application to the interacting system to buildings say, up to about 40 stories in height.

Figure 1.44 shows the deformation patterns of a braced and unbraced frames subjected to lateral loads. Also shown therein are the horizontal shear forces between the two frames connected by rigid floor slabs. Observe that the braced frame acts as a vertical cantilever beam, with the slope of the deflection greatest at the top, indicating that in this region the braced frame contributes the least to the lateral stiffness.

The rigid frame has a shear mode deformation, with a slope greater at the base of the structure where the shear is maximum. Because of the distinctly different deflection characteristics of the two, particularly at the top and bottom stories of the building, the frame tends to pull back the brace in the upper portion of the building while pushing it forward in the lower portion. As a result, the frame participates more effectively in the upper portion of the building where lateral shears are relatively less. The braced frame carries most of the shear in the lower portion of the building. Thus, the two systems help each other. The frame tends to reduce the lateral deflection of the brace frame at the top, while the braced frame, in turn, supports the moment frame near the base. A typical variation of horizontal shear carried by each frame is shown in Figure 1.44b, in which the lengths of the arrows conceptually indicate the magnitude of interacting shear forces.

1.9 CORE AND OUTRIGGER SYSTEMS

The structural elegance and efficiency of outriggers, well rooted in history, have also become key elements in the efficient and economic design of high-rise buildings. While outriggers have only been incorporated into high-rise buildings within the last 35 years, the outrigger as a structural element has a much longer history. The sailing ships of the past and present have used outriggers to help resist the wind forces in the sails, making the adoption of tall and slender masts possible. In high-rise buildings, the core can be related to the mast of the ship, the outrigger to the spreader, and the exterior columns to the stays or shrouds. Just as in sailing ships, these outriggers serve to reduce the overturning moment in the core.

In high-rise buildings, this same benefit is realized by a reduction of the base core overturning moments and the associated reduction in potential core uplift forces. The overturning moment resisted through a couple between the windward stay and the mast is similar to the moment transferred to gravity-loaded exterior columns in high-rise buildings.

Modern high-rise buildings almost always incorporate central elevator cores along with column-free floor space between the core and exterior support columns. While this results in greater functional efficiency, it also effectively disconnects the two major structural elements available to resist the overturning forces present in a high-rise building. This uncoupling of the interior core and the perimeter frame reduces the overall resistance of the structure to the overturning forces. Without the benefit of interaction between the two, the resistance is merely equal to the sum of the independent resistances of the individual elements. The incorporation of outriggers in this same system couples these two components and enhances the system's ability to resist overturning forces dramatically.

For buildings of up to 35–40 stories, steel-braced cores have been effectively utilized as the sole load resisting system. These braced systems are effective in resisting the forces and associated deformations due to shear racking since their resistance varies approximately linearly with the building height. However, the resistance that core systems alone provide to the overturning component of drift decreases approximately with the cube of the height. Such core systems

become progressively more inefficient as the height of the building increases. In addition to stiffness limitations, a core system alone can also generate excessive uplift forces in the core structure along with high overturning forces in the building's foundation system. With the system's inability to take advantage of the overall building depth, designing for the resulting uplift forces can be of concern.

The quest for innovative structural schemes is a continuous process, particularly so in the design of high-rise structures. The intention is to limit the lateral drift to acceptable limits without paying a high premium in steel tonnage. The savings in steel tonnage and cost can be dramatic if certain techniques are employed to utilize the full capacities of the structural elements. Various bracing techniques have been developed to this end: this section deals with one such system, particularly applicable to super and ultra tall buildings. The system is often referred to as a *core-and-outrigger* system. The unique feature of this system is the axial stiffness of the perimeter columns is invoked for increasing the resistance to overturning moments. Simply stated, structural form consists of a central braced core with horizontal, "outrigger" trusses connecting the core to the outer columns. The core may be centrally located with outriggers extending on both sides (Figure 1.45a) or it may be located on one side of the building with outriggers extending to the building columns on one side (Figure 1.45b).

When subjected to lateral loads, the column-restrained outriggers resist the rotation of the core, causing the lateral deflections of the building to be smaller than if the free-standing core alone resisted the loading. The result is to increase the effective depth of the structure when it flexes as a vertical cantilever, by inducing tension in the windward columns and compression in the leeward columns.

In addition to those columns located at the ends of the outriggers, it is usual to mobilize other peripheral columns to assist in restraining the outriggers. This is achieved by including a deep spandrel girder, or a "belt truss," around the structure at the levels of the outriggers. Typically the outriggers and belt truss are at least one, and often three-to-four-stories deep. It is also possible to use diagonals extending through several floors to act as outriggers as shown in Figure 1.46. And finally, girders that connect the core to the exterior columns at each floor may be designed as outriggers by using moment connections to the core columns and, if desired, to the exterior columns as well, as shown in Figure 1.47. In all these cases it should be noted that while the outrigger system is effective in increasing the structure's flexural stiffness, it does not increase its resistance to shear, which has to be carried mainly by the core.

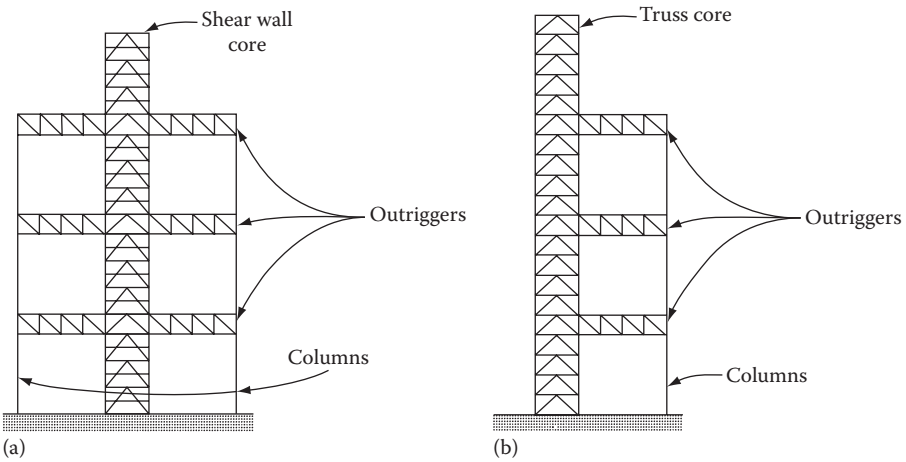


FIGURE 1.45 Core and outrigger system: (a) centrally located core; (b) offset core.

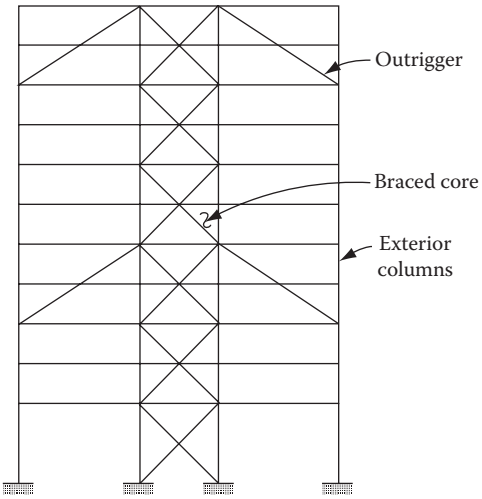


FIGURE 1.46 Outrigger system with diagonal bracing.

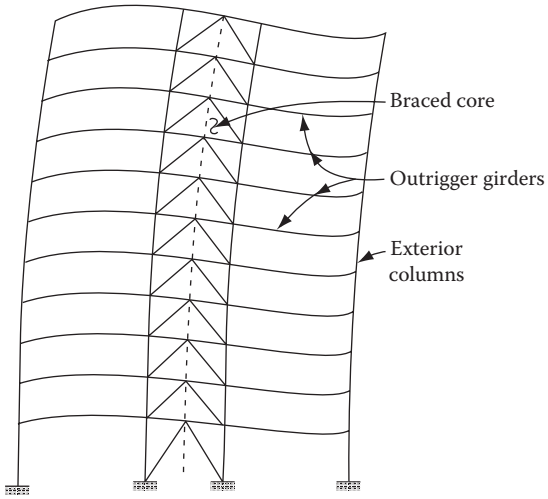


FIGURE 1.47 Moment-connected girders acting as outrigger.

1.9.1 Behavior

One of the most popular methods of providing lateral bracing is to use core bracing with moment frames at the exterior. However, for buildings taller than 40 stories or so, the area of the central core required for mechanical and electrical closets, and vertical transportation is much too small to provide an efficient brace frame system. A method of increasing the efficiency of the system is to use a "cap" or "hat" truss to tie the braced core to the exterior columns using outriggers (see Figure 1.48). The tied columns, in addition to supporting the gravity loads, also assist in resisting the overturning moments. The tie-down action of the cap and outrigger trusses induces a restoring couple at top. This results in a point of inflection in the deflection curve. This reversal in curvature reduces the bending moment in the core, and hence the building drift. The belt truss functioning as a horizontal fascia stiffener mobilizes other exterior columns to take part in restraining the rotation of the cap truss. A general improvement of up to 25%–30% in stiffness can be realized by using a sufficiently stiff belt and outrigger trusses.

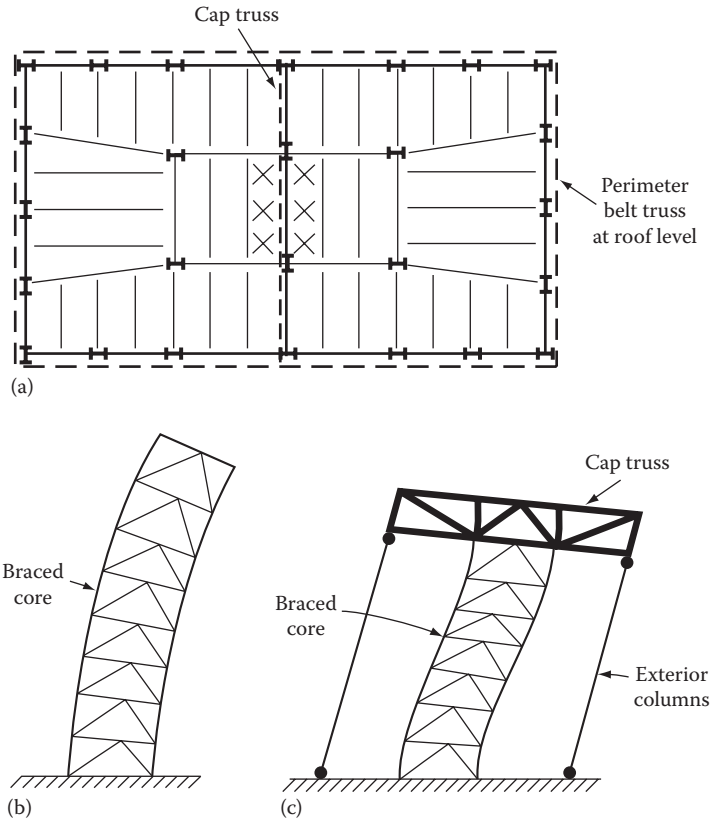


FIGURE 1.48 Building with central core and cap truss: (a) roof plan; (b) lateral deformation; (c) curvature reversal due to tie-down action of cap truss.

The system is also beneficial in equalizing the differential axial strain between the interior and exterior columns that may result from temperature effects, and unequal axial shortening due to gravity load imbalance.

The behavior of the system is best explained by referring to Figure 1.48. Although the building shown therein has a belt truss connecting all exterior columns, we will assume that their effect is equivalent to two columns, one at each end of the cap truss as shown in Figure 1.48c. This idealization is not necessary for analysis, but it keeps the explanation simple. The structure now becomes a tied cantilever with a single degree of redundancy. Its rotation is restrained at the top by the stretching and shortening of the windward and leeward columns. The resulting tensile and compressive forces are analytically equivalent to a restoring couple opposing the rotation of the core. The cap truss may therefore be conceptualized as an equivalent spring located at the top of the cantilever. Its stiffness, defined as a restoring couple due to a unit rotation of the core, may be calculated as follows. Assuming the cap truss is infinitely rigid, the axial compression and tension in the columns is equal to the product of the rotation of core and their distance from the center of the core. If the columns are located at a distance of $d/2$ from the center of the core, the axial deformation of the columns is equal to $\theta \times \frac{d}{2}$ where θ is the rotation of the core and d is the distance between the exterior columns. Since by our definition, for $\theta = 1$, the axial deformation is equal to $d/2$ units. The axial load P in the columns corresponding to this deformation is

$$P = \frac{AEd}{2L}$$

where
P is the axial load in the columns
A is the area of columns
E is the modulus of elasticity of the column material
d is the distance between the exterior columns
L is the height of the building

The restoring couple which is the rotational stiffness of the cap truss is given by summation of the product of the axial load in the columns and their distance from the center of core. Using the notation K for the rotational stiffness we get

$$\begin{aligned} K &= \sum Pd_i \\ &= P \times \frac{d}{2} \times 2 \\ &= Pd \end{aligned}$$

The equivalent spring located at the top reduces the deflection of the core by inducing a reverse curvature as shown in Figure 1.48c. The ensuing reduction in drift for a given building geometry is primarily a function of the area of the exterior columns.

Now let us consider the interaction of the core with an outrigger located not at the top but somewhere along the height, for example, at three-quarters the height, mid-height and one-quarter the height. How does the location of the outrigger influence its effectiveness? Is top location the best? What are the implications if it is moved toward the bottom, say to the mid-height of the building? Before we answer these rather intriguing questions, it is useful to study the structural response for a few specific locations of the outrigger, for example, at the top, three-quarters of the height, mid-height and quarter height.

1.9.1.1 Outrigger Located at Top

The conceptual model is shown in Figure 1.49. The rotation compatibility condition at $Z = L$ can be written as

$$\theta_W - \theta_S = \theta_L$$

where
θ_W is the rotation of the cantilever at $Z = L$ due to uniform lateral load W, in radians
θ_S is the rotation due to spring restraint located at $Z = L$, in radians. The negative sign indicates that the rotation of the cantilever due to the spring stiffness acts in a direction opposite of the rotation due to external load
θ_L is the final rotation of the cantilever at $Z = L$, in radians

For a cantilever with uniform moment of inertia I, and modulus of elasticity E subject to uniform horizontal load W,

$$\theta_W = \frac{WL^3}{6EI}$$

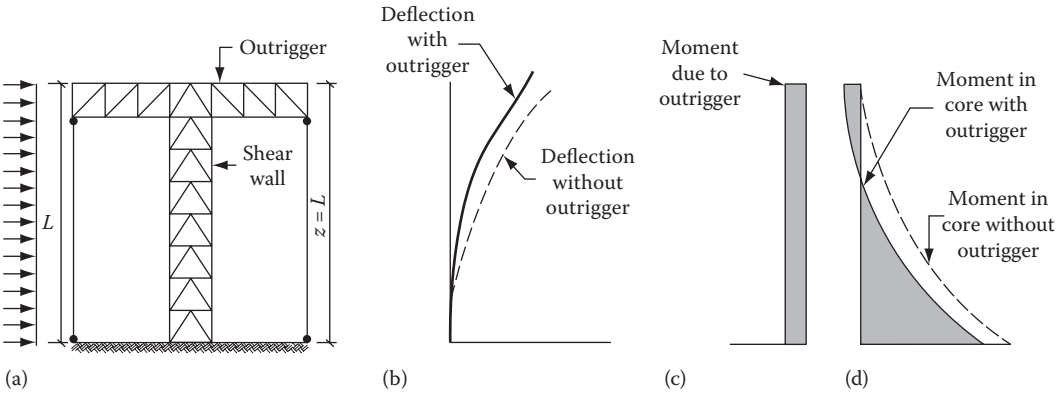


FIGURE 1.49 Outrigger located at top: (a) analytical model; (b) deflected shape; (c and d) moment diagrams.

If M_1 and K_1 represent the moment and stiffness of the spring located at $Z = L$, the rotational compatibility equation can be written thus

$$\frac{WL^3}{6EI} - \frac{M_1 L}{EI} = \frac{M_1}{K_1}$$

and

$$M_1 = \frac{WL^3}{1/K_1 + L/EI}$$

The resulting drift Δ_1 at the building top can be obtained by superposing the deflection of the cantilever due to external uniform load W and the deflection due to the moment induced by the spring; thus,

$$\Delta_1 = \Delta_{\text{load}} - \Delta_{\text{spring}}$$

$$= \frac{WL^4}{8EI} - \frac{M_1 L^2}{2EI}$$

$$= \frac{L^2}{2EI}\left(\frac{WL^2}{4} - M_1\right)$$

1.9.1.2 Outrigger Located at Three-Quarter Height from Bottom

The interaction effect between the central core and the exterior columns is explained with reference to Figure 1.50. The general expression for lateral deflection y of a cantilever subjected to a uniform lateral load is given by

$$y = \frac{W}{24EI}(x^4 - 4L^3 x + 3L^4)$$

Note that x is measured from the top.

Differentiating with respect to x, the general expression for slope of the cantilever is given by

$$\frac{dy}{dx} = \frac{W}{6EI}(x^3 - L^3)$$

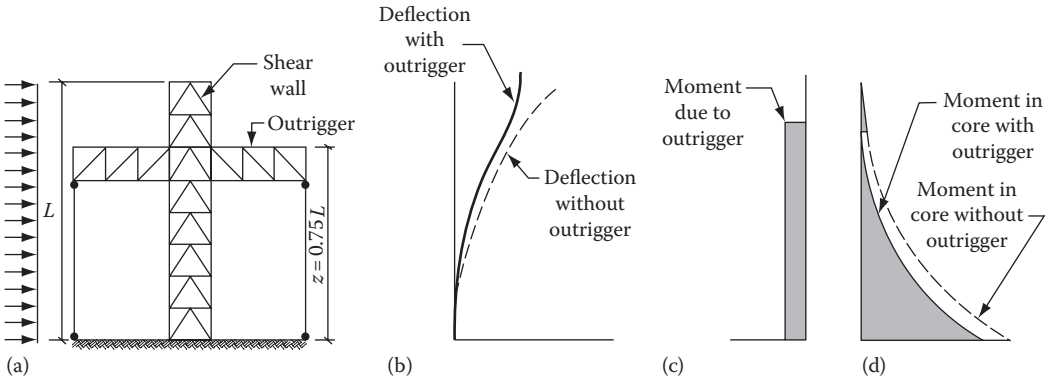


FIGURE 1.50 Outrigger located at quarter-height from top: (a) analytical model; (b) deflected shape; (c and d) moment diagrams.

The slope at the spring location is given by substituting $Z = 3L/4$, that is, $x = L/4$ in Equation (1.??). Thus,

$$\frac{dy}{dx}\left(\text{at } z = \frac{3L}{4}\right) = \frac{W}{6EI}\left(\frac{L^3}{64} - L^3\right)$$

$$= \frac{WL^3}{6EI} \times \frac{63}{64}$$

Using the notation M_2 and K_2 to represent the moment stiffness of spring at $Z = 3L/4$, the compatibility equation at location Z can be written thus

$$\frac{WL^3}{6EI}\left(\frac{63}{64}\right) - \frac{M_2}{EI}\left(\frac{3L}{4}\right) = \frac{M_2}{K_2}$$

Nothing that $K_2 = 4K_1/3$, the expression for M_2 can be written thus

$$M_2 = \left\{\frac{WL^3/6EI}{1/K_1 + L/EI}\right\}\frac{63/64}{3/4} = \left\{\frac{WL^3/6EI}{1/K_1 + L/EI}\right\} 1.31$$

Nothing that the terms in the parenthesis represent M_1, Equation (??) can be expressed in terms of M_1

$$M_3 = 1.75M_1$$

The drift is given by the equation

$$\Delta_2 = \frac{WL^4}{8EI} - \frac{M_3 L}{2EI}\left(L - \frac{L}{4}\right)$$

or

$$\Delta_2 = \frac{L^2}{2EI}\left(\frac{WL^2}{4} - 1.23M_1\right)$$

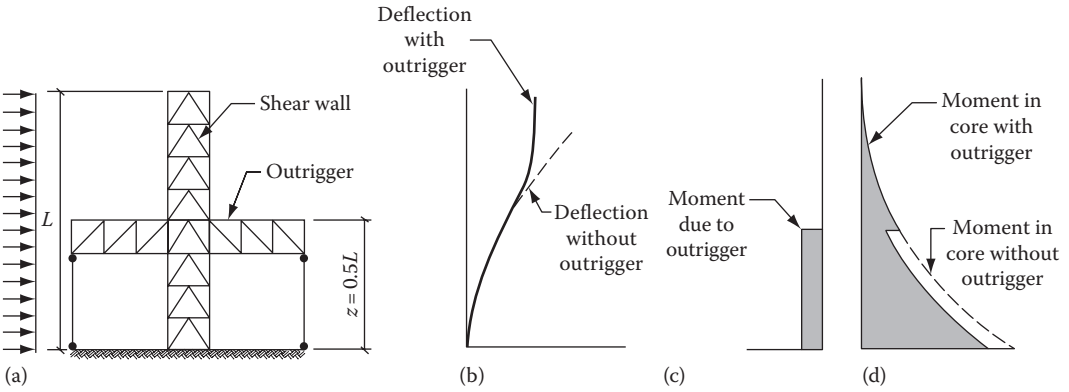


FIGURE 1.51 Outrigger located at mid-height: (a) analytical model; (b) deflected shape; (c and d) moment diagrams.

1.9.1.3 Outrigger at Mid-Height

We use Figure 1.51 to study this condition. The rotation at $Z = L/2$ due to external load W can be shown to be equal to $7WL^3/48EI$, giving the rotation compatibility equation

$$\frac{7WL^3}{48EI} - \frac{M_3 L}{2EI} = \frac{M_3}{K_3}$$

where M_3 and K_3 represent the moment and stiffness of the spring at $Z = L/2$. Noting that $K_3 = 2K_1$, the expression for M_3 works out as

$$M_3 = \left\{ \frac{WL^3/6EI}{1/K_1 + L/EI} \right\} \times \frac{7}{4}$$

Since the expression in the parentheses is equal to M_1, M_3 can be expressed in terms of M_1

$$M_3 = 1.75M_1$$

The drift is given by the equation

$$\Delta_3 = \frac{WL^4}{8EI} - \frac{M_3 L}{2EI}\left(L - \frac{L}{4}\right)$$

or

$$\Delta_3 = \frac{L^2}{2EI}\left(\frac{WL^2}{4} - 1.31M_1\right)$$

1.9.1.4 Outriggers at Quarter-Height from Bottom

Referring to Figure 1.52, the rotation at $Z = L/4$ due to uniform lateral load can be shown to be equal to $\frac{WL^3}{6EI}\left[\frac{37}{64}\right]$ giving the rotation compatibility equation

$$\frac{WL^3}{6EI}\left(\frac{37}{64}\right) - \frac{M_4 L}{4EI} = \frac{M_4}{K_4}$$

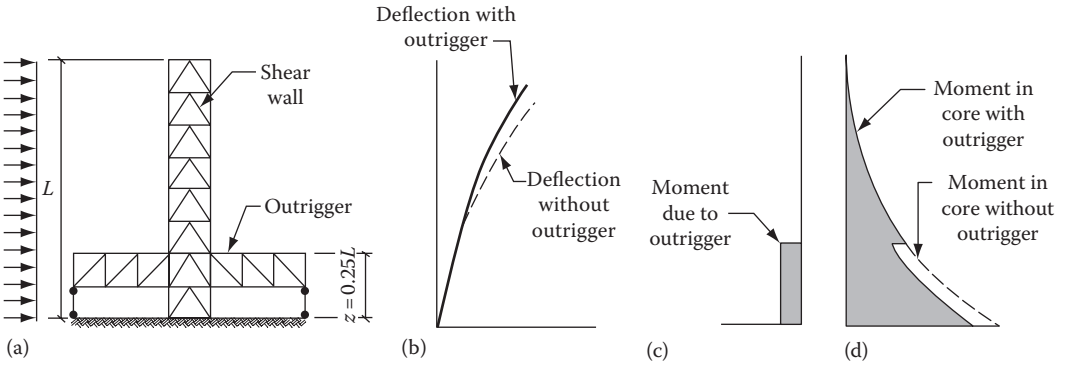


FIGURE 1.52 Outrigger located at quarter height from bottom: (a) analytical model; (b) deflected shape; (c and d) moment diagrams.

where M_4 and K_4 represent the moment and stiffness of the spring at $Z = L/4$. Noting that $K_4 = 4K_1$, M_4 in the above equation can be expressed in terms of M_1.

$$M_4 = 2.3M_1$$

The drift for this case is given by the expression

$$\Delta_4 = \frac{WL^4}{8EI} - \frac{M_4 L}{4EI}\left(L - \frac{L}{8}\right)$$

or

$$\Delta_4 = \frac{L^2}{2EI}\left(\frac{WL^2}{4} - M_1\right)$$

The equations for Δ_1, Δ_2, Δ_3 and Δ_4 give the building drift for the four selected locations of the belt and outrigger trusses, $z = L$, $z = \frac{3L}{4}$, $z = \frac{L}{z}$ and $z = \frac{L}{4}$.

The value of K_1 which corresponds to stiffness of the spring when it is located at $Z = L$ can be derived as follows. A unit rotation given to the core at the top results in extension and compression of all perimeter columns, the magnitudes of which are given by their respective distances from the center of gravity of the core. The resulting force multiplied by the lever arm gives the value for stiffness K_1. Thus, if p_1 is measured in kips and the distance in feet, K_1 has units of kip feet. The force p in each exterior column is given by the relation $p = AE\delta/L$; since by definition δ corresponds to column extension or compression due to unit rotation of the core, $\delta = d/2$, where d is there distance between the exterior columns.

Therefore,

$$p = \frac{AE}{L}\left(\frac{d}{2}\right)$$

and its contribution to the stiffness K_1 is given by the relation

$$K_i = p_i d$$

$$= \frac{A_i E}{L} \frac{d^2}{2}$$

The total contribution of all exterior columns on the long faces is given by the summation relation

$$K_1 = \sum_{i=1}^{n} K_i = \frac{d^2 E}{2L} \sum_{i=1}^{n} A_i$$

The contribution of the columns on the short faces can be worked out in a similar manner.

1.9.2 Optimum Location of a Single Outrigger

The preceding analysis has indicated that the beneficial action of an outrigger is a function of two distinct characteristics, the stiffness of the equivalent and the magnitude of rotation of the cantilever at the spring location due to external loads. It is obvious that the stiffness of the equivalent spring varies inversely as the distance of the outrigger from the base. For example, it is at minimum when located at the top and a maximum when at the bottom. Because the rotation of a cantilever subject to a uniformly distributed horizontal load varies parabolically from a maximum at top to zero at the bottom, from the point of view of spring stiffness, it is desirable to locate the outrigger at the bottom, whereas from a consideration of rotation, the converse is true. It is obvious that the optimum location is somewhere in between.

The method of analysis for determining the optimum location is based on the following assumptions also shown in Figure 1.53A and 1.53B:

1. The area of the perimeter columns and the moment of inertia of the core are uniform throughout the height.
2. The outrigger and the belt trusses are infinitely rigid and induce only axial forces in the exterior columns.
3. The lateral resistance is provided only by the bending resistance of the core and the tie-down action of the exterior columns.
4. The core is rigidly fixed at the foundation.
5. The rotation of the core due to shear deformation is negotiable.
6. The intensity of lateral load remains constant for the whole height.
7. The structure is linearly elastic.

1.9.2.1 Analysis Outline

Consider Figure 1.53 that shows a single outrigger located at a distance x from the building top. The analysis for determining the optimum truss location consists of the following steps:

1. Calculate the restoring moment M_x due to the rotational restraint of the outrigger located at x.
2. Derive an algebraic equation for the deflection at top due to M_x.
3. Differentiate the equation derived earlier with respect to x and equate it to zero.

This step results in a third-degree polynomial equation, the solution of which yields the optimum location for a single outrigger.

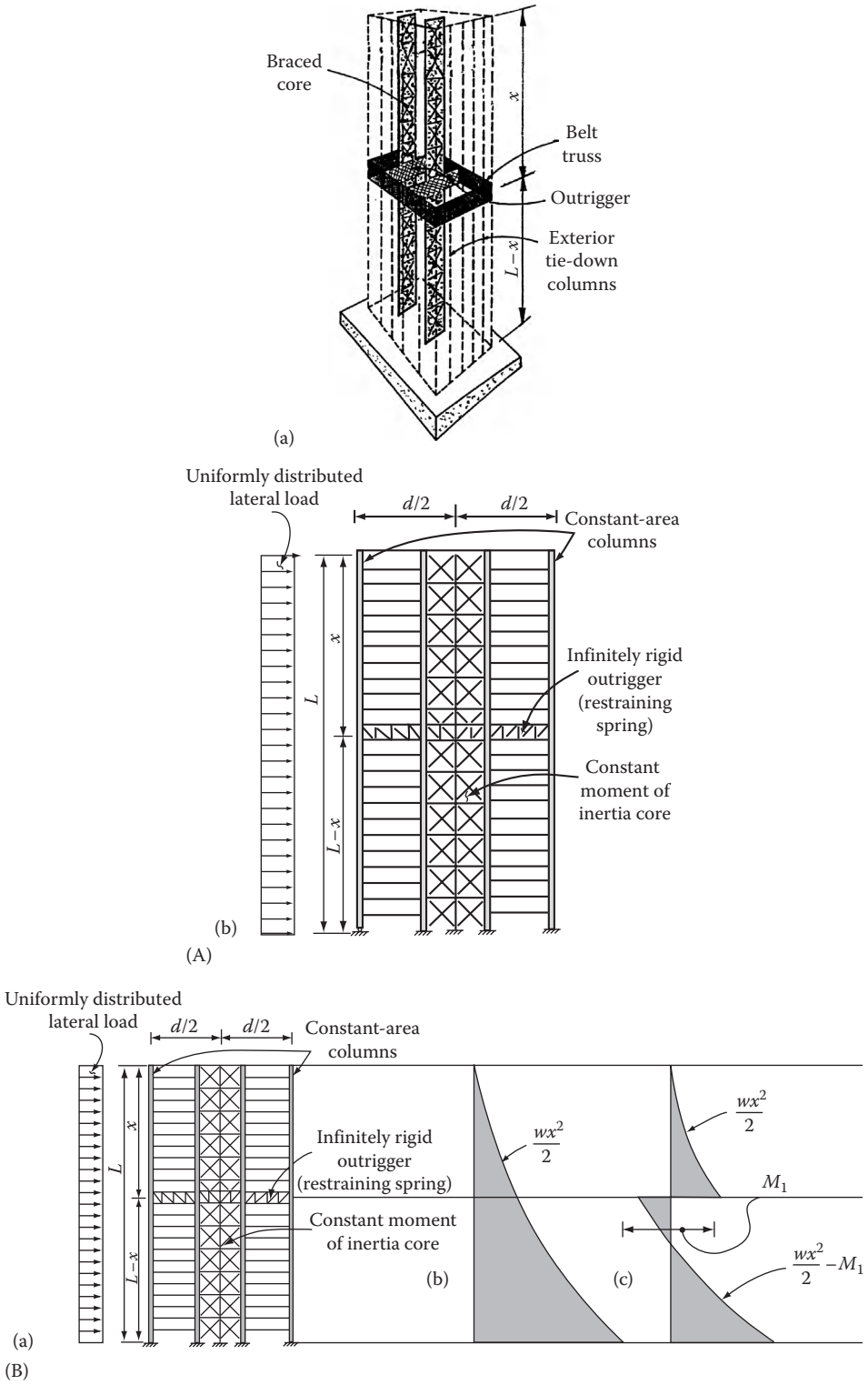


FIGURE 1.53 (A) Idealized analytical model of a single outrigger located at distance x from top: (a) schematics; (b) analysis assumptions. (B) Single outrigger, analysis assumptions: (a) schematic section; (b) moment without outrigger; and (c) moment with outrigger.

1.9.2.2 Detail Analysis

Referring to Figure 1.53, the rotation θ of the cantilever at the distance x from the top, due to a uniformly distributed load W is given by the relation:

$$\theta = \frac{W}{EI}(x^3 - L^3)$$

The rotation at the top due to the restoring couple M_x is given by the relation

$$\theta = \frac{M_x}{EI}(L - x)$$

The compatibility relation at x is given by

$$\frac{W}{6EI}(x^3 - L^3) - \frac{M_x}{EI}(L - x) = \frac{M_x}{K_x}$$

where

W is the intensity of the wind load per unit height of the structure
M_x is the restoring moment due to outrigger restraint
K_x is the spring stiffness at x equal to $\frac{AE}{L - x} \times \frac{d^2}{2}$
E is the modulus of elasticity of the core
I is the moment of inertia of the core
A is the area of the perimeter columns
L is the height of the building
x is the location of truss measured from the top
d is the out-to-out distance of columns

Next, obtain the deflection at the top of the structure due to M_x:

$$Y_M = \frac{M_x(L - x)(L + x)}{2EI}$$

From our definition, the optimum location of the belt truss is that location for which the deflection Y_M is a maximum. This is obtained by differentiating Equation (??) with respect to x and equating to zero.

Thus,

$$\frac{d}{dx}\left[\frac{W(x^3 - L^3)(L + X)}{12(EI)^2(1/AE + 1/EI)}\right] = 0$$

$$4x^2 + 3x^2L - L^3 = 0$$

This cubic equation has a single positive root, $x = 0.445L$. The solution is by trial and error. Therefore, to minimize drift, the outrigger must be located at a distance $x = 0.455L$ from the top or, say, approximately at mid-height of the building.

1.9.2.3 Computer Analysis

In the preceding discussion, several assumptions were necessary to simplify the problem for hand calculations. However, in a practical building, many of these assumptions are rarely satisfied.

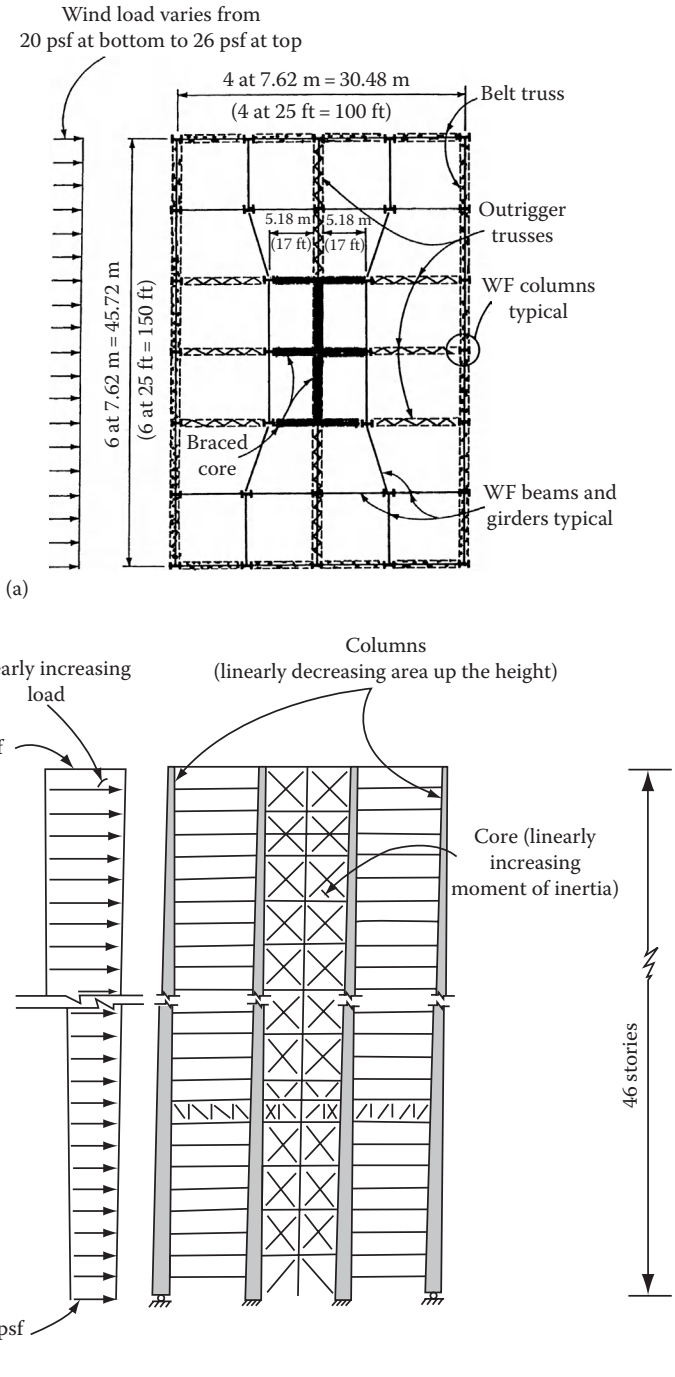


FIGURE 1.54 Improved model of a single outrigger located at distance x from top: (a) building plan and (b) schematic elevation.

For example:

- The lateral load does not remain constant up the building height. It varies in a trapezoidal or triangular manner, the former representative of wind loads and the latter, seismic loads.
- The cross-sectional areas of both the exterior columns and interior core columns typically reduce up the building height. A linear variation is perhaps more representative of a practical building column, particularly so for a tall building of say 40-plus stories.
- As the areas of core columns decrease up the height, so does its moment of inertia up the height.

Incorporating the aforementioned modifications aligns the analytical model somewhat closer to a practical structure, but renders hand calculations all but impossible. Therefore, a computer-assisted analysis has been performed on a representative 46-story steel building using the modified assumptions previously mentioned. A schematic plan of the building, and an elevation of the improved model, subject to varying lateral loads are shown in Figure 1.54. The lateral deflections at the building top are shown in a graphical format in Figure 1.55 for various outrigger locations.

The deflections shown are in a nondimensional format and are relative to that of the core without the outrigger. Thus, the vertical ordinate with a value of unity at the extreme right of Figure 1.55 is the deflection of the building without the restraining effects of the outrigger. The deflections that include the effect of the outriggers are shown in curve "S." This curve is obtained by successively varying the outrigger location starting at the very top and progressively lowering its location in single-story increments, down through the building height.

It is seen that lowering the outrigger down from the top location decreases the building drift progressively until the outrigger reaches level 26. Moving it either above or below this

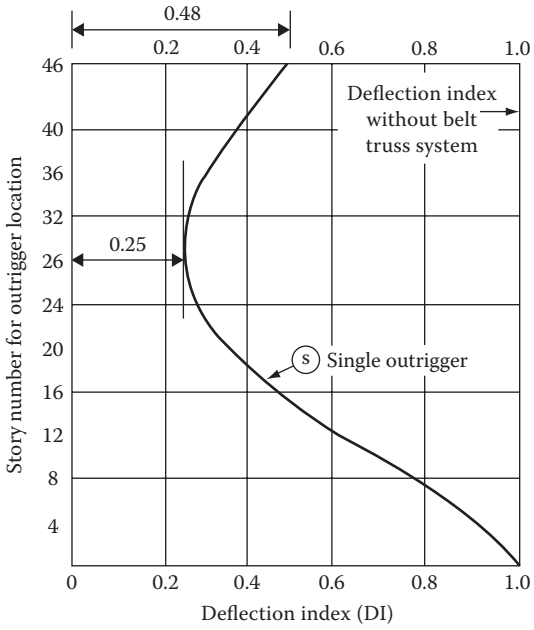


FIGURE 1.55 Deflection index DI, versus outrigger level, *Note*: $\text{DI} = \dfrac{\text{Deflection at top without outrigger}}{\text{Deflection at top with outrigger}}$.

"optimum location" only reduces its efficiency. Observe that this level is at distance of 0.435*L* from the top, very close to the optimum location of $x = 0.455L$ derived earlier in Section 1.9.2.1 for the building with uniform characteristics. Furthermore, it can be seen from Figure 1.55 that the efficiency of the outrigger placed at mid-height, that is, at level 23, is very close to that when it is at the optimum location. Therefore, as a rule of thumb, the optimum location for a single outrigger may be considered at mid-height.

Observe that when the outrigger is at the top, the building drift is reduced to nearly half the deflection of the unrestrained core. A rather impressive reduction indeed, but what is more important is that the deflection continues to reduce as the outrigger is lowered from level 46 downward. The deflection parameter reaches a minimum value of 0.25 as shown in Figure 1.55 when the outrigger is placed at the optimum location, level 26. Further lowering of the outrigger will not reduce the drift, but increases it. Its beneficial effect vanishes to nearly nothing when placed very close to the bottom of the building, say, at level 2 of the example building.

1.9.2.4 Conclusions

- Given a choice, the best location for a single outrigger is at mid-height of the building.
- An outrigger placed at top, acting as a cap or hat truss is about 50% less efficient than that placed at mid-height. However, in many practical situations, it may be more permissible to locate the outrigger at the building top. Therefore, although not as efficient as when at mid-height, the benefits of a cap truss are nevertheless quite impressive, resulting in up to a 50% reduction in building drift.

To better appreciate the benefits of belt and outriggers, let us revisit the graph shown in Figure 1.55. Instead of the nondimensional deflection parameter, DI, we will use drift index criteria, Δ_t/H, to reveal the efficiency of the system.

The building analyzed is a 600 ft tall, 46-story building with a floor-to-floor height of approximately 13 ft. Let us assume, for purposes explaining the graphs, that the building without the outrigger and belt truss, experiences a deflection 55 in. at top. The drift index Δ_t/H is equal to $\frac{55}{12 \times 600} = \frac{1}{131}$ which is quite high compared to the typical target value of $\frac{1}{400}$ commonly used in practice. By introducing a belt and an outrigger at top (which we referred to earlier as cap truss system), we see from Figure 1.55 that the deflection index drops down to 0.48. Thus, the deflection at the top is reduced to $55 \times 0.48 = 26.4$ in. The reduction is quite impressive but the deflection index equal to $\frac{26.4}{600 \times 12} = \frac{1}{273}$ is still unacceptable.

Let us examine how we can diminish the deflection, not by adding more steel to the building but by changing the location of belt and outriggers. For this purpose, we shift their location to level 40 at first, and then to level 32. From Figure 1.55, it is seen that the DI is reduced to 0.34 when the trusses are located at level 40, and to 0.27 when located at level 32. Again from Figure 1.55, it is seen that the deflection decreases progressively reaching its optimum for truss location at level 26. The DI is 0.25 with the building drift equal to $0.25 \times 55 = 13.75$ in. The drift index $\Delta t/H = \frac{13.75}{600 \times 12} = \frac{1}{524}$, well within the target drift = $\frac{1}{400}$.

Observe that we are able to tame the building deflection from an intimidating 55 in. at top, to a mere 13.5 in. Need we say more about the efficiency of the system?

1.9.3 Optimum Locations of Two Outriggers

In the preceding conceptual analyses, only one compatibility equation was necessary because the one-outrigger structure is once-redundant. On the other hand, a two-outrigger structure is

twice-redundant, requiring a solution of two compatibility equations. To seek a solution to the problem, we proceed as before assuming the sectional areas of the exterior columns and the moment of inertia of the core decrease linearly up the height. A trapezoidal distribution is assumed, as before, for the lateral load. Schematics of conceptual analytical model behavior of the structural system are shown in Figures 1.56 through 1.58.

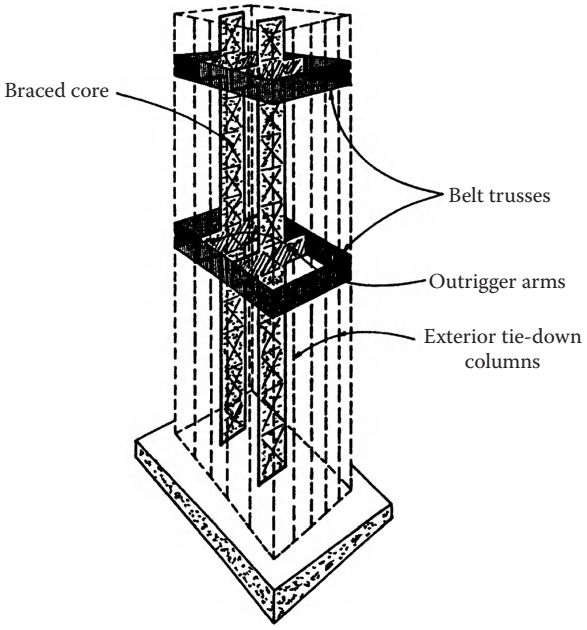


FIGURE 1.56 Building schematics with two outriggers.

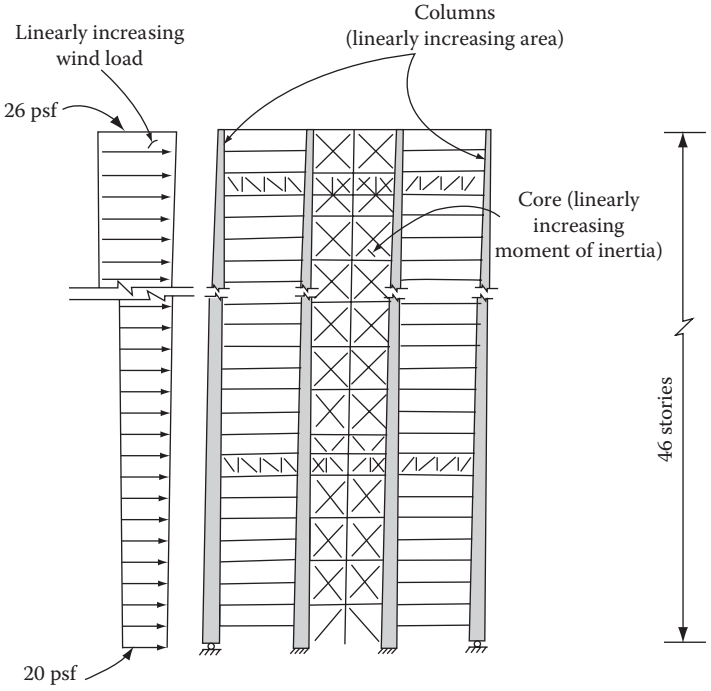


FIGURE 1.57 Analytical model with two outriggers and belt trusses.

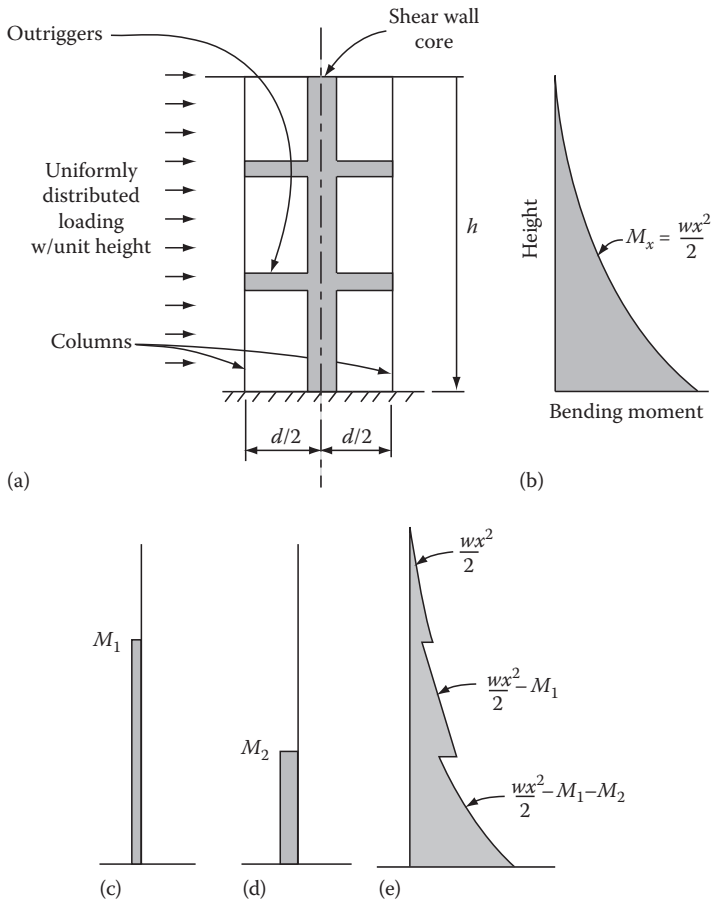


FIGURE 1.58 Conceptual method of analysis for two outrigger system: (a) two outrigger structure; (b) external moment diagram; (c) M_1 moment diagram; (d) M_2 moment diagram; (e) core resultant moment diagram.

The method of analysis for calculating the deflections at the top is similar to that used for the single outrigger. The moments at the outrigger locations are chosen as the unknown arbitrary constants M_1 and M_2, as shown in Figure 1.58. The structure is then rendered statically determinate by removing the rotational restraints at the outrigger locations. Next, the compatibility equations for the rotations at the truss locations are set up and solved simultaneously to obtain the values to M_1 and M_2. The final deflection at the top is obtained by a superposition of the deflection due to the external load and a counteracting deflection due to the moments M_1 and M_2. The resulting deflections are summarized in Figure 1.59. The building deflection at top for a given location of the two outriggers is presented for three conditions by assuming that the lateral loads are resisted by (1) core alone, (2) core acting together with a single outrigger, and (3) core acting in conjunction with two outriggers.

As before, the vertical ordinate shown with a value of unity at the extreme right of Figure 1.59 is the deflection index at the top derived by neglecting the restraining effect of the outriggers. The resistance is provided only by the cantilever action of the braced core alone. Curve S represents the top deflection of the core restrained by a single outrigger located anywhere up the height of the structure. The curves designated as 4, 8, …, 46 represent the deflections at the top for two outriggers located anywhere up the height of the structure. To plot each curve, the location of the upper outrigger was considered fixed in relation to the building height, while the location of the lower outrigger was moved in single-story increments, starting from the floor immediately below the top outrigger.

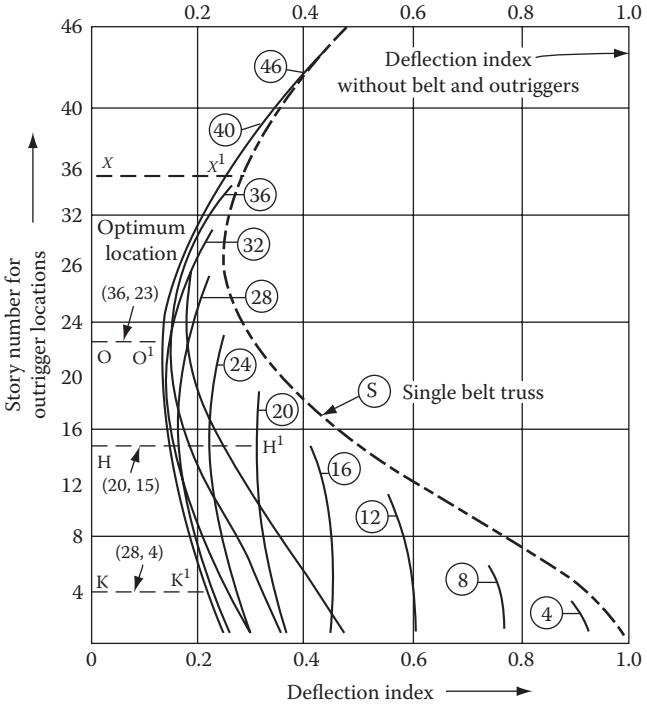


FIGURE 1.59 Drift index versus belt and outrigger location.

The number designations of the curves represent the floor number at which the upper outrigger is located. The second outrigger location is shown by story levels on the vertical axis. The horizontal distance between the curves and the vertical axis is the relative building drift for the particular combination of outrigger locations given by the curve designation and the story level. For example, let us assume that the relative deflection at the top is desired for a combination (20, 15), the numbers 20 and 15 representing the floors at which the upper and lower outriggers are located. To find the deflection index for this particular combination, the procedure is to select the curve with the designation 20, go down the vertical axis to level 15, and draw a horizontal line from this level to curve 20. The required relative top deflection is the horizontal distance between level 15 and curve 20 (distance HH^1 in Figure 1.57). Similarly, the length KK^1 gives the relative deflection at the top for the combination (28, 4). It is seen from Figure 1.58 that the relative location of the outriggers has a significant effect on controlling the drift. Furthermore, it is evident that a deflection very nearly equal to the optimum can be achieved by placing the trusses at levels other than at their optimum locations. For the example building, a relative deflection of 0.15, which differs negligibly from the optimum value of 0.13, is achieved by placing the outriggers at (40, 23), (32, 33), etc.

1.9.3.1 Recommendations for Optimum Locations

Based on the conceptual study presented thus far, the following recommendations are made for optimum locations of outriggers.

- The optimum location for a single outrigger is, perhaps unexpectedly, not at the top. The reduction in the drift with the outriggers located at top is about 50%, as compared to a maximum of 75% achievable by placing it at approximately mid-height. However, since other architectural requirements take precedence in a structural layout, the benefits of placing a truss at the top are still worth pursuing.

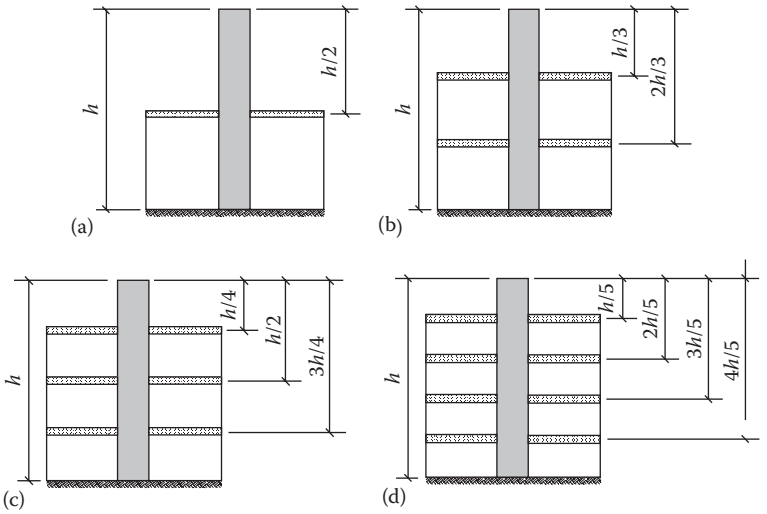


FIGURE 1.60 Optimum location of outriggers: (a) single outrigger; (b) two outriggers; (c) three outriggers; and (d) four outriggers.

- A two-outrigger structure offers more options in the placements of outriggers. Reductions in building deflections close to the optimum results may be achieved with outriggers placed at levels entirely different from the optimum locations. Thus, the engineer and architect have some leeway in choosing the outrigger locations. However, as a rule of thumb, the optimum location for a two-way outrigger structure is at one-third and two-third heights. And for a three-rigger system, they should be at the one-quarter, one-half, and three-quarter heights, and so on. Therefore, for the optimum performance on an outrigger structure, the outriggers should be placed at $(1/n + 1)$, $(3/n + 1)$, $(4/n + 1)$, …, $(n/n + 1)$ height locations. For example, in an 80-story building with four outriggers (i.e., $n = 4$), the optimum locations are at the 16th, 32nd, and 64th levels. A summary of the recommendations is shown in Figure 1.60.

1.9.4 Vulnerability of Core and Outrigger System to Progressive Collapse

Structural redundancy and robust connections are essential factors in providing resilience against progressive collapse. Progressive collapse occurs when the sudden loss of a critical element such a building column initiates a chain reaction disproportionate to the loss of the element, resulting in partial or total collapse of the structure. Redundancy, defined as the ability of the structural system to redistribute and support its gravity loads after loss of a critical member, is perhaps one of the best ways of limiting partial or total progressive collapse.

The vulnerability of high-rise buildings to progressive collapse after a blast event at a street level has become a growing concern, particularly after the collapse of the Alfred P. Murrah Federal Building in 1995.

Design of buildings for conventional loads such as wind and seismic, provides a certain level of resistance to progressive collapse. However, systems in which ductile detailing is not provided as, for example, for buildings in SDC A or B, the inherent resilience to progressive collapse would be minimal.

For a conceptional evaluation of the inherent resilience of a belt and outrigger system, consider the schematics of a high rise shown in Figure 1.61. It can be seen from the figure that once a perimeter column is lost, the vertical load in that column will have two back-up load paths. First, it will redistribute itself upward along the column line as a tension force until it reaches the level of the belt truss. At this level, a majority of the load will transfer as compressive forces through the belt

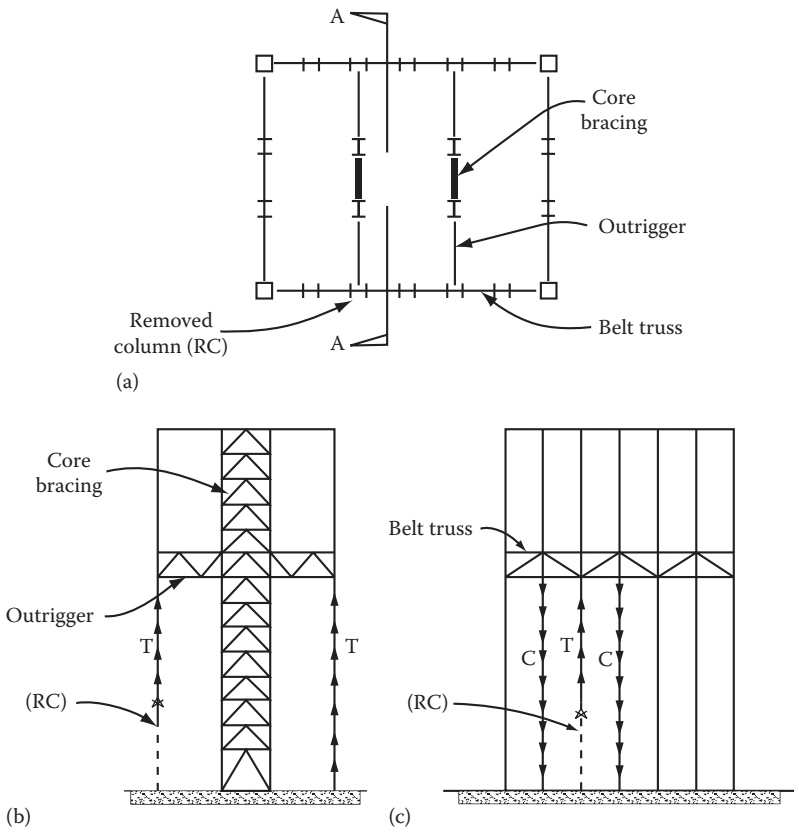


FIGURE 1.61 Vulnerability of core and outrigger system to progressive collapse: (a) plan; (b) schematic section; (c) elevation.

truss to adjacent columns, principally to those immediately next to the removed column. In the secondary load path, the outriggers will transfer the tension load to the building core, acting as cantilevers. Observe that the exterior columns at the opposite side of the core will also experience tension. Because of the aforementioned alternate load paths, damage in a belt and outrigger system is likely to be minimal. Adequate tension splices would be necessary to redistribute the vertical loads in tension.

1.9.5 Offset Outriggers

A variation of the traditional outrigger system is the *offset outrigger* concept proposed by Brian Stafford Smith in 1996. In the conventional outrigger system, the outrigger trusses are connected directly to the core columns and to columns located directly outboard of the core at the building face. The manner in which the outboard columns resist part of the overturning moments caused by lateral loads, as explained previously, is as follows.

The outrigger trusses that are connected to the core and outboard columns restrain rotation of the core and convert part of the moment in the core into a vertical component at the outboard columns. Elongation and shortening of the columns and the flexibility of the outrigger itself will permit some rotation of the braced core at the outrigger locations. In most designs, however, the rotation is so small that the core undergoes reverse curvature below the outrigger.

In an offset outrigger concept, a similar transfer of overturning moment occurs from the core to the exterior columns, but without a direct connection between the outriggers and the core.

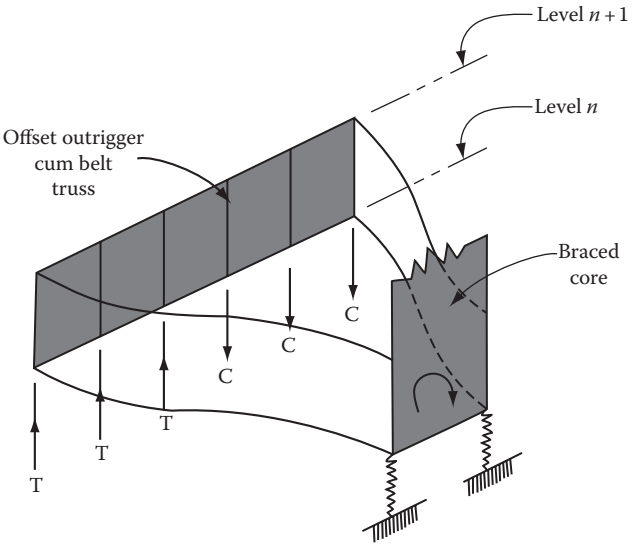


FIGURE 1.62 Schematic force–transfer in offset outriggers.

The basic idea is to use the in-plane stiffness of the floor diaphragms to transfer moment from the plane of the braced core to a lateral system that is offset from the core. The lateral system such as a belt truss located at the building fascia, converts the horizontal couple into vertical couple resulting in axial compression and tension in the columns below the belt truss.

The structural response is very similar to that in a building with traditional outriggers. The only difference is that in the offset system we have succeeded in moving the resistance to a location that is almost always architecturally preferred. In a manner of speaking, we have achieved the same structural response as in traditional outrigger without using outriggers. The belt truss located at the building exterior acts as a vertical outrigger. Because the load path for lateral loads has to jump from the braced core to the perimeter belt truss via floor diaphragms, the system's efficiency is considerably less than in conventional outriggers connected directly to the core. The in-plane stiffness of the floors that transfer horizontal forces from the core to the perimeter belt truss plays a significant part in determining the system's efficiency. Unless the stiffness is beefed up quite considerably, you won't get much benefit from this concept.

Schematics of force transfer using offset outriggers are shown in Figure 1.62.

1.9.6 Example Projects

Figure 1.63a and b shows photographs of The First Wisconsin Center, a 42-story, 1.3 million square foot (120,770 m^2) bank and office building that utilizes the concept of belt and outrigger system for lateral bracing. The building rises 601 ft (183 m) from a two-level glass enclosed plaza and sports three belt trusses located at the bottom, middle, and top of the building. The belt truss at the bottom serves also as a transfer truss, eliminating every other exterior column below that level. Outrigger trusses are used at the top and middle of the building to engage the braced core to the exterior columns. Mechanical levels are located at the truss floors. The architecture and engineering for the project is by the Chicago Office of Skidmore, Owings & Merrill.

As a second example, Figure 1.64 shows a schematic partial elevation of wind bracing system for a building in Houston, Texas called One Houston Center. The building consisting of 48 stories, rises to a height of 681 ft (207.5 m) above grade. To minimize the building drift, 2-story deep outrigger trusses are used to tie the perimeter columns to the K-braced core. Three outrigger trusses on each side of the core occur between the 33rd and 35th levels. The use of these outrigger trusses helped in

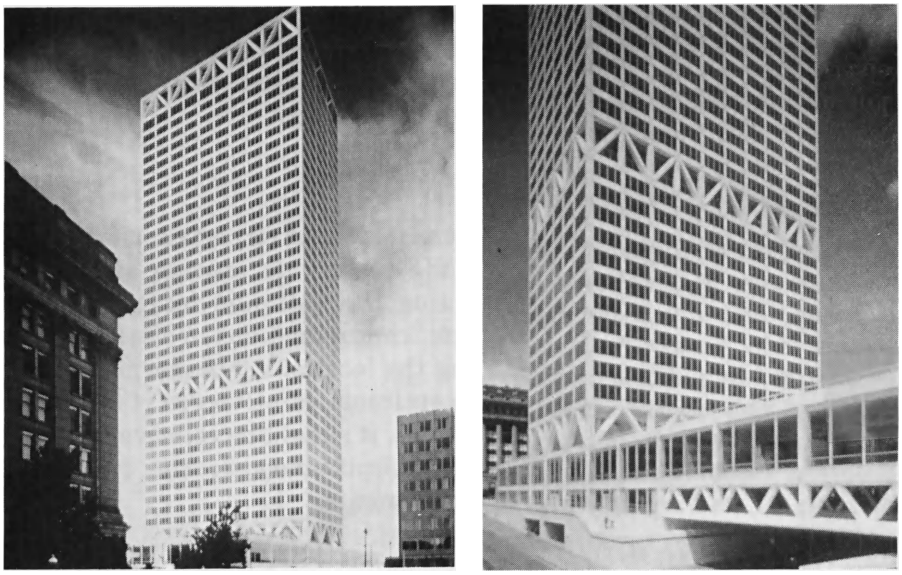


FIGURE 1.63 First Wisconsin Center, Milwaukee.

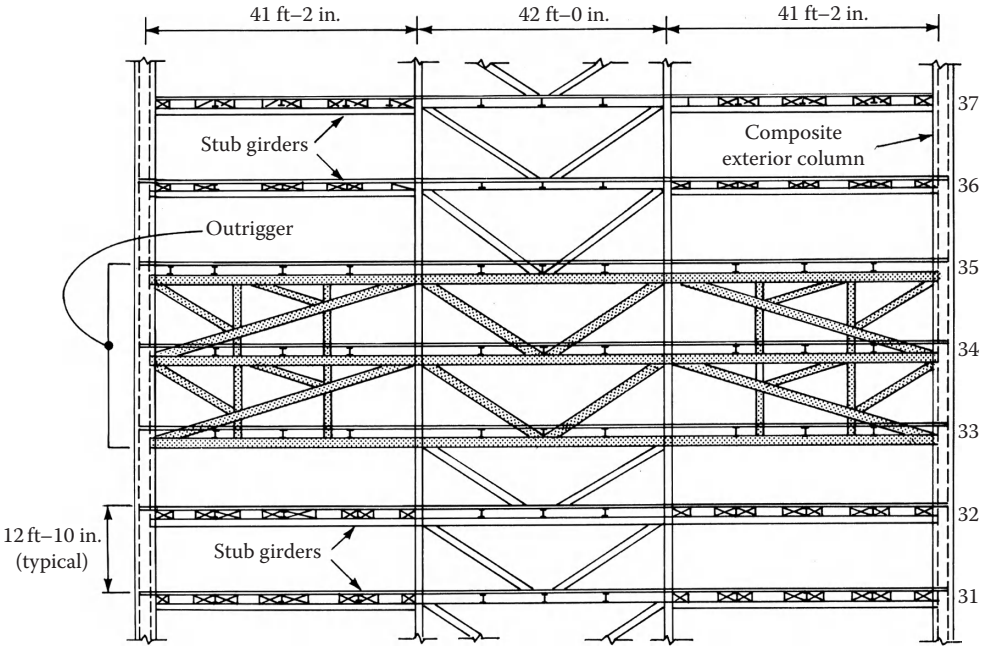


FIGURE 1.64 One Houston Center, Houston, Texas.

reducing the drift to less than 1/460 of the building height. The structural engineering for the project is by the Houston office of Walter P. Moore & Associates.

A novel floor framing system called the stub girder system is used in this project to frame typical bays measuring 41 ft 2 in. by 30 ft (12.54 m by 9.14 m). This system, discussed later in Chapter 4, attempts to reduce building cost by simultaneously reducing structural steel tonnage and floor-to-floor height.

1.10 FRAME TUBE SYSTEMS

The idea here is to develop a fully three-dimensional structural system that engages the entire building inertia to resist lateral loads. The goal is to achieve a higher degree of efficiency toward lateral load resistance for taller buildings. The resulting organization for a framed tube system is generally one of closely spaced exterior columns and deep spandrel beams rigidly connected together, with the entire assemblage continuous along each façade and around the building corners.

The resulting arrangement approximates a tube cantilevered from the ground with openings punched through the exterior walls. The closely spaced and deep exterior members often have a secondary benefit with respect to exterior cladding system costs. Exterior columns may eliminate the need for immediate vertical mullions of the curtain wall partially or totally.

The behavior of framed tubes under lateral loads is indicated in Figure 1.65, which shows the distribution of axial forces in the exterior columns. The more the distribution is similar to that of a fully rigid box cantilevered at the base, the more efficient the system is considered to be. For the case of a solid-wall tube, the distribution of axial forces would be expected to be nearly uniform over the windward and leeward walls and linear over the sidewalls. As the tubular walls are punched, creating the beam–column frame, shear frame deformations are introduced due to shear and flexure in the tubular members as well as rotations of the member joints. This reduces the effective stiffness of the system as a cantilever. The extent to which the actual axial load distribution in the tube columns departs from the ideal is termed the "shear lag effect." In behavioral terms, the forces in the columns toward the middle of the flange frames lag behind those nearer the corner and are thus less than fully utilized. The goal then is to limit shear lag effects such that the interior columns become as nearly effective as those nearer to the corners.

In simple terms, as characterized earlier, a tube system can be defined as a three-dimensional system that utilizes the entire building perimeter to resist lateral loads. The necessary requirements to create a wall-like tube structure is to place columns on the exterior relatively close to each other and to use deep spandrels moment connected to the columns. The structural optimization reduces to examining different column spacings and member proportions. In practice, the framed tubular behavior is achieved by placing columns at 10 ft (3.05 m) to as much as 20 ft (6.1 m) apart, with spandrel depth varying from 3 to 5 ft (0.90 to 1.52 m).

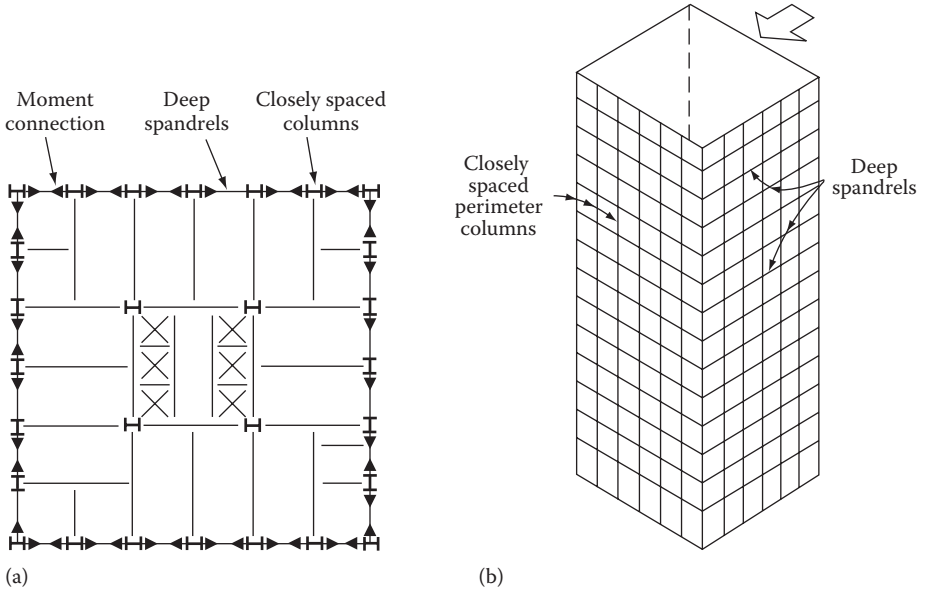


FIGURE 1.65 Framed tube: (a) schematic plan; (b) isometric view.

A somewhat different system that permits larger spacing of columns is the so-called braced tube that has diagonal or K-type of bracing at the building exterior. Yet another scheme is to use two or more tubes tied together to form a bundled tube. In this section, we discuss framed tube system. Other systems are described in later sections.

1.10.1 Behavior

To understand the behavior of a framed tube, consider a 50-story building, square in plan, as shown in Figure 1.62, consisting of closely spaced exterior columns and deep spandrel beams. Assuming that the interior columns are designed for gravity loads only, their contribution to lateral load resistance is negligible. The floor system, considered as a rigid diaphragm, distributes the loads to various elements according to their stiffness. Its contribution to lateral resistance in terms of its out-of-plane action is typically negligible. The system resisting the lateral loads thus comprises four orthogonal panels rigidly connected at the corners to form a tube in plan as shown in Figure 1.65. In tube structures, typically, the "strong" bending direction of the columns is aligned along the face of the building to benefit the most from their individual bending action. The frames parallel to the lateral load act as "web" of the perforated tube, while the frames normal to the loads act as flanges. When subjected to bending under the action of lateral forces, the primary mode of action is similar to that of a conventional tube, in which the columns on opposite sides of the neutral axis are subjected to tensile and compressive forces. In addition, the frames parallel to the direction of the lateral load are subjected to inplane bending, resulting in a shear racking action associated with an independent rigid frame.

The discrete columns and spandrels may be considered, in a conceptual sense, equivalent to the walls of a hollow tube cantilevering from the ground with a basic stress distribution as shown in Figure 1.66. Although the structure has a tubelike form, its behavior is much more complex than that of a solid tube; unlike a solid tube it is subjected to shear lag effects.

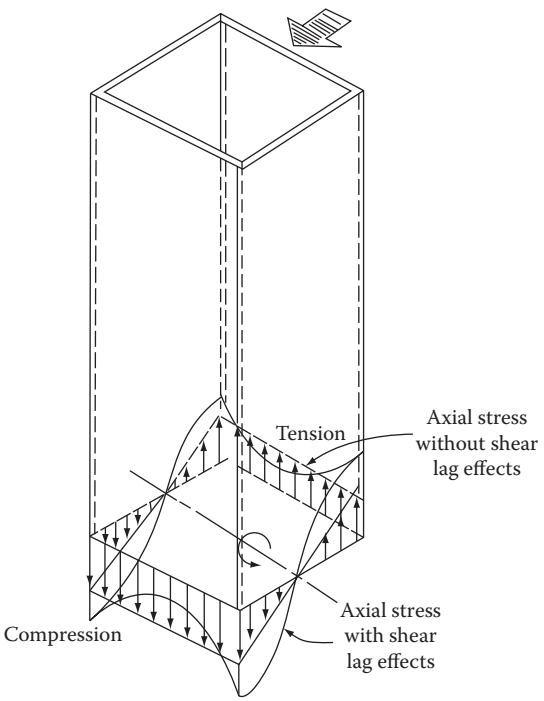


FIGURE 1.66 Axial stress distribution in a square hollow tube with and without shear lag.

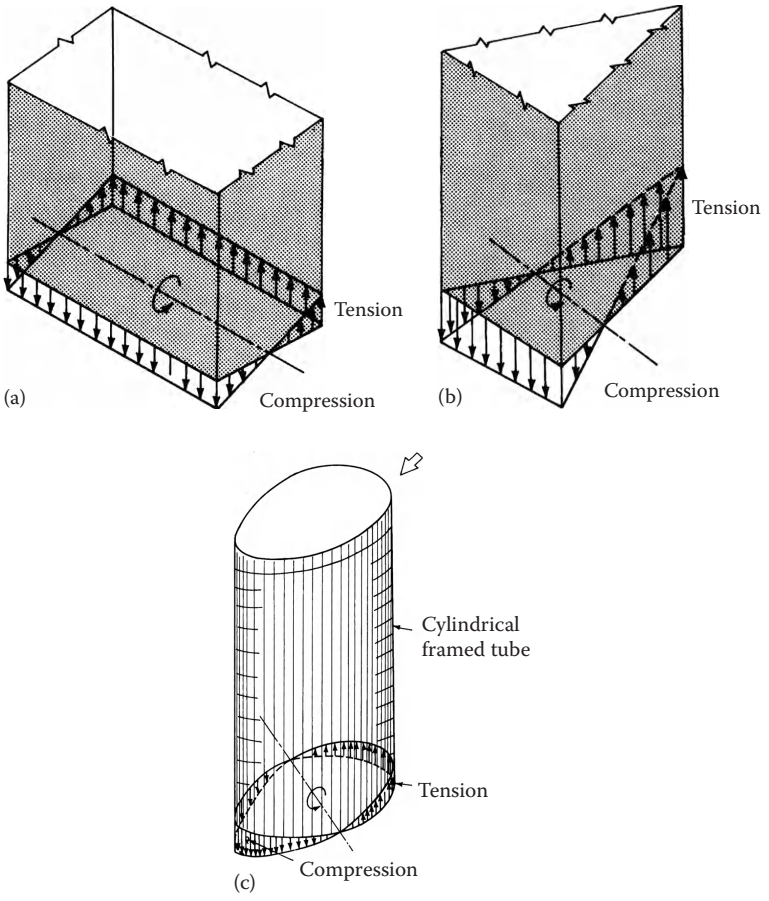


FIGURE 1.67 Axial stress distribution in tube structures neglecting shear lag effects: (a) rectangular tube; (b) triangular tube; (c) circular tube.

The influence of shear lag is to increase that axial stresses in the corner columns and reduce the same in the inner columns of both the flange and the web panels as shown in Figure 1.66. Ignoring the shear lag consequences for now, the analogy of the hollow tube can be used to visualize the axial stress distribution in rectangular, circular, and triangular buildings as shown in Figure 1.67. This idea of utilizing the entire building foot-print to resist lateral loads has allowed designers considerable freedom in manipulating building plans. The rigorous organization of orthogonal bay spacing considered necessary some 30 years ago is no longer a requisite. See Figure 1.68 for some examples of plan configurations.

Depending upon the height and plan dimensions of the building, the spacing of exterior columns is usually 10–15 ft (3–4.6 m), although a spacing as close as 3.8 ft (1.0 m) was used for the nonexistent 110-story World Trade Center twin towers, New York. The efficiency of the system is directly related to building height-to-width ratio, plan dimensions, spacing, and size of columns and spandrels.

In simplistic terms the behavior of tube is similar to that of a hollow cantilever without shear lag effects. In reality, however, its response to lateral loads is in a combined bending and shear mode. The overall bending is due to axial shortening and elongation of the columns while the shear deformation is due to the rotation of exterior columns and spandrels. One of the principles for an efficient tube design is to minimize shear lag effects.

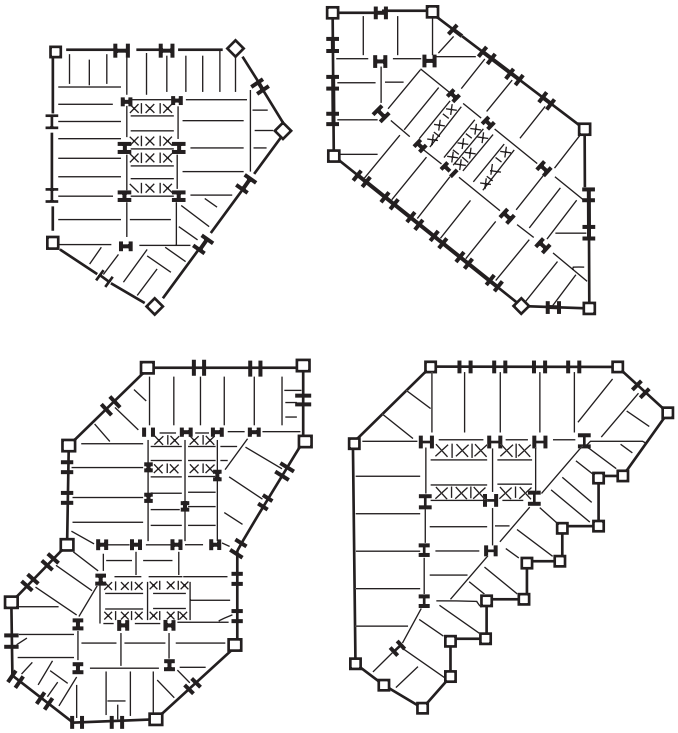


FIGURE 1.68 Free-form tubular configurations.

1.10.2 Shear Lag

The primary bending action of the hollow tube gets complicated by shear lag effects due to the flexibility of the spandrel beams. The effect of shear lag is to increase the axial stresses in the corner columns and reduce the same in the interior columns of both the flange and the web panels. As will be seen presently, even for a solid-wall hollow tube, there is a considerable shear lag effect.

A circular hollow tube, tapering up the building height, not unlike a chimney, is perhaps an ideal structural configuration for a tube building. Obviously, this form is not admissible as a human habitat, but will serve as a model for understanding shear lag effects in hollow tubes.

To fix ideas, suppose that the hollow box shown in Figure 1.69 represents a 110 by 110 ft (33.5 by 33.5 m), 50-story steel building with a floor-to-floor height of 13 ft (3.94 m), for a total height of 650 ft (198 m). Assume further, that a unit quantity of steel equal to 24 psf (1149 Psa) of the floor area is required for the building. Let us pretend, once more, that the entire 24 psf of steel is allocated to the perimeter wall of the tube building. The equivalent thickness of steel wall is determined thus:

$$\text{Total steel available} = 24 \times 110 \times 110 \times 50 = 14{,}520{,}000 \text{ lb } (64{,}585 \text{ kN})$$

$$\text{Area of perimeter wall} = 4 \times 110 \times 650 = 286{,}000 \text{ ft}^2 \ (26{,}569 \text{ m}^2)$$

$$\text{Equivalent thickness of wall} = 14{,}520{,}000/286{,}000 \times 3.4 = 15 \text{ in. } (381 \text{ mm})$$

In comparison to the plan dimensions of 110 by 110 ft, the calculated wall thickness of 15 in. (381 mm) is relatively thin. It is evident that the building has a tendency to behave like a thin-walled beam, in which the shear stresses and strains are much larger than in a solid section. Because of the resulting large shear strains, the usual assumption of plane sections remaining plane after bending,

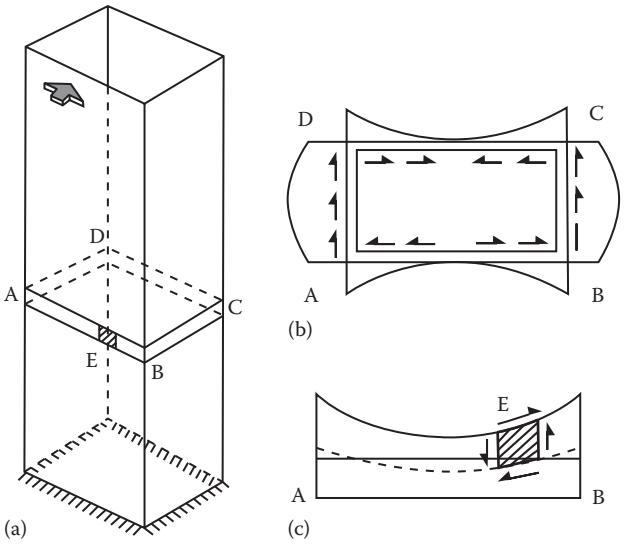


FIGURE 1.69 Shear lag effects in tube structures: (a) cantilever tube subjected to lateral loads; (b) shear stress distribution; (c) distortion of flange element caused by shear stress.

used in engineers' bending theory is no longer applicable. The large shearing strains cause in-plane distortions as shown in Figure 1.69c. The final outcome is that under lateral load, bending stresses will not be proportional to the distance from the neutral axis of the section; the stress at the center of the flanges "lags" behind the stress near the corners because of the lack of shear stiffness of the wall panel. This phenomenon known as shear lag, plays an important role in the design of tubular high-rise structures. The bending stresses in the webs are also affected in a similar manner as shown in Figures 1.66 and 1.70.

The shear lag effects in tubular buildings consisting of discrete columns and spandrels may readily be appreciated by considering the basic mode of action involved in resisting lateral forces. The primary resistance comes from the web frames which deform so that the columns T are in tension and C are in compression (Figure 1.71). The web frames are subjected to the usual in-plane bending and racking action associated with an independent rigid frame. The primary action is modified by the flexibility of the spandrel beams which causes the axial stresses in the corner columns to increase and those in the interior columns to decrease.

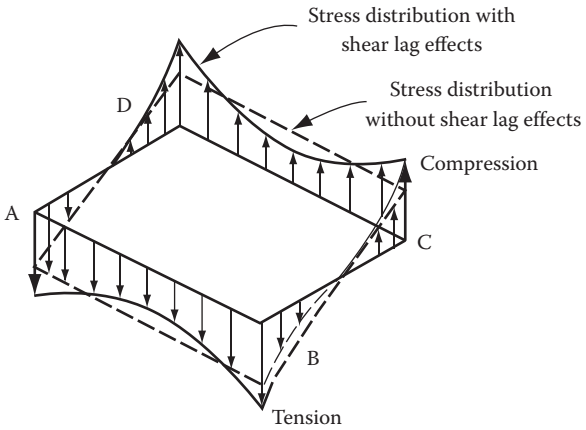


FIGURE 1.70 Axial stress distribution.

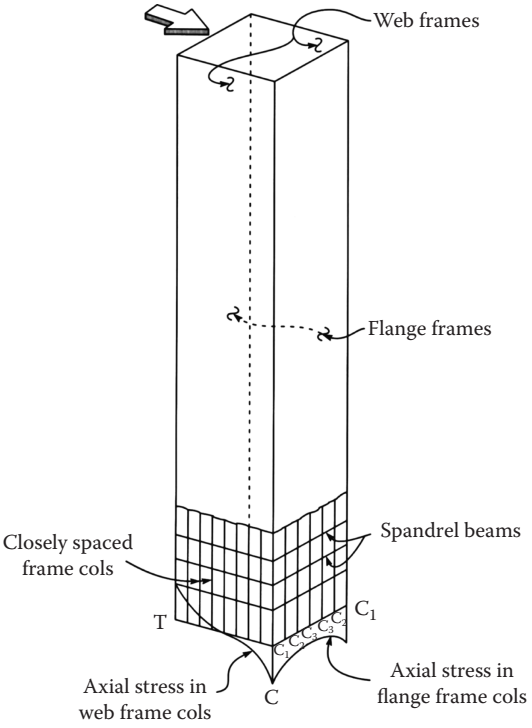


FIGURE 1.71 Shear lag in framed tube.

The principal interaction between the web flange frames occurs through the axial displacements of the corner columns. Referring back to Figure 1.71, when column C, for example, is under compression, it will tend to compress the adjacent column C_1 since two are connected by the spandrel beams. The compressive deformations of C_1 will not be identical to that of corner column C since the flexible connecting spandrel beam will bend. The axial deformation of C_1 will be less, by an amount depending on the stiffness of the connecting beam. The deformation of column C_1 will in turn induce compressive deformations to the next inner column C_2, but the deformation will again be less. Thus, each successive interior column will experience a smaller deformation and hence a lower stress than the outer ones. The stresses in the corner column will be greater than those from pure tubular action, and those in the inner columns will be less. The stresses in the inner columns lag behind those in the corner columns, hence the term shear lag.

The difference between stress distribution, as predicted by engineer's beam theory and the actual distribution is illustrated in Figures 1.66, 1.70 and 1.71. Because the column stresses are distributed less effectively than in an ideal tube, the moment resistance and the flexural rigidity are reduced. Thus, although a framed tube is quite an efficient system for tall buildings, it does have limitations due to shear lag effects.

1.11 IRREGULAR TUBE

The framed tube concept can be executed with any reasonable arrangement of columns around the building perimeter as noted earlier in Figure 1.68. However, non-compact plans, and plans with reentrant corners, reduce the efficiency of the system. For framed tubes, a compact plan may be defined as one with an aspect ratio of not greater than 2.0. Elongated plans with higher aspect ratios impose a cost premium because (1) for wind design, the elongated building elevation acts like a sail collecting large wind loads; (2) the resulting shear forces most usually require closer spacing and

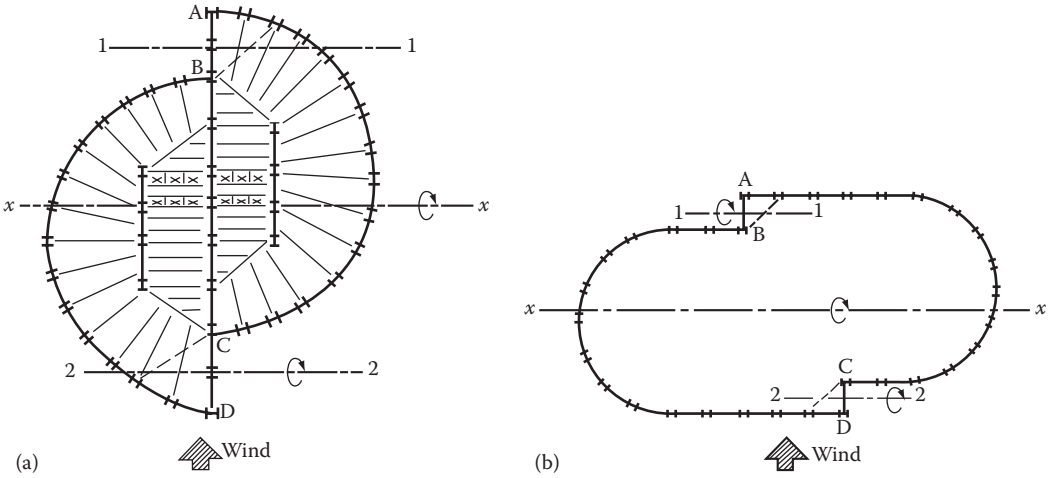


FIGURE 1.72 Offset tubes: (a) semicircular tube; (b) rectangular tube with semicircular sides.

or larger size columns and spandrels parallel to the wind; (3) shear lag effects are more pronounced particularly for columns oriented perpendicular to the direction of wind.

In a similar manner, a sharp change in the tubular form results in a less efficient system because the shear flow must pass around the corners solely through axial shortening of the columns. Also, a secondary frame action at these locations alters the axial load distribution in the columns as explained below.

Consider the framed tubes shown in Figure 1.72. To keep the explanation simple, let us assume that shear lag effects are negligible. For lateral loads in the North-South direction, the leeward columns A and B are subjected to compression while the windward columns C and D to tension. In addition to the primary action, the frames AB and CD experience a secondary bending action about their own local axes 1-1 and 2-2, as shown in Figure 1.69. The resulting axial forces in columns A and B are compression and tension respectively. The effect of local bending is to increase the compressive force in A while decreasing the same for column B. The final axial force is the summation of the two; the primary tube action and the secondary local frame action.

1.12 TRUSSED TUBE

For very tall buildings, the dense grid of beam and column members of a framed tube has quite an impact on the façade architecture. The need to control shear lag for improving the system efficiency can only be realized by relatively small perforations in the tubular walls. An example is the 3 ft 4 in. spacing used for the now absent World Trade Center Towers.

A trussed tube however opens up the façade and is uniquely suited to the qualities and character of structural steel. This is achieved by introducing a minimum number of diagonals on each façade and making the diagonals intersect at the same point at the corner column. The system is tubular in that the fascia diagonals not only form a truss in the plane but also interact with the trusses on the perpendicular faces to affect the tubular behavior.

The principle of façade diagonalization can readily be used for partial tubular concepts. For example, in long rectangular buildings, the end frames on the short face may be diagonalized, whereas on the long face a moment-resisting frame may be used.

An effective trussed tube action may be achieved by replacing vertical columns with closely spaced diagonals in both directions, as shown in Figure 1.73. However, this system is not popular because it creates certain challenges in detailing of curtain walls.

The diagonally braced tube, shown in Figure 1.74, is by far the most usual method of increasing the efficiency of the framed tube. It is an eloquent statement of structural efficiency accomplished

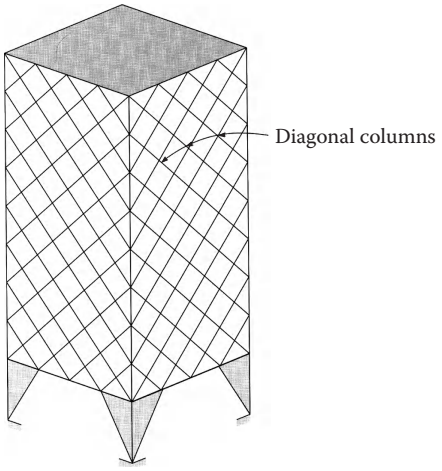


FIGURE 1.73 Tube building with closely spaced diagonal columns.

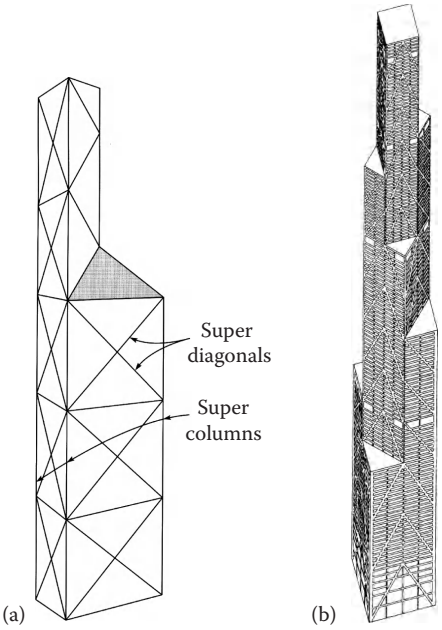


FIGURE 1.74 (a) Tube building with multilevel diagonal bracing; (b) rotating tube with super diagonals.

by adding a minimum number of diagonals on each façade. Observe that the diagonals must intersect at the same point at the corner columns. The system then is three dimensional in that the fascia diagonals interact with the trusses on the perpendicular faces.

The diagonals connected to the columns at each intersection, virtually eliminate the effects of shear lag in both the flange and web frames. As a result, the structure behaves more like a braced frame, with greatly diminished bending of the exterior columns and girders. Consequently, the spacing of the columns can be larger with the size of the columns and spandrels smaller than in framed tubes, thus allowing larger size windows. In a braced-tube, the bracing contributes also to the improved performance of the tube in carrying gravity loading. Differences between gravity load in the columns are evened out by the braces transferring axial loading from the more highly to the less stressed columns. This benefit although present in a framed tube, is not as much.

1.13 BUNDLED TUBE

In the previous sections, we discussed tubular systems generally applicable to prismatic profiles including a variety of nonrectilinear plans, such as circular, hexagonal, triangular, and other polygonal shapes. However, for buildings with significant vertical offsets, the discontinuity in the tubular forms introduces serious inefficiencies. A bundled tube that can be configured with multiple cells, on the other hand, provides for vertical offsets without much loss in efficiency. Additionally, it allows for wider column spacings that would be possible with a single cell tube. In principle, many building shapes can be configured using bundled tubes (Figure 1.75).

1.13.1 Behavior

The structural principle behind the bundled tube concept is that the interior rows of columns and spandrels act as interior webs minimizing shear lag effects. Without their beneficial effect, as stated earlier, the exterior columns in a framed tube toward the center of the building would be of little use in resisting the overturning moment. The bundled tube system may therefore be thought of as an extension of the perimeter tubular system with stiffened interior frames.

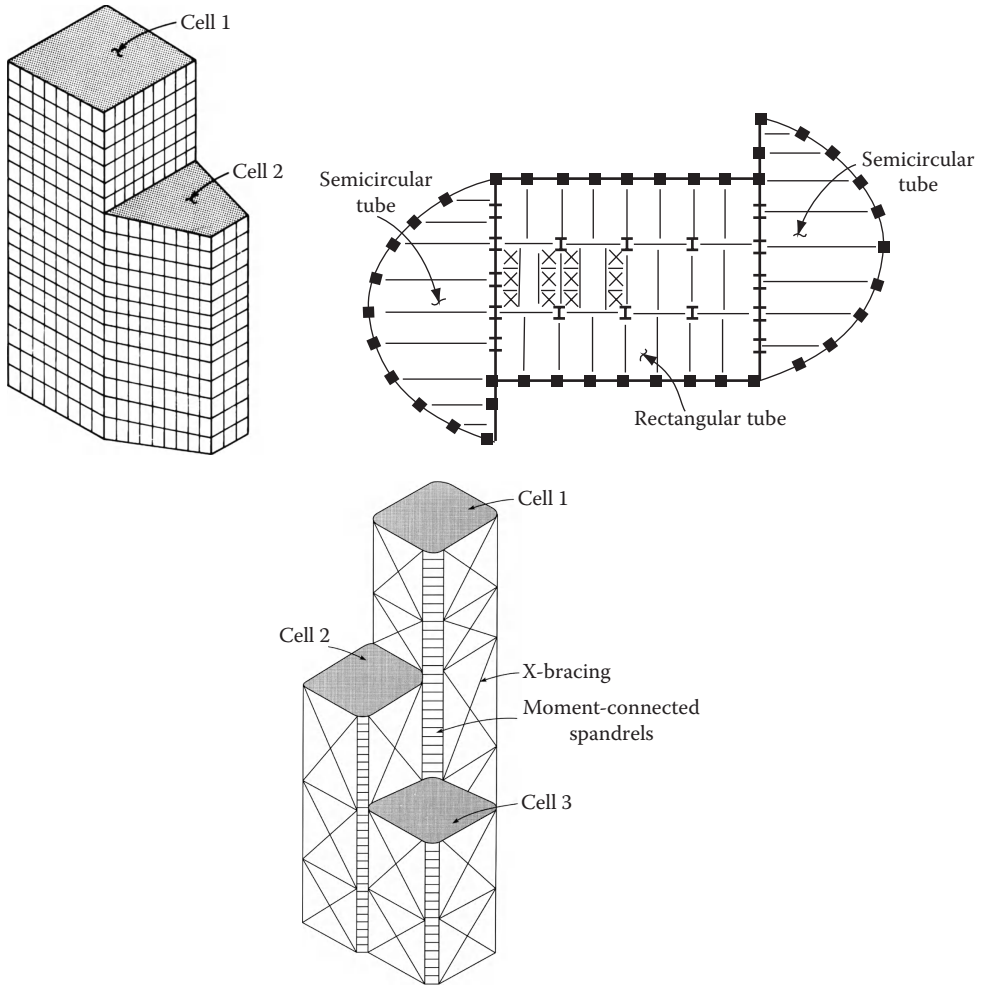


FIGURE 1.75 Bundled tubes.

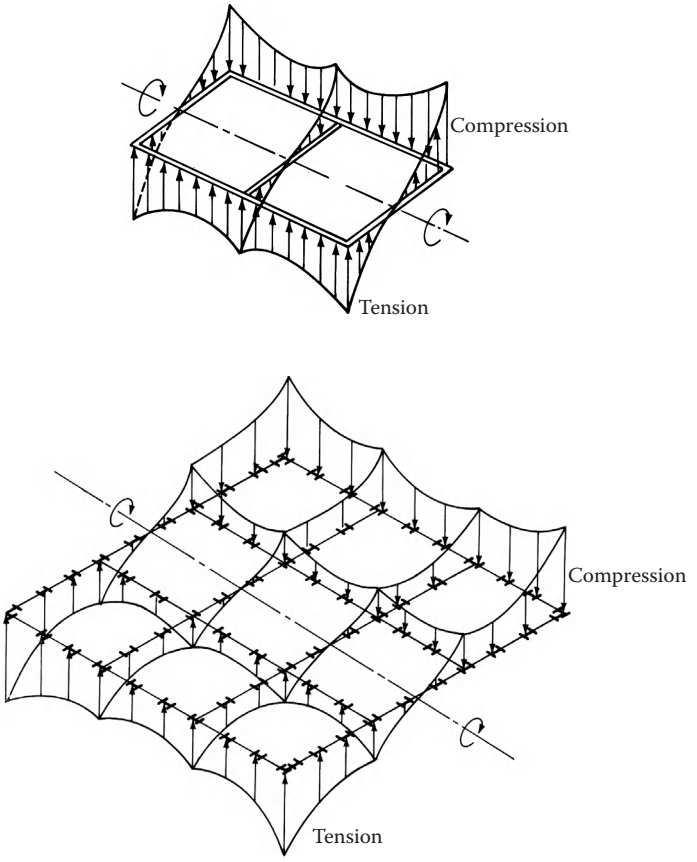


FIGURE 1.76 Bundled tube behavior.

The individual cells of a bundled tube may be curtailed at different heights without diminishing structural efficiency. The torsional loads are readily resisted by the closed form of individual cells, permitting greater spacing of columns, and shallower spandrels. Larger window openings are possible. The shear lag experienced by conventional framed tubes is reduced by the addition of interior framed "web" panels across the entire width of the building. When subjected to lateral forces the presence of interior web reduces the nonuniformity of column axial forces caused by shear lag. This effect is shown schematically in Figure 1.76.

1.14 ULTIMATE HIGH-EFFICIENCY SYSTEMS FOR ULTRA TALL BUILDINGS

The concept behind high-efficiency systems, first envisioned by master builder, the late Dr. Fazlur Khan of Skidmore, Owings & Merrill, is based on three requisites:

1. Transfer as much, if not all of the gravity load into those columns that resist over-turning moments caused by lateral loads.
2. Position these columns as far back as possible, from the geometric center of the building.
3. Interconnect these columns with a structural system capable of resisting shear forces due to lateral loads.

The first principle is not entirely new. The idea of transferring additional gravity loads into columns of braced frames and/or frame columns of moment frames to mitigate uplift forces is quite routine.

This is so particularly when the width of the lateral bracing system is narrow compared to the building height. Coaxing additional loads into these columns helps in anchoring the building firmly at the base.

The reason for the second requirement is to maximize the bending rigidity of the building. By placing the columns at the periphery of the building, we are in effect, maximizing the bending rigidity of the building.

The need for interconnecting the columns with a shear-resisting system is to coerce them to function as tension-and-compression-resisting columns of a giant brace, and not so much as individual bending members. Without the presence of an interconnecting shear-resisting system, the columns would function as individual bending members with the resistance merely equal to the sum of the independent resistances of the individual columns.

To explain the concept further, let us consider a tall building, square in plan, some 120-stories tall, with closely spaced columns and deep spandrels at the perimeter. The system commonly referred as tube system, has been the workhorse of tall building construction for the past 30 years.

Referring to our tall building shown in Figure 1.77, one obvious option for channeling all gravity loads into the perimeter columns would be to eliminate interior columns all together. This would leave a column-free volume inside of the building but would be prohibitively expensive to clear-span

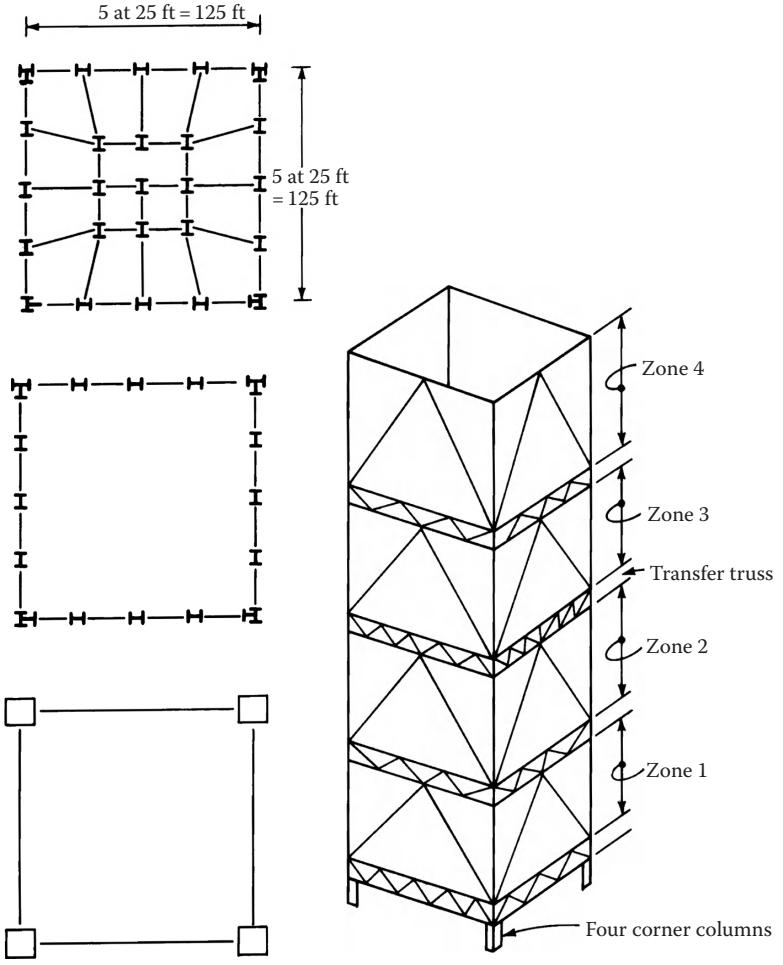


FIGURE 1.77 Structural concept for ultra-high-rise building.

the entire floor area at each level. Therefore, we need to figure a way of transferring the entire gravity load into the perimeter columns without incurring a significant premium in the framing the floor system. This is considered next.

An obvious solution for achieving this goal is to provide transfer-floors recurring at selected levels, say, for example, at every 12th-floor for the 120-story building considered here. At these levels, a system of one or two-story-deep Vierendeel trusses are used to clear span the entire area within the perimeter columns. Where appropriate, the transfer levels would be used as sky lobbies or other common usage areas.

The transfer trusses are then used to support interior columns within certain designated zones between the transfer levels. In our building, with transfer levels at every-12th-floor, we would be picking up interior columns that are 11-story tall—a relatively economical task given the presence of one story-deep, two-way Vierendeel trusses. One advantage of the system is that the interior columns do not have to line-up for the entire height of the building—within any given zone between two consecutive transfer levels, the columns may be placed to suit the interior space planning desired for specific occupancies. Yet another benefit is it lends itself to opening up certain floors without any interior columns. This is possible because floors above the no-column floors may be hung from the Vierendeel trusses above, while those below the no-column floor are supported on the transfer trusses below. The obvious advantage is that we are able to achieve channeling of the entire gravity load to the four corner columns without paying a significant penalty in steel tonnage.

The third requisites, as stated previously, is to mobilize the corner columns as chords of a gigantic vertical brace. This is done best, by using fascia diagonals.

2 Lateral Load-Resisting Systems for Composite Buildings

PREVIEW

It is worth noting, for the sake of modesty, that the basic ideas of composite behavior were conceived long before the parents of anyone reading this work was born. In the United States, composite construction first appeared in 1894. In that year, Joseph Milan, an engineer from Vienna, built a composite arch bridge in Rock Rapids, Indiana, using steel I-beams encased in concrete. Replacement of the curved beams with straight beams was the next step. By the turn of the century, the use of concrete encased steel beams was common for both buildings and bridges. However, it took some 30 more years to codify composite design. The design rules first appeared in the 1930 New York Building Code by permitting a bending stress of 20 ksi, an 11% increase over the 18 ksi allowed for noncomposite beams at that time.

The success of combining steel and concrete in composite floor systems motivated the development of composite building systems, particularly those using composite columns some 30 years ago. Economic studies have consistently shown that a composite column is about four to six times more economical than an all-steel column.

The two building systems, concrete and steel, evolved independently of each other, and up until the 1960s, engineers were trained to think of tall buildings in either steel or concrete. Dr Fazlur Khan, of Skidmore, Owings & Merrill, broke this barrier in 1969 by blending steel and concrete into a composite system for use in a relatively short, 20-story building, in which the exterior columns and spandrels were encased in concrete to provide the required lateral resistance. The system was basically a steel frame stabilized by reinforced concrete. However, today the advent of high-strength concrete has ushered in the era of super columns and mega frames where the economy, stiffness, and damping characteristics of large concrete elements are combined with the lightness and constructability of steel frames. Without this type of framing, many of our contemporary tall buildings may never have been built in their present form.

The term composite system has taken on numerous meanings in recent years to describe many combinations of steel and concrete. Herein, the term means any and all combinations of steel and reinforced concrete elements and is considered synonymous with other definitions such as mixed systems, hybrid systems, etc. The term is used to encompass both gravity- and lateral-load resisting elements.

Since the advent of the first composite system, engineers have not hesitated to use a whole range of combinations of steel and reinforced concrete members to capitalize on the advantages of each material. This has resulted in numerous systems making distinct categorization a nearly impossible task. However, the systems can be best described by studying them under three categories: (1) composite members, (2) composite subsystems, and (3) composite building systems.

2.1 COMPOSITE MEMBERS

To get an insight into the composite behavior of structural steel and reinforced concrete systems, it is instructive to study common techniques of compositing the following structural units:

1. Composite slabs
2. Composite beams

3. Composite columns
4. Composite diagonals
5. Composite shear walls

The behavior of slabs, beams, girders, and spandrels under gravity loads is covered in Chapter 3. In this section their behavior under lateral loads is considered.

2.1.1 Composite Slabs

In steel buildings, the use of high-strength, light-gauge (16–20 gauge) metal deck with concrete topping has become a standard (see Figure 2.1). The metal deck has embossments pressed into the sheet metal to achieve composite action with concrete topping. Once the concrete hardens, the metal deck becomes the tension reinforcement. The resulting composite slab acts as a diaphragm providing for the horizontal transfer of shear forces to the vertical bracing elements. Furthermore, it acts as a stability bracing for the compression flange of steel beams.

The shear forces in the diaphragm mostly occur in the concrete topping because the in-plane stiffness of concrete slab is significantly more than that of the metal deck. Thus, the horizontal forces must transfer from the slab to the beam top flange through welded studs. This diaphragm behavior, an important aspect of seismic design, is discussed later in Chapter 6.

The metal deck serves as a form for concrete placement, and as positive reinforcement for the composite concrete slab. Welded wire fabric is placed in the upper part of the slab to provide continuity reinforcement over beams and girders and to control shrinkage cracking often supplemented

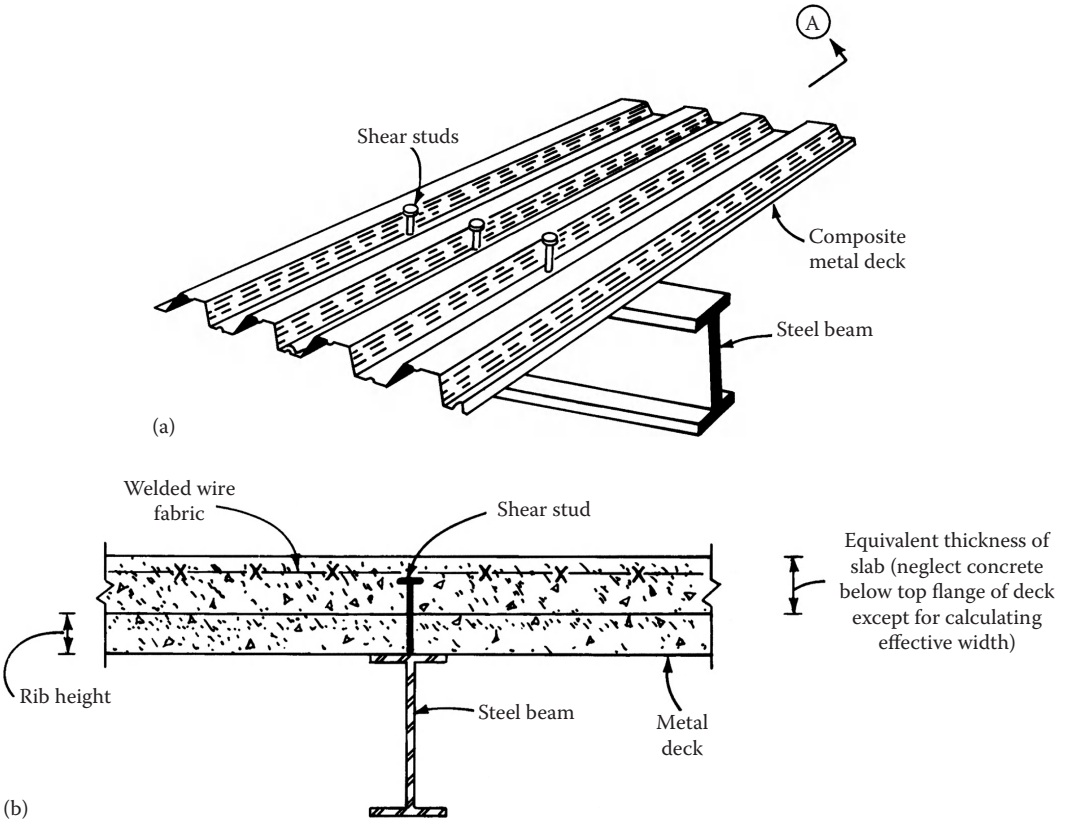


FIGURE 2.1 Composite beam with formed metal deck: (a) schematic view; (b) section A.

with additional reinforcement. Deck shoring is used rarely since it is expensive and slows down the construction sequence, so the deck gauge is selected to satisfy forming strength requirements. Once the concrete slab is in place the strength of the slab system usually far exceeds normal loading demands. Fire protection requirements usually dictate thickness of concrete topping. Deck manufacturers test assemblies for fire resistivity and then obtain appropriate fire resistivity rating from the various code authorities. Typically, floor assemblies in high-rise buildings must satisfy a fire resistivity of 2 h. Popular 2 h systems today (2010) are metal deck supporting 3¼ in. of 110 lb/ft^3 lightweight concrete or alternatively metal deck supporting 4½ in. of 150 lb/ft^3 normal weight concrete. Neither of these systems requires sprayed on fire proofing concrete over metal deck, which requires fireproofing of the underside of the deck to attain the 2 h fire resistivity.

Steel beams also serve as a part of the shoring system for the placement of concrete. Rarely are beams shored because of the cost and time involved.

Structurally, the principal attributes of a composite section are realized by developing the appropriate use for each material. Concrete, which is weak in tension, carries compressive loads while steel, strong and stable in tension, carries tension loads.

2.1.2 Composite Girders

Consider a typical steel moment frame consisting of beams rigidly connected to columns. The stiffness of the frame is controlled mostly by the stiffness of the girder. This is because in a typical frame with a column spacing of 25–35 ft, and a floor-to-floor height of 12½–13½ ft, the columns are much stiffer than the beams. Therefore, to limit the sway under lateral loads, it is more prudent to increase the girder stiffness rather than the column stiffness. Although frame beams are designed as non-composite, it is a usual practice to use shear studs at a nominal spacing of say, 12 in., particularly in buildings assigned to high Seismic Design Categories (SDCs). The shear connectors primarily provided for the transfer of diaphragm shear also increase the moment of inertia of the girder. The increase, however, is not for the entire length of the girder because under lateral loads, it bends in a reverse curvature. Since concrete is ineffective in tension, the increase in the moment of inertia can be counted on only in the positive moment region. A rational method may be used to take advantage of the increased moment of inertia. One such method is to use a dual approach: (1) Bare steel beam properties for strength calculations, and (2) an average of bare steel and composite properties for drift calculations.

2.1.3 Composite Columns

Two types of composite columns are used in buildings (see Figure 2.2a and b). The first, commonly referred to as *encased composite column*, consists of a steel core surrounded by a reinforced concrete envelope. The second referred to as *filled composite column* consists of a steel pipe or tube filled with high-strength concrete. In the first type, the steel core, most usually a wide flange section placed within the reinforced concrete column is designed as an erection column to carry construction loads only.

Conceptually the behavior of a composite column is similar to a reinforced concrete column, if the steel section is analytically replaced with an equivalent mild steel reinforcement. In fact, this concept provides the basis for generating the interaction diagram for the axial load and moment capacities of composite columns.

Compositing of exterior columns by encasing steel sections with concrete is by far the most frequent application of composite columns. The reasons are entirely economic, because forming around interior columns is quite involved and is not readily applicable to jump forms. Exterior columns, on the other hand, are relatively open-faced: formwork can be "folded" around steel columns for placement of concrete, then unfolded and jumped to the next floor. Repeating the cycle without having to dismantle the entire framework results in economy.

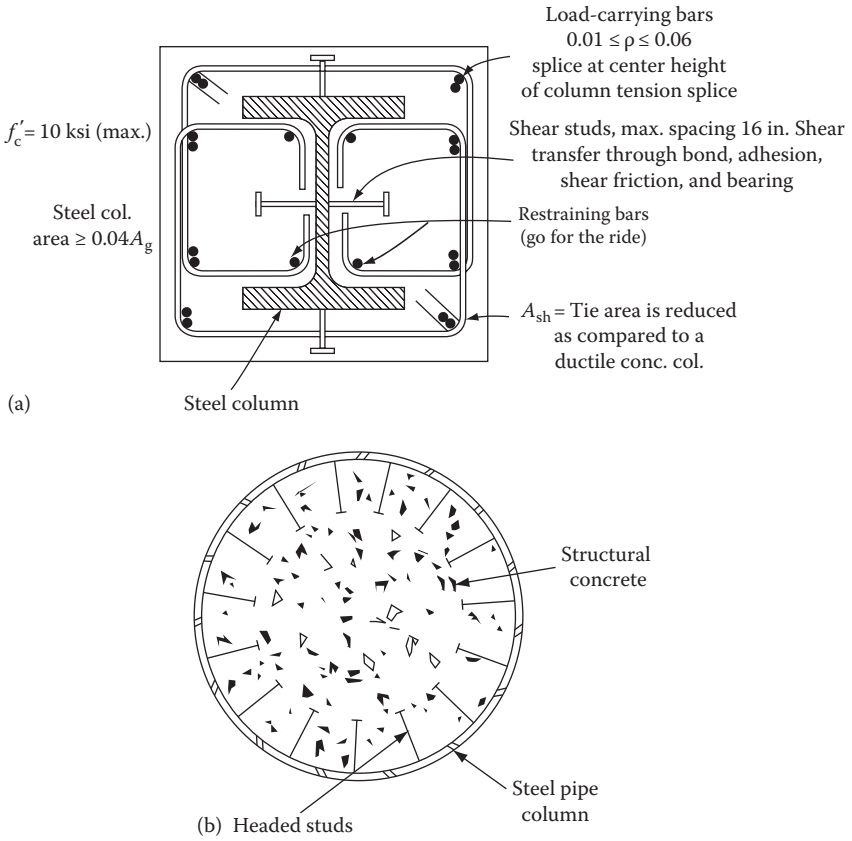


FIGURE 2.2 (a) Concrete encased composite column; design considerations. *Note*: Bond and adhesion must be ignored in calculating shear transfer. (b) Concrete-filled composite pipe column.

However, in Japanese construction, it is common practice to composite the interior columns as well. Their construction makes extensive use of welding for vertical as well as transverse reinforcement (Figure 2.3). Both longitudinal and transverse reinforcement are required in concrete encased composite columns. For buildings assigned to SDC A or B, the minimum reinforcement requirement of the ACI 318-05/08 is adequate to maintain the integrity of concrete encasement. For buildings in the higher SDC, however, more confinement of the vertical bars is required to achieve ductility under cyclic earthquake ground motions. This can be provided by closed hoops as in non-composite ductile concrete columns.

The second type of composite column consists of a steel pipe or tube filled will high-strength concrete. Typically, neither vertical nor transverse reinforcement is used. However, as in the case of encased composite column, shear connectors are provided to enhance interaction between the concrete and steel elements. Since no formwork is required, this type of construction is used both at the interior and exterior of buildings.

2.1.4 Composite Diagonals

As a part of a vertical truss, diagonals in a braced frame resist lateral forces primarily through axial stresses. As a result, braced frames are more economical than moment-resisting frames. However, their use is often limited, because of potential interference with architectural planning concerns.

An example of a tall building that has composite diagonals and columns is the Bank of China constructed between the years of 1985 and 1990 in Hong Kong. The building is 1007.7 ft (305 m)

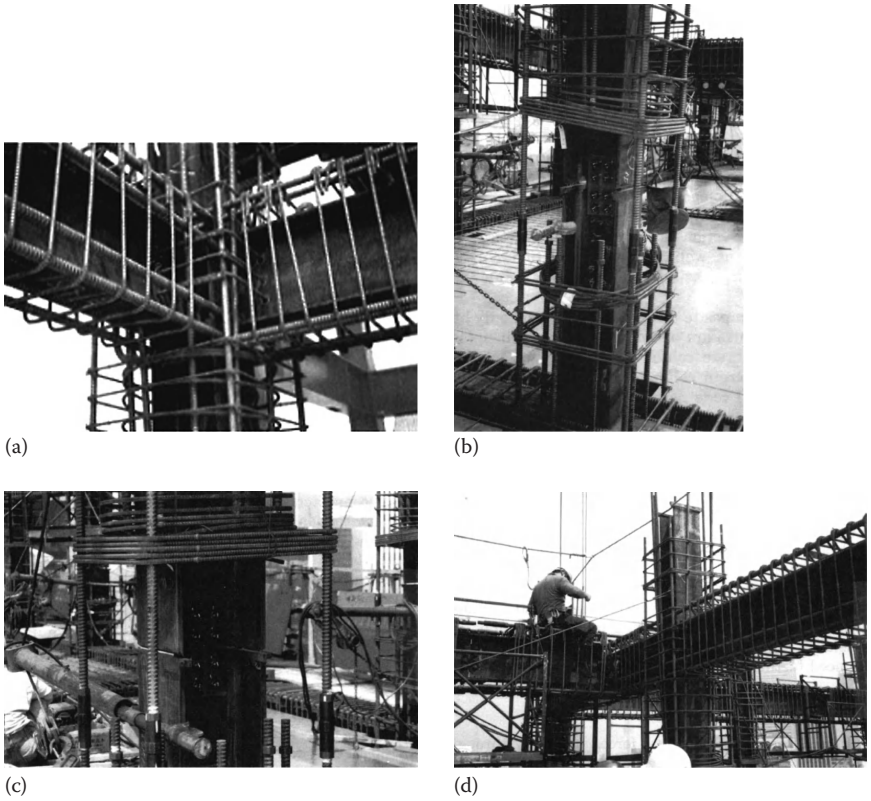


FIGURE 2.3 Japanese composite construction details: (a) beam column intersection; (b and c) composite column with welded ties; (d) general view.

high with two masts reaching a height of 1205.4 ft (367.4 m). It is a 72-story building with a four-level basement. The entire structure is supported by four composite columns at the corners of the building, with the triangular frameworks transferring the weight of the structure on to these columns. See Figure 2.4 for building schematics.

2.1.5 Composite Shear Walls

One of the most common uses of composite shear walls is in steel frame buildings, in which selected bays are infilled with a reinforced concrete wall. In essence, this results in a reinforced concrete shear wall with structural steel boundary elements and coupling beams (see Figure 2.5).

If the coupling beams were pin-connected at each end to the boundary elements, they would be ineffective in improving the lateral resistance of the wall. This is because the two wall piers would resist lateral loads independently. On the other hand, if the coupling beams are infinitely stiff, they would fully couple the two piers coercing them to work as a single unit. If the coupling beam stiffness is in between the two extremes, as is the case in most buildings, the corresponding response will also be in between the two limits.

Composite steel plate diaphragms are appropriate when extremely high shear forces must be transferred from one system to another at the base of the building. An example of this use may be found again in the Bank of China Tower, Hong Kong, in which the entire base shear is transferred from the building perimeter to the building core at the base, via a steel-plated floor diaphragm.

Possible details of composite shear plate walls are shown in Figure 2.6. In these details, structural steel framing surrounds the steel plates with the entire steel assembly encased in reinforced

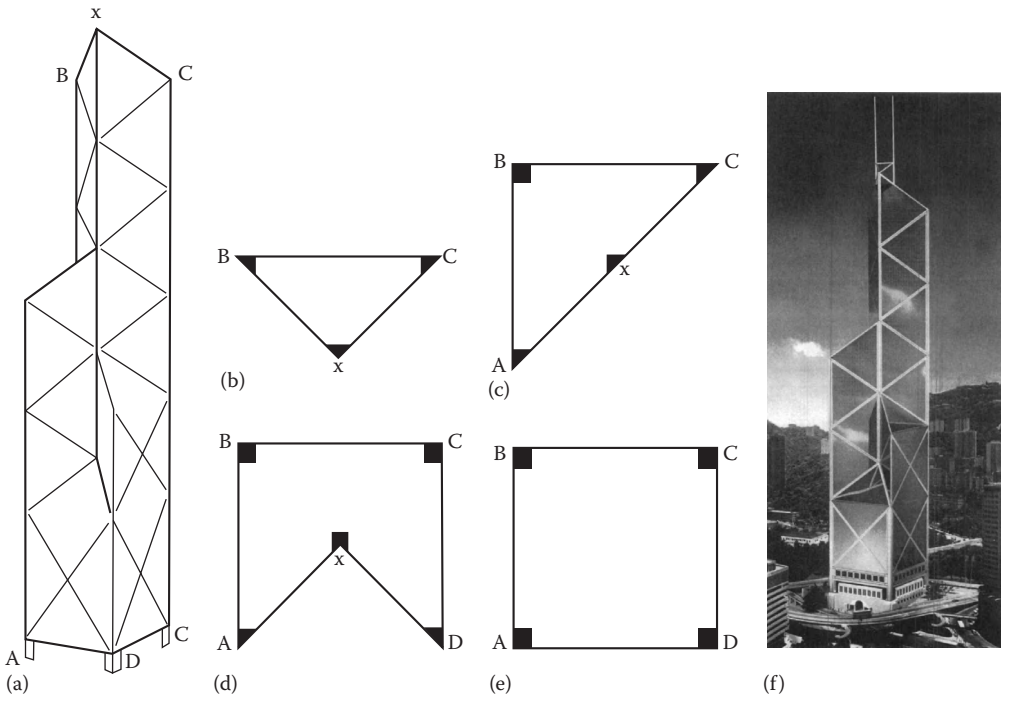


FIGURE 2.4 Bank of China, structural schematics: (a) elevation; (b–e) plans; (f) photograph.

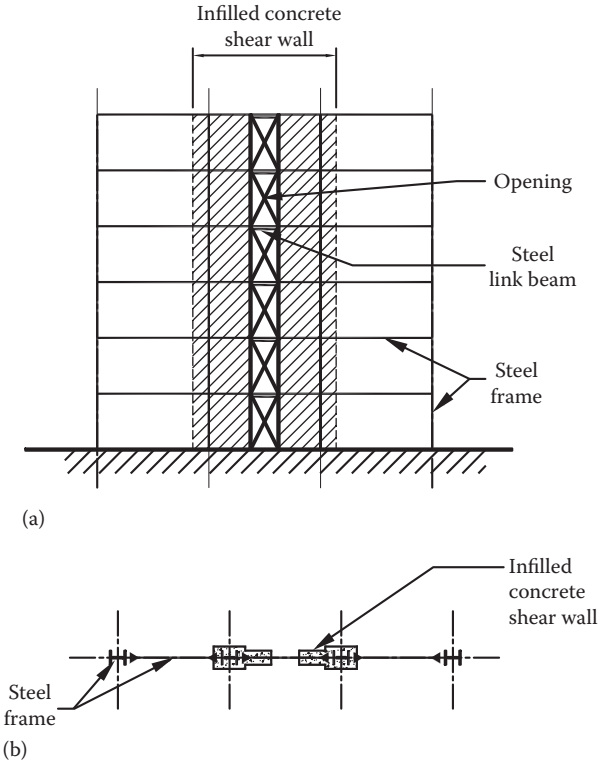


FIGURE 2.5 Reinforced concrete infill in steel frame: (a) elevation; (b) plan.

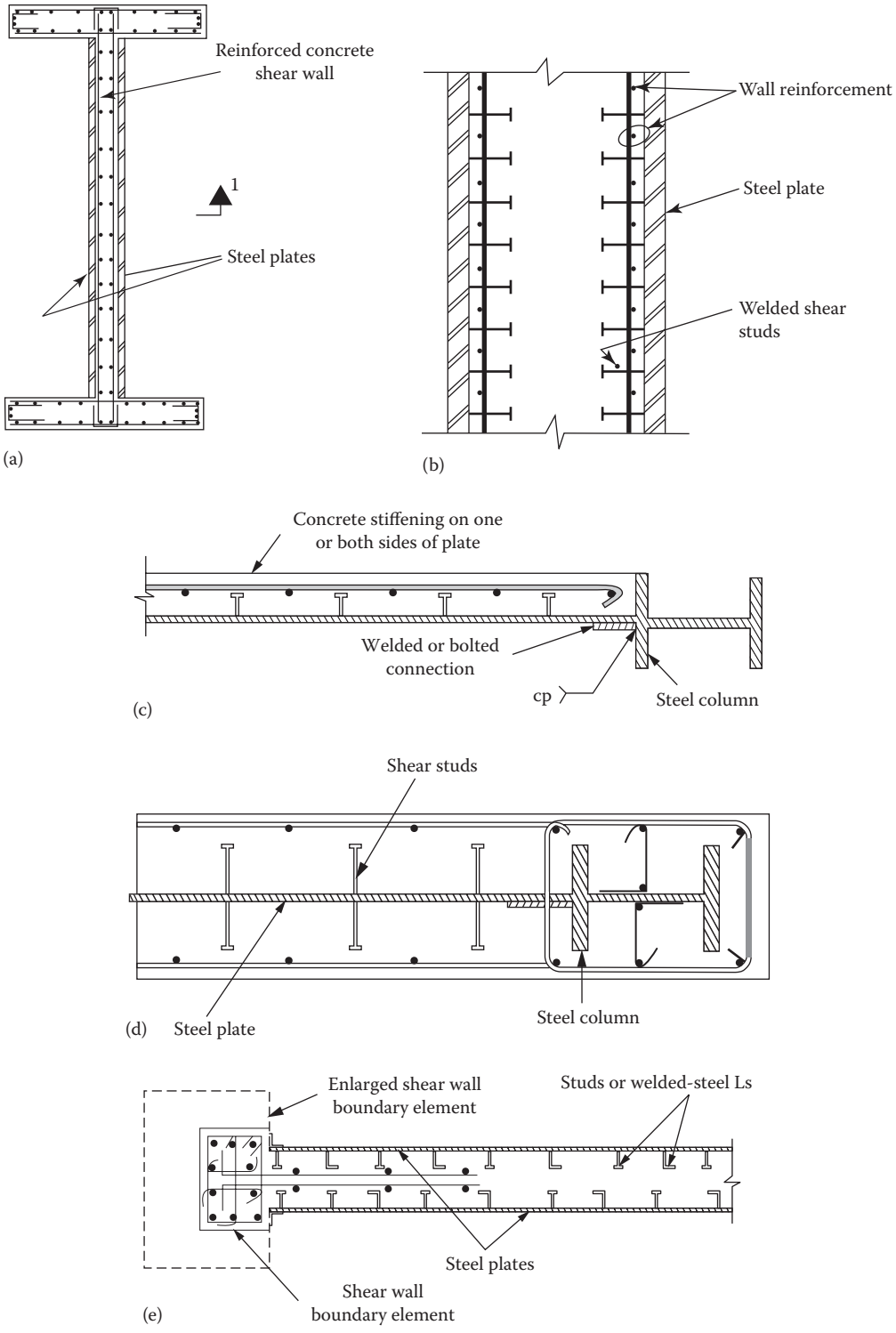


FIGURE 2.6 Composite steel plate shear walls: (a) plan; (b) section; (c) and (d) shear wall with single plate; (e) steel plate on both faces of wall;

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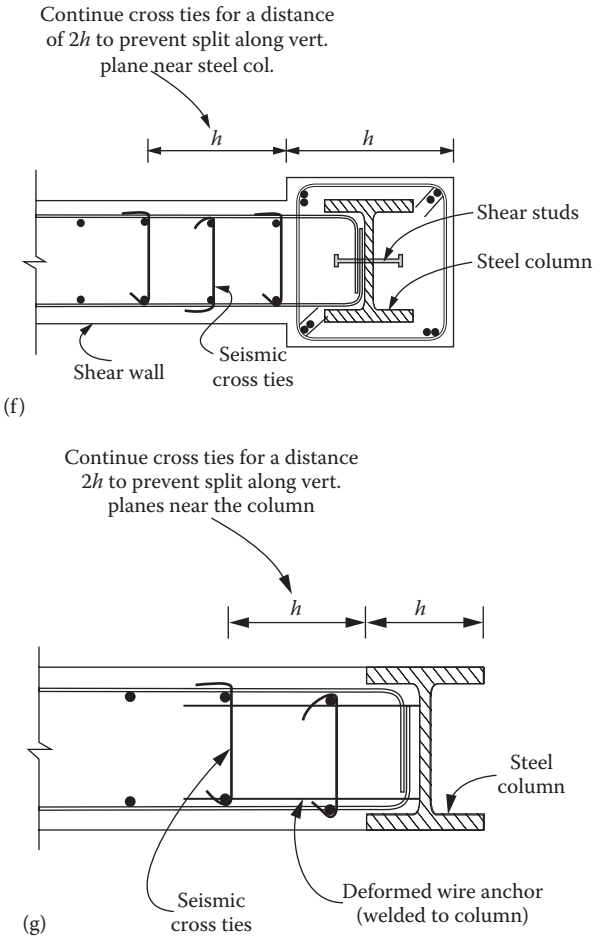


FIGURE 2.6 (continued) (f) and (g) concrete wall with steel boundary elements.

concrete. The steel columns not only resist gravity loads but also act as boundary members resisting overturning forces. The shear-wall web is a steel plate welded to the boundary members. A practical detail would be to provide a steel tab continuously fillet-welded in the shop to the beams and columns. The steel plate can then be attached to the tabs with erection bolts. Field fillet welds can be installed between the steel plate and the tabs. If the plates need to be installed in pieces because of size limitations in shipping or erection, field splices can be of fillet welds using a common backup plate. If there are openings in the wall, additional steel boundary members or flanges must be installed as required.

To prevent buckling of the steel plate, the completed steel assembly is encased in reinforced concrete. This also fireproofs the steel. The encasement should be thick enough to provide the stiffness needed to prevent buckling and should be reinforced for strength. Common details would include a regular pattern of welded studs on each side of the plate preventing buckling of the composite plate.

A system of reinforced concrete shear walls interconnected with steel link beams may be considered, in a large sense, as a composite unit (see Figure 2.7). Moment transfer between the steel beam and walls occurs through shear studs welded to the top and bottom flanges of the embedded link beams as shown in Figure 2.8.

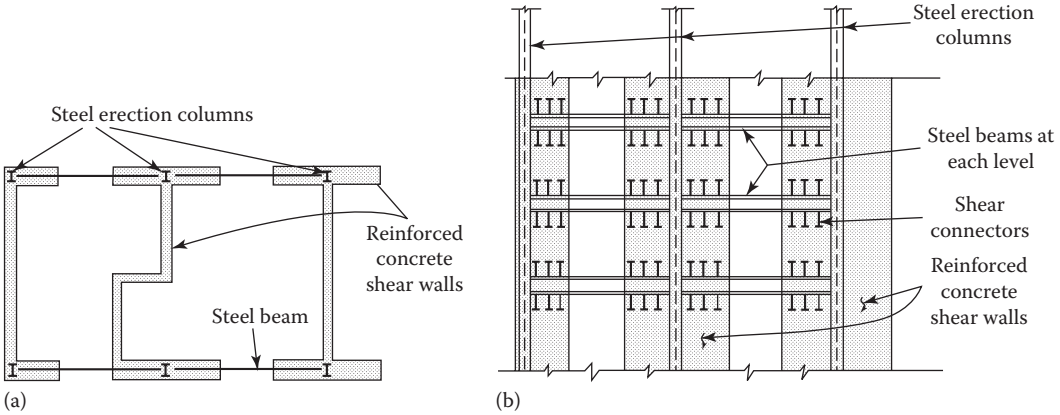


FIGURE 2.7 Composite shear wall with steel link beams: (a) plan; (b) elevation.

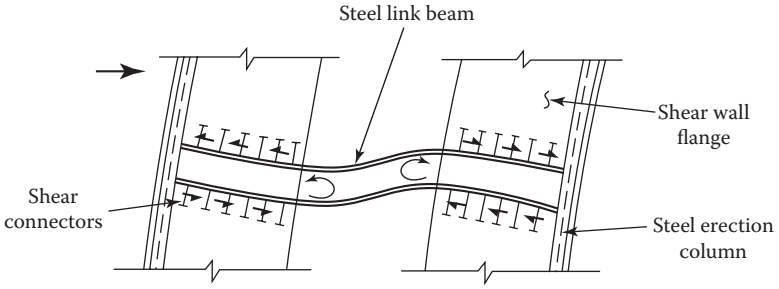


FIGURE 2.8 Moment transfer between steel link beam and concrete wall.

2.2 COMPOSITE SUBSYSTEMS

These systems may be categorized as follows:

1. Composite moment frames
2. Composite braced frames
3. Composite eccentrically braced frames
4. Composite shear wall-frame interacting systems
5. Composite tube systems
6. Vertically mixed systems
7. Mega frames with super columns
8. High-efficiency structures

2.2.1 Composite Moment Frames

Refer to Figure 2.9 for schematics of a composite moment frame consisting of steel beams and composite columns. The columns may consist of either concrete encased or filled composite columns, with moment connected steel beams.

Seismic provisions of AISC 341-05 recognize three types of composite moment frames:

1. Ordinary moment frames
2. Intermediate moment frames
3. Special moment frames

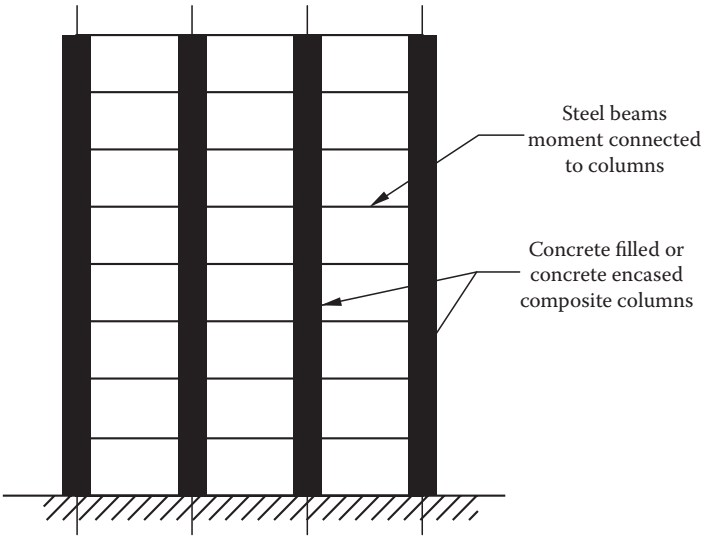


FIGURE 2.9 Composite moment frame.

Intermediate and special moment frames must satisfy detailing requirements that are more stringent than those for ordinary moment frames. The reason is to assure a ductile response when these buildings are pushed beyond elastic limit in a major seismic event.

Early composite designs in the United States focused on developing moment frames by combining perimeter steel spandrel beams with encased composite columns. The moment connections, which were typically considered as fully restrained, consisted of welded perimeter steel beams to steel columns as shown in Figure 2.10. The lateral design was generally controlled by wind as the buildings were generally in areas of low seismicity. Most often the steel column with in the exterior

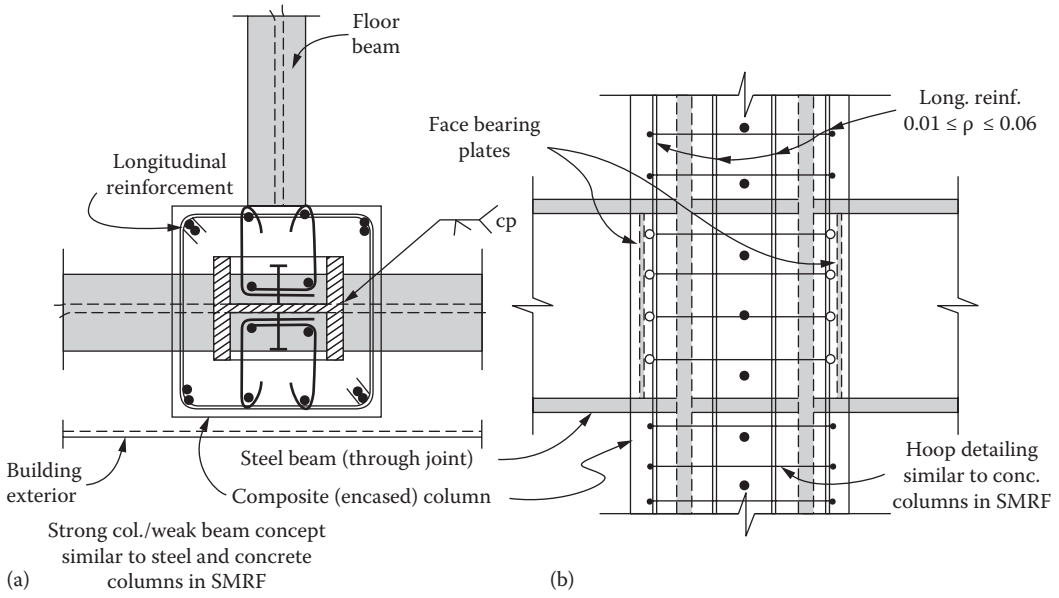


FIGURE 2.10 Spandrel beam-to-composite column connection: (a) plan; (b) elevation.

composite column was designed solely for erection loads. Although it is possible to composite interior columns, few such buildings, if any, have been built in the United States. The practice is, however, more prevalent in Japanese construction as noted previously.

2.2.1.1 Ordinary Moment Frames

The term "ordinary" refers to systems in which the members and connections are not designed to provide the maximum potential ductility. However, to provide acceptable seismic performance with a reduced ductility demand, the lateral design forces are increased over those required for the other two types of frames. Because of their limited ductility, certain restrictions are imposed on their use in buildings assigned to higher SDC. Where permitted, ordinary moment frames are often more cost-effective because the expense of detailing for ductility, far outstrips the cost of additional material required by design for higher lateral loads.

Composite moment frames may be designed by combining structural steel and reinforced concrete in a number of ways. In the United States, the most commonly used system to date in areas of low seismicity has included steel beams and composite columns. The columns may consist of encased or filled composite columns. The connections in composite ordinary moment-resisting frames are generally designed to develop the full moment capacity of the steel beams.

2.2.1.2 Special Moment Frames

The term "special" refers to the characteristics of the frame in which the members and connections are designed and detailed to provide maximum ductility and toughness, implying excellent energy dissipation and seismic performance during severe earthquake shaking. In recognition of the ductility, seismic provisions allow a maximum reduction in the design base shear. Because of the recognized ductility and the limited interference with architectural planning, special moment frames are one of the most commonly used lateral systems.

Composite special moment frames are similar in configuration to ordinary moment-resisting frames. As in steel or concrete systems, more stringent detailing provisions are required to increase the system's ductility and toughness. The commensurate reduction in design lateral force is very similar to that in steel or concrete special moment frames. The design intent is to confine inelastic hinging in beams, while the columns and connections remain essentially elastic. The design base shear prescribed for this system is similar to the special moment-resisting frame systems of steel or reinforced concrete. Likewise, no limitations have been placed on their usage in buildings assigned to a higher SDC.

Composite special moment-resisting frames are subject to the same potential failure mechanisms as experienced by special moment-resisting frame steel buildings during the Northridge, California, earthquake of 1994, and the Kobe, Japan, earthquake of 1995. The design approach for composite special moment-resting frames attempts to provide the maximum possible frame ductility, toughness, and energy-dissipation capacity. This requirement results in more stringent provisions for member and joint detailing. Generally these frames are designed to limit inelastic action to the beams, with the intent of preventing potential yielding in columns and connections.

The design and detailing provisions for composite special moment-resisting frames should incorporate all the corresponding provisions of steel and concrete special moment frames. The design should include the strong column-weak beam concept. For composite columns, transverse reinforcement requirements should be equivalent to those required for reinforced concrete columns in special moment-resisting frames. Special details are required to satisfy closed-hoop and cross-tie requirements for encased composite columns. Examples are shown in Figure 2.11.

In composite special moment frames, the beams should be designed to meet the more restrictive $b_f/2t_f$ and d/t_w compactness limits and the lateral bracing requirements of steel special moment-resisting frames. The additional restrictions are necessary to increase the resistance to local

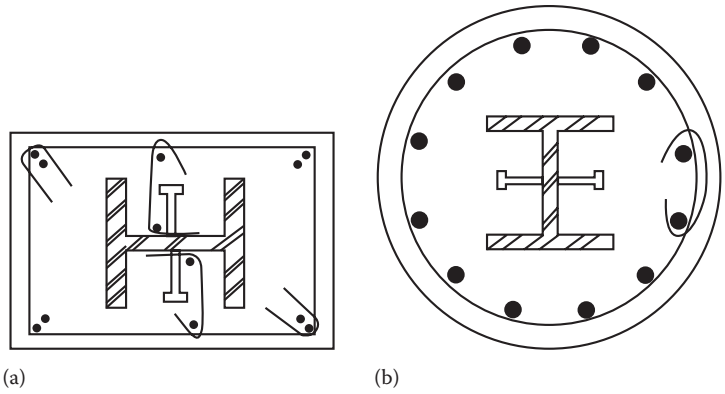


FIGURE 2.11 Seismic tie-arrangement in composite columns: (a) rectangular column; (b) circular column.

and lateral torsional buckling, allowing the beams to develop their full plastic flexural capacity. However, steel flanges connected to concrete slabs with shear connectors are exempted from this provision. This is because lateral torsion and local buckling are inhibited by the shear connectors and concrete slab.

The procedure for the analysis of composite frames, in a manner of speaking, is identical to the one that we use for steel or concrete buildings. For this purpose, the elastic properties of composite elements may be transformed into equivalent steel using standard procedures. As with steel frames, it may be more accurate to include a finite joint size in the frame model, particularly when composite columns are quite large.

The design of composite frames is not significantly different from the procedures for structural steel or reinforced concrete moment frames. Encased composite columns should have a minimum ratio of structural steel gross column area of 4%. The shear strength of columns generally ignores the contribution of concrete. However, the contribution of the shear strength of the reinforcing ties based on an effective shear width b_f of the section, as noted in Figure 2.12, is permitted. For filled composite columns; it is conservative to neglect the contribution of concrete to the shear strength of the column. Where shear strength becomes critical, the composite column may be treated as a reinforced concrete column with the steel considered as shear reinforcement. Transfer of forces between structural steel and reinforced concrete should be made through shear connectors, ignoring the contribution of bond or friction.

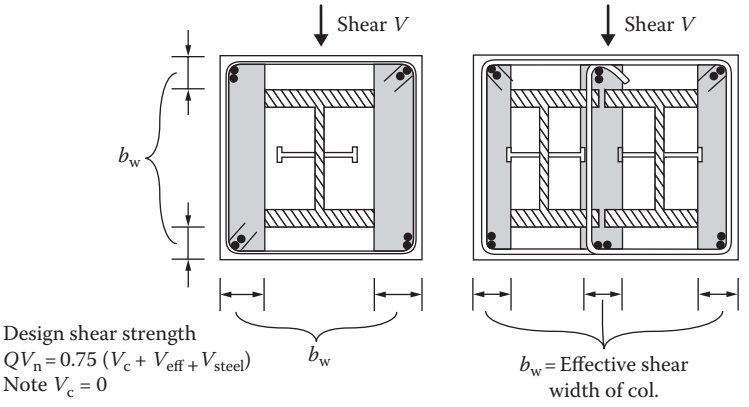


FIGURE 2.12 Encased composite column; shear design parameters.

2.2.2 Composite Braced Frames

Braced frames, by and large, are constructed in steel, although there are some examples of concrete-braced frames. Two types of composite braced frames are recognized in AISC 341-05/10: (1) Concentric bracing, where various bracing members meet at a common point; and (2) a relatively new form of braced frame called eccentric brace. Developed during the 1970s and 1980s, this system combines the ductility of moment frames with the high stiffness of concentrically braced frames. It consists of a short portion of beam, referred to as the link, designed to act as a ductile fuse to dissipate energy during seismic overloads. With a proper selection of length of link beams, the stiffness of this system can approach that of a concentrically braced frame. The ability to combine the ductility of moment frames and the stiffness of concentrically braced frames has led to increasing use of the system in buildings assigned to a higher SDC. Shown in Figure 2.13 are some examples of composite concentric braced frames.

The lateral load capacity of a concentrically braced composite frame is somewhat limited for resisting loads generated during a major seismic event. This is because its energy-dissipation capacity deteriorates during repeated inelastic cycles. In moderate earthquakes, the brace remains essentially elastic: the response is thus quite satisfactory. Certain techniques such as infilling the steel tubes used as diagonals, with concrete may inhibit the onset of local buckling which in turn will improve cyclic response.

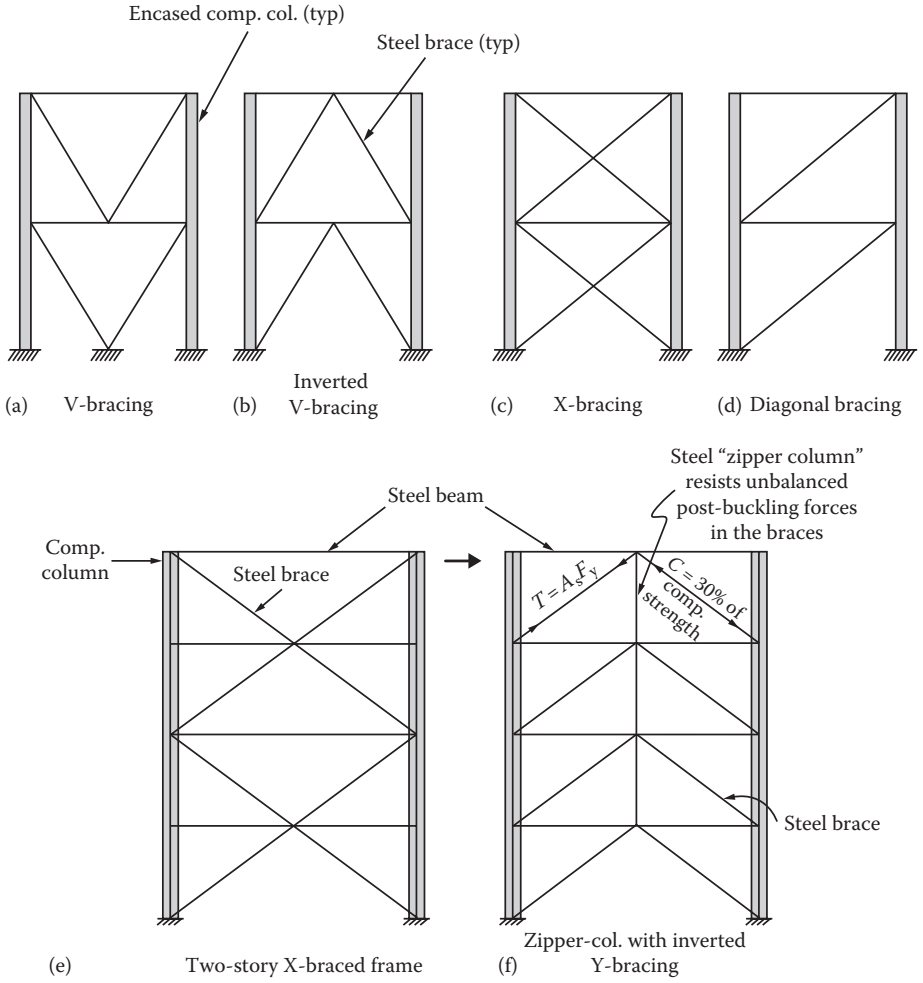


FIGURE 2.13 Composite concentrically braced frames: (a) V-bracing; (b) inverted V-bracing; (c) X-bracing; (d) diagonal bracing; (e) two-story X-bracing; (f) zipper column with inverted V-bracing.

Design of connections should be similar to that of steel-braced frames in which the connections are intended to develop the capacity of the brace elements. In composite construction, the connection design must consider the increased capacity due to the presence of concrete.

The design of composite concentrically braced frames is similar to that for steel or concrete. Encased composite columns should have a minimum ratio of structural steel to gross column area of 4%. Transfer of forces between structural steel and reinforced concrete should be made through shear connectors, ignoring the contribution of bond or friction. The capacity design of reinforced concrete columns should meet the requirements for columns in ordinary moment-resisting frames. The detailing of both composite and reinforced concrete columns should provide ductility comparable to that of ordinary moment-resisting frames.

Composite brace design in concentrically braced frames must recognize that these elements are expected to provide for the inelastic action during large seismic overloads. Braces consisting of concrete encased steel elements should include reinforcing and confinement steel sufficient to provide the intended stiffening effect even after the brace has buckled during multiple cycles of seismic motion. As a result, it is recommended that these elements should meet detailing requirements similar to those for composite columns. Composite braces in tension should be designed considering the resistance provided only by the steel.

The general intent of the connection design is to provide strength to develop the capacity of the braces in tension or compression. For composite sections, the additional strength of the concrete must be considered, since it would be unconservative to consider only the strength of the structural steel section. Brace buckling and the resulting large rotation demands at the brace ends should be considered in connection detailing. Schematic details of brace to encased composite column are shown in Figure 2.14a and b.

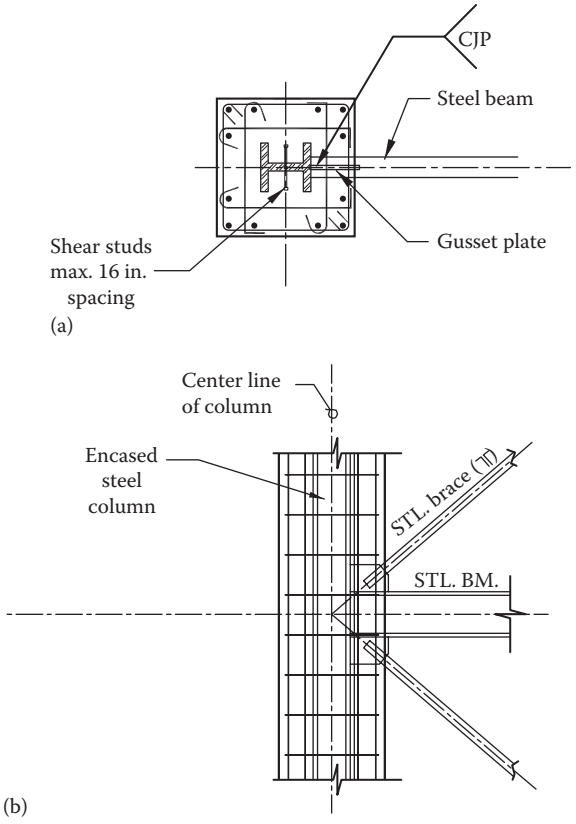


FIGURE 2.14 Composite concentric braced frame: Connection schematics; (a) plan; (b) elevation.

2.2.3 Composite Eccentrically Braced Frames

Some examples of eccentrically braced frames are shown in Figure 2.15. In general, beams in composite eccentrically braced frames consist of structural steel sections. Any concrete encasement of the beam should not extend into the link regions where large inelastic action is expected to develop (see Figure 2.16a). Columns and braces can be of either structural steel or composite construction. The analysis, design, and detailing of the system is similar to that for steel eccentrically braced frames. Since the force transfer mechanisms between the steel and concrete rely on bearing and shear friction, special attention must be paid to the design of connections to realize the intended inelastic action in the ductile links.

Composite action of concrete slab with steel link beam may become significant in determining the capacity of the link section. This should be considered in sizing the braces and columns.

The design of composite columns must consider the maximum load that will be generated by yielding and strain hardening of the link beams. Encased composite columns should have a minimum ratio of structural steel-to-gross column area of 4% unless they are designed as reinforced concrete columns. Transfer of forces between steel and reinforced concrete should be through shear connectors, ignoring the contribution of bond or friction.

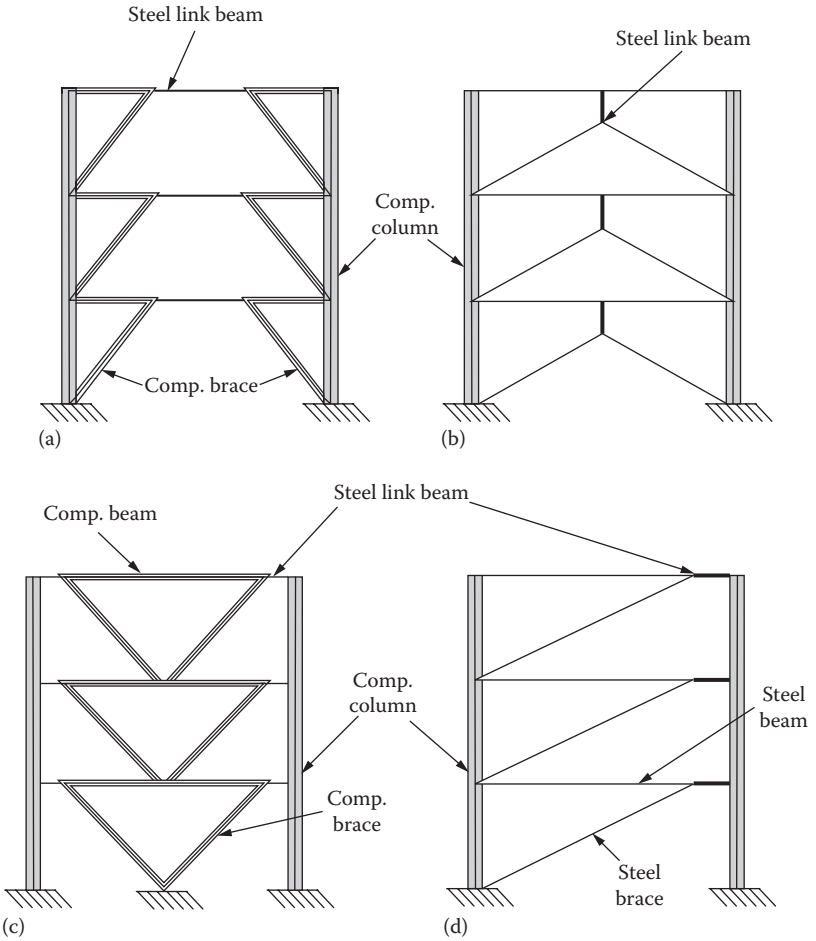


FIGURE 2.15 Examples of composite eccentrically braced frames.

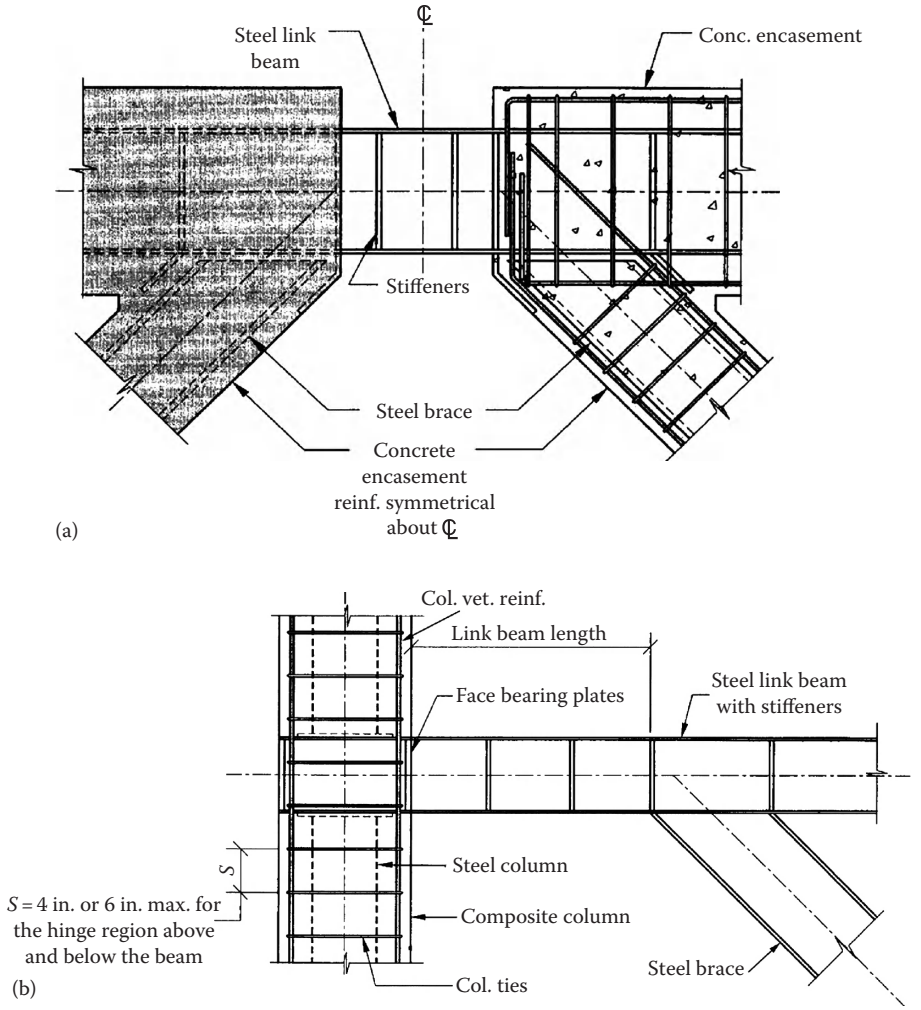


FIGURE 2.16 Schematic details of link beam: (a) link at center of beam; (b) link adjacent to column.

Composite brace design in eccentrically braced frames must recognize that these members are intended to remain essentially elastic during large seismic overloads. The design strength must consider the yielding and significant strain hardening that can occur in properly designed and detailed link elements. Both axial and bending forces generated in the braces by the strain-hardened link beams must be considered.

2.2.4 Composite Construction

Composite construction attempts to combine the merits of concrete stiffness with the speed of steel erection. The underlining motivation is to realize a lighter structural system that has the inherent stiffness of concrete.

Needless to say, this method of construction requires a high degree of construction sequencing. Typically, in composite construction, the erection of steel frame advances to a predetermined height using steel erection columns which are then encased in reinforced concrete as shown in Figure 2.17. The step-by-step process is as follows. After completion of the foundation system, the steel frame

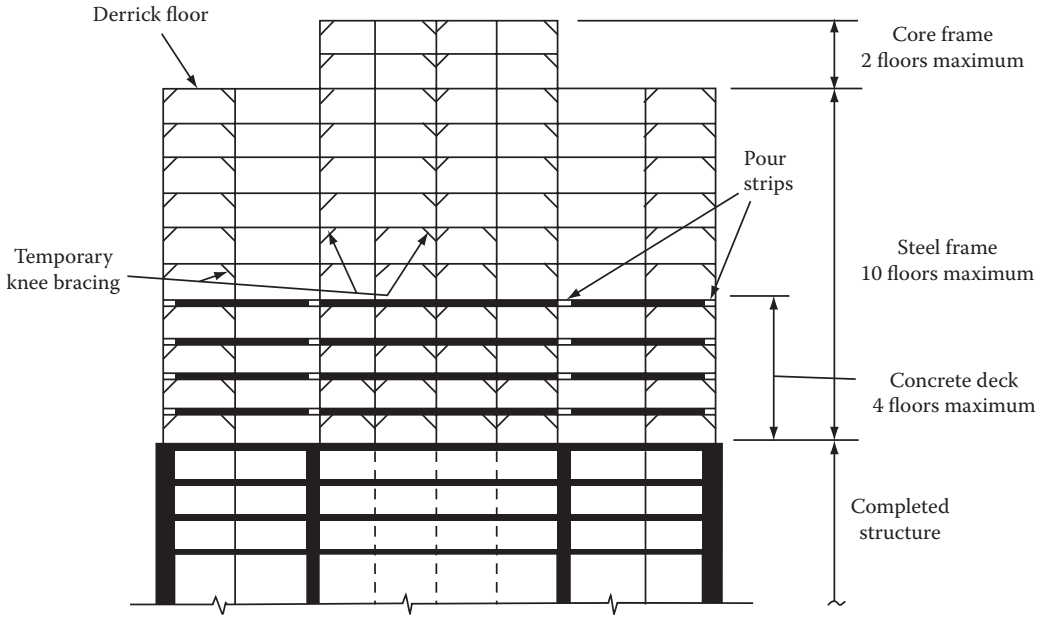


FIGURE 2.17 General construction sequence in composite structures.

erection is started using standard procedures and AISC tolerances. The general idea is to erect the steel frame prior to concreting of composite members and shear walls.

The size of erection steel columns depends on the maximum separation of the derrick floor from the concreted level. This is established during the early phase of design. Metal deck installation and welding of frame beams follows closely behind the steel erection. Diaphragm action of the deck is established by welding the metal deck to beam flanges and by installing shear studs. Concrete topping is placed on metal deck several floors above the composited level.

The floor slab serves as a platform to place concrete in the shear walls and columns. With proper sequencing of different trades, construction of a composite building with shear walls and composite columns can proceed at a pace equal to the overall speed on a conventional steel building.

2.2.5 Temporary Bracing

In a steel building, welded or bolted connections are made or braces connected between columns almost immediately behind the erection of the steel frame to provide stability and resistance to lateral loads. For composite buildings, however, final resistance and stability to design lateral loads is not achieved until concrete around erection steel has cured. Compositing of erection steel may occur anywhere from a minimum of 8 to as much as 15 floors behind the erection of the steel bare frame. The erector is now faced with the issue of dealing with a light structural steel frame projecting several stories up in the air. Traditional stability bracing such as steel cables used to stabilize and plumb the structure is typically not adequate to secure the non-self supporting steel frame sticking up several stories above the concreted level. Thus, it is apparent that additional consideration must be given by the structural engineer in concert with the erector, to define the influence of lateral loads, the response of the bare steel frame, and the stability bracing required during construction.

2.3 COMPOSITE BUILDING SYSTEMS

In Section 2.1, we discussed composite members, and in Section 2.2, we moved on to study how they are combined to develop building blocks, which we referred to as subsystems. In this section, we further discuss how the subsystems are combined to effectuate a structural system for the entire building.

In today's (2011) architecture, it is a rare event indeed, to come across a building that is regular in plan, and prismatic for the entire height. To resist lateral loads effectively and economically in such non regular buildings, it is almost always necessary to combine several subsystems in a single building.

The number of subsystems discussed in this chapter is not really that large. But it should be kept in mind that several other subsystems, not specially addressed here, are also available. Thus, the number of combinations of subsystems that may be applicable to a given building, in a manner of speaking, is infinite. Admittedly, this is an over statement but it behooves to remember that structural system for a particular building is a response to a unique set of demands. That said, let us take a look at some of the more common composite systems.

2.3.1 Reinforced Concrete Core with Steel Surround

Core walls enclosing building services such as elevators, mechanical and electric rooms, and stairs have been used extensively to resist lateral loads in tall concrete buildings. The use of simple shapes such as C and I shaped walls around elevators interconnected with coupling beams constitutes one of the most typical methods of providing resistance to lateral loads.

In the composite version of this system, a central concrete shear wall core is designed to resist the entire lateral load while the remainder of framing surrounding the core is designed for gravity loads using structural steel, metal deck and concrete topping (see Figure 2.18). The choice of construction sequence, whether the concrete core or steel surround goes up first, is often project specific. In one version, concrete core is built first, using jump or slip forms, followed by erection of steel surround, as shown in Figure 2.19. Although structural steel erection may not proceed as fast as in a conventional steel building, the overall construction time is likely to be less because the building's vertical transportation, consisting of stairs and elevators and mechanical and electrical services can be installed with in the core while erection of steel outside of the core is still in progress. In another version, steel erection columns within the shear walls serve as erection columns, and erection of steel for the entire building proceeds as in a conventional steel building. After the steel erection has reached a predetermined level, concreting of the core takes place using conventional techniques. To facilitate jumping of forms from one level to the next through the floors that are already in place, temporary openings are provided in the floor framing.

In this system, the steel surround is designed as a simple framing for gravity loads only. Since there are no moment connections with welding or heavy bolting, erection of steel proceeds much faster. The only nonstandard connection is between shear walls and floor beams. Various techniques have been developed for this connection, chief among them, are the embedded plate and pocket details, as shown in Figure 2.20. The floor construction invariably consists of composite metal deck with concrete topping. Since steel columns carry only gravity loads they tend to be small, both at the building interior and exterior, thus increasing the space-planning potential.

The structural behavior of this type of building is no different from that of a concrete building designed to resist the entire lateral forces in the core. However, to supplement the torsional resistance of the concrete core, it is advisable to provide a reasonable lateral bracing system at the building perimeter. If the entire lateral load including those due to torsion is resisted by the central core, the steel surround may be designed as simple framing for gravity loads only. Since there are no moment connections, steel erection proceeds much faster.

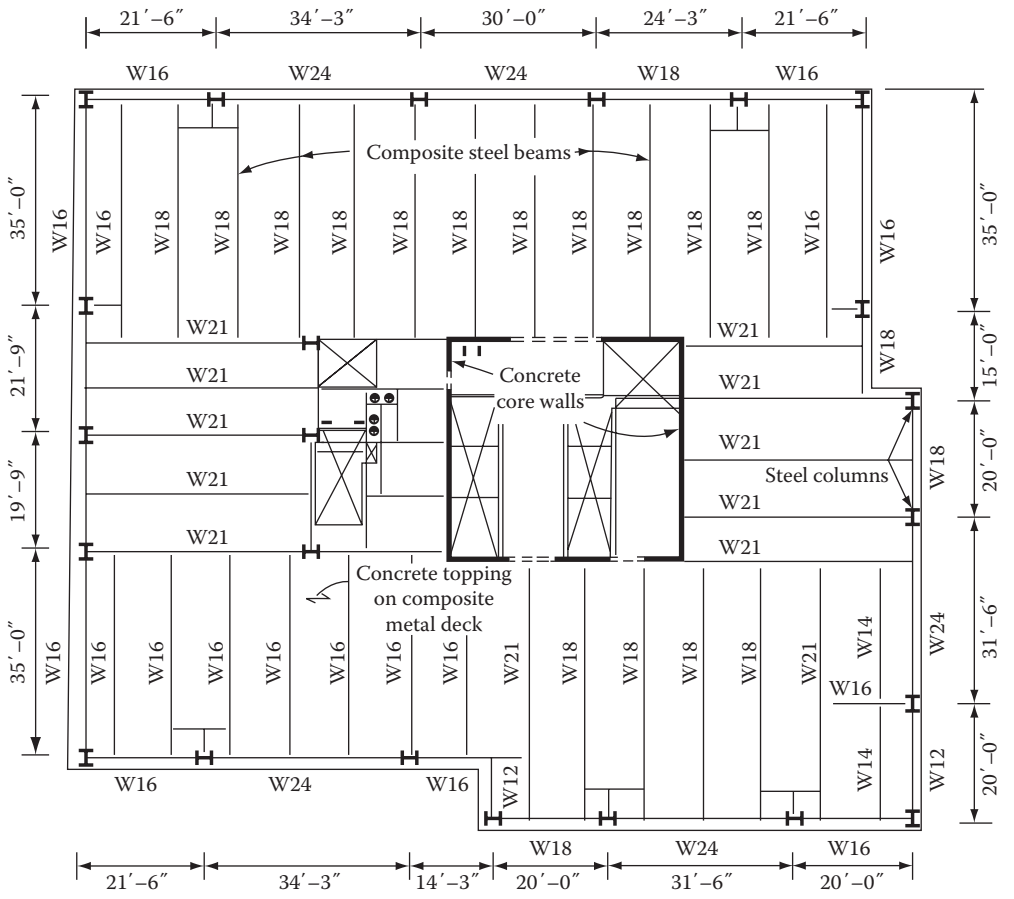


FIGURE 2.18 Typical floor plan, building with central core and steel surround.



(a) (b)

FIGURE 2.19 Core supported composite building: (a) concrete core; (b) core with steel surround.

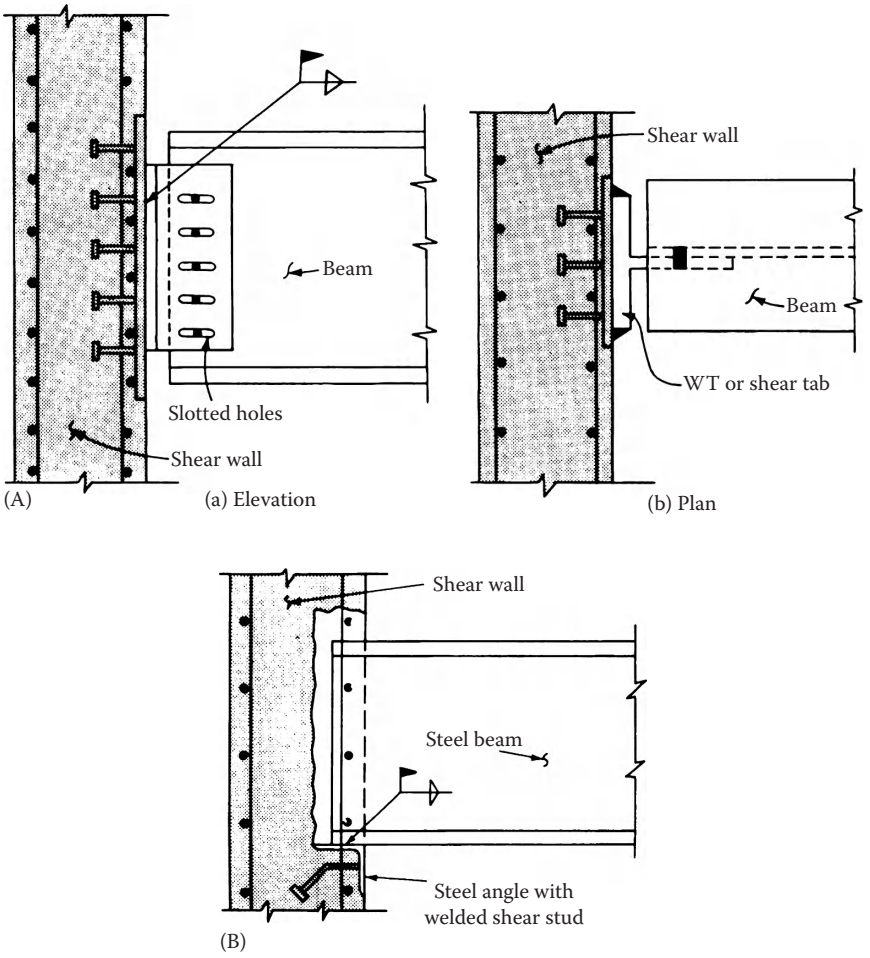


FIGURE 2.20 Beam-to-shear wall connection: (A) embedded plate detail; (a) elevation; (b) plan; (B) pocket detail.

The floor within the core may be constructed either in concrete or structural steel. The connection between the floor slab and core walls is often project specific. The weld plate detail shown in Figure 2.20a is, however, the most popular, particularly in a slip-formed construction. The weld plates are set with the outer surface flush with the wall surface. The plate is anchored to the wall by shear connectors welded to the plate. Experience in slip-form construction indicates that it is prudent to oversize the plates to compensate for misalignment. Subsequent to the installation of weld plates, structural tee or shear tab connections with slotted holes are field welded to the embed plates.

Slip forming is a special construction technique that uses a mechanized moving platform system. The process of slip forming is similar to an extrusion process. The difference is, in an extrusion process the extrusion moves; in a slip-forming process the die moves while the extrusion, namely the concrete, remains stationary. The average upward speed of slip form is typically 6–18 in./h.

2.3.2 Shear Wall-Frame Interacting Systems

This system has applications in buildings that do not have sufficiently large cores to resist the entire lateral loads. This may require interaction of shear walls with moment frames to supplement the lateral stiffness of the shear core. When located at the interior, the moment frames are usually of

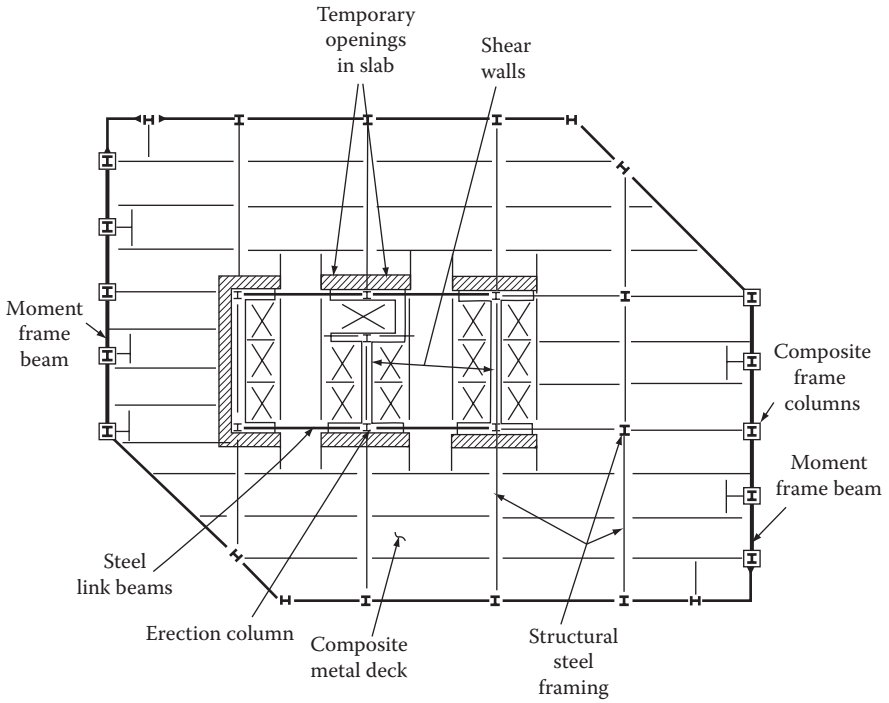


FIGURE 2.21 Typical floor plan showing interacting shear walls and moment frames.

steel construction because the cost of formwork for compositing interior columns far outstrips the advantages gained by additional strength and stiffness. Because the interior columns typically have beams framing in two directions, placement of mild steel reinforcement and formwork around the columns tend to be extremely cumbersome. On the other hand, it is relatively easy to place formwork around the exterior columns. And if desired, reinforced concrete may be cast around the exterior spandrels without undue complexity. A schematic plan of a building with interacting shear walls and composite moment frames is shown in Figure 2.21. Also shown in the figure are temporary openings in slabs provided for jumping of formwork through the floors.

2.3.3 Composite Tube Systems

A framing system originally used extensively in Louisiana and Texas is the so-called *composite concrete tube*. It combines the well-known virtues of tube system with the speed of steel construction. As in concrete or steel construction, closely spaced perimeter columns and deep spandrels constitute the backbone of the system. Two versions are popular in North America. In the first version, composite columns and composite spandrels are used, and in the other, composite columns and steel spandrels are used.

In the first version, a light steel section is used as erection beam to stabilize the steel columns. However, in the design of concrete spandrel, its strength and stiffness contribution are generally neglected. Schematic plan and sections for the two versions of composite tubes are shown in Figures 2.22 and 2.23. In either system, speed of construction, rivaling that of an all-steel building, is maintained by first erecting the steel skeleton, and then encasing the exterior columns with reinforced concrete. In a typical construction, the steel frame is erected some 10–12 stories ahead of compositing of the perimeter columns. The key to the success of tube construction as noted earlier, lies in the rigidity of closely spaced exterior composite columns and deep spandrels. This results in an exterior façade that behaves more like a bearing wall than as a moment frame.

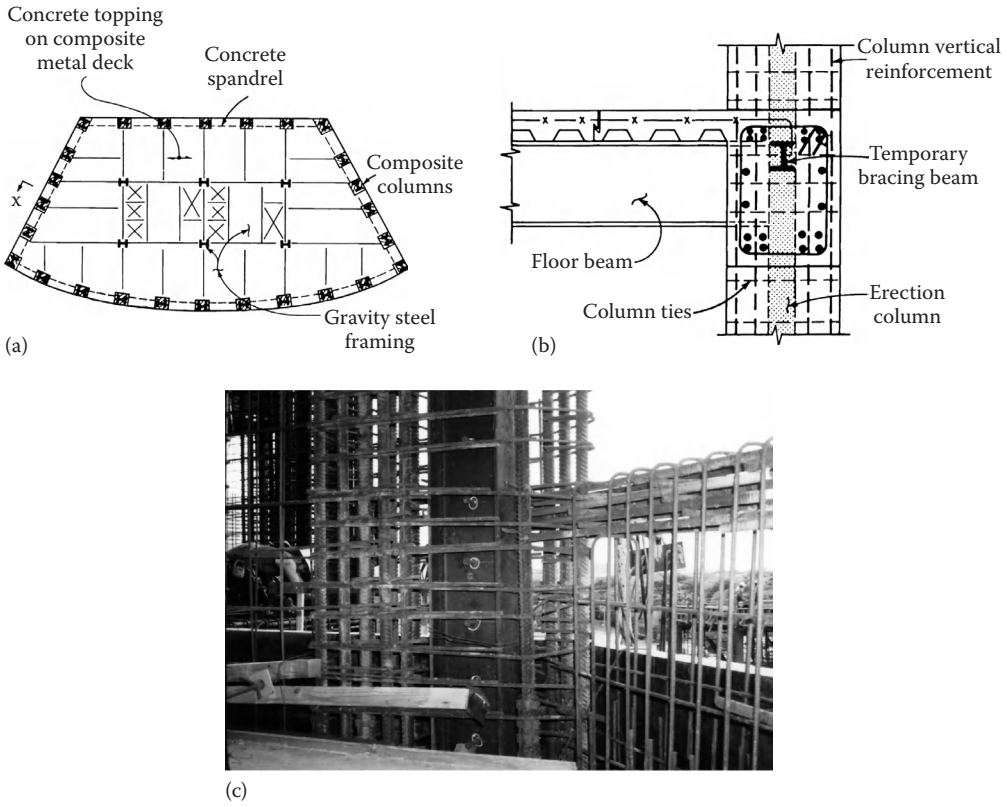


FIGURE 2.22 Composite tube building with concrete spandrels: (a) floor plan; (b) section at spandrel; (c) photograph of composite column and concrete spandrel.

2.3.4 Vertically Mixed Systems

Mixed-use buildings are those that provide for two or more types of occupancies in a single building. For example, lower levels of the building may be for parking; middle levels for office floors; and the top levels for residential units, such as apartments and hotel rooms. Therefore, it makes economical sense to stack up different systems vertically up the building height using a system that is most logical for the particular occupancy. For example, beamless flat ceilings with a minimum of floor-to-floor height are preferred in residential occupancies. Additionally, large spans of the order of 40 ft (12.2 m) required for lease space in office buildings are much too large for apartments. Therefore, additional columns can be introduced without unduly affecting the layout of residential units. The decrease in span combined with the desire for shallow construction depth, almost always results in the use of reinforced concrete flat plate in residential occupancies, typically in the upper levels.

In certain types of buildings, however, the use of reinforced concrete for the lower levels and structural steel for the upper levels may result in an optimum structural system. As an example, shown in Figures 2.24 and 2.25 are schematics of a building constructed in Houston, Texas, circa 1980. The bracing for the concrete portion of the building is provided by the shear walls, while a braced steel core provides for the lateral stability of the upper levels. A suggested technique for transition of steel columns into concrete columns is to embed steel column into the reinforced concrete column below the transition level. Transfer of axial loads from steel columns to concrete columns occurs through friction and bearing of shear studs welded to steel columns.

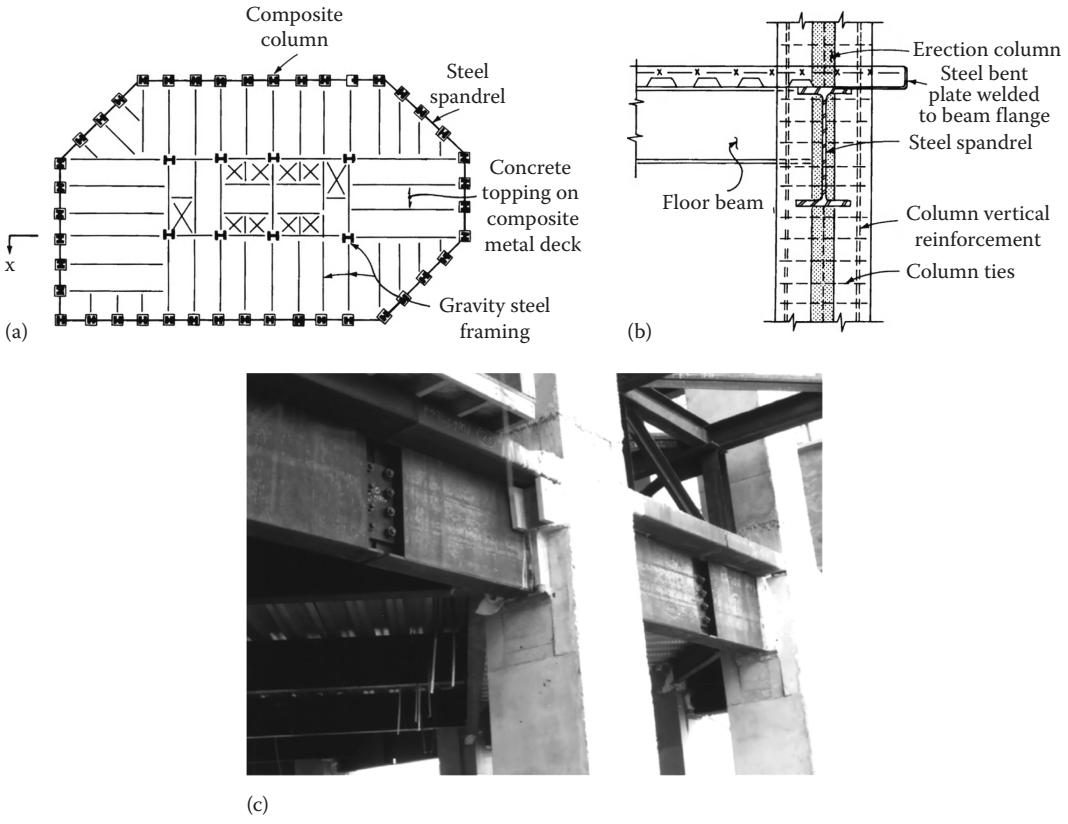


FIGURE 2.23 Composite tube building with steel spandrels: (a) floor plan; (b) section at spandrel; (c) photograph of composite column and steel spandrel.

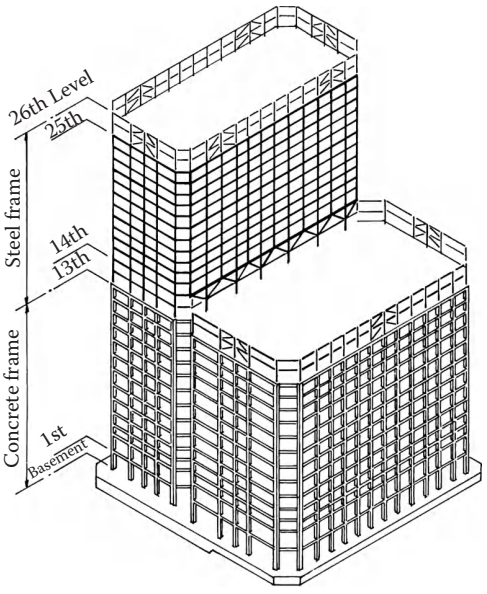


FIGURE 2.24 Vertically mixed system: Schematic perimeter framing.

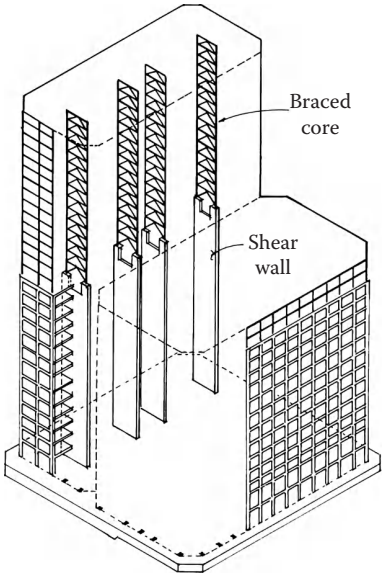


FIGURE 2.25 Vertically mixed system: Schematic bracing.

2.3.5 Mega Frames with Super Columns

One of the most efficient methods of resisting lateral loads in ultra-tall buildings is to provide "super columns," placed as far apart as possible preferably at the perimeter of the building, and to interconnect the columns with a shear-resisting system such as Vierendeel frames, or super diagonals. This idea developed in the 1990s has given rise to a whole new vocabulary for composite systems characterized by their use of super columns.

The construction of super columns can take on many forms. One system uses large-diameter steel pipes filled with high-strength concrete. Generally, neither longitudinal nor transverse reinforcement is used in the columns, thus simplifying construction. Another method as mentioned previously is to encase steel columns using conventional forming techniques.

2.3.6 High-Efficiency Structure: Structural Concept

An ultra-tall building may be defined as a skyscraper when its silhouette has a slender form with a height-to-width ratio well in excess of 10. The slender proportion imposes engineering challenges far greater than those for its lower brothern, requiring the structural engineer to come up with innovative lateral bracing schemes. The ideal structural form is one that can at once resist the effects of bending, torsion, shear, and vibration in a unified manner. A perfect form is a chimney with walls located at the farthest extremity from its center, but as an architectural form it is less than inspiring for a building application. The next best and a more practical form is a skeletal structure with columns located at the farthest extremity from the building center. However, two additional requirements need to be addressed within this basic concept to achieve high efficiency: (1) transfer as much gravity load, preferably all the gravity load into these columns to enhance their capacity for resisting tensile forces due to overturning effects; and (2) inter-connect the columns with a system capable of resisting the entire shear due to lateral loads. The ultimate structure for a rectangular building, then, will have just four corner columns interconnected with a shear resisting system in both directions. Such a concept, proposed by the author for a super tall building, is shown in Figure 2.26, in which the total gravity loads and overturning moments are resisted by four composite columns. The columns are intentionally located in-board from the building corners to allow for architectural freedom in modulating the building façade. The shear in the transverse direction is resisted by a

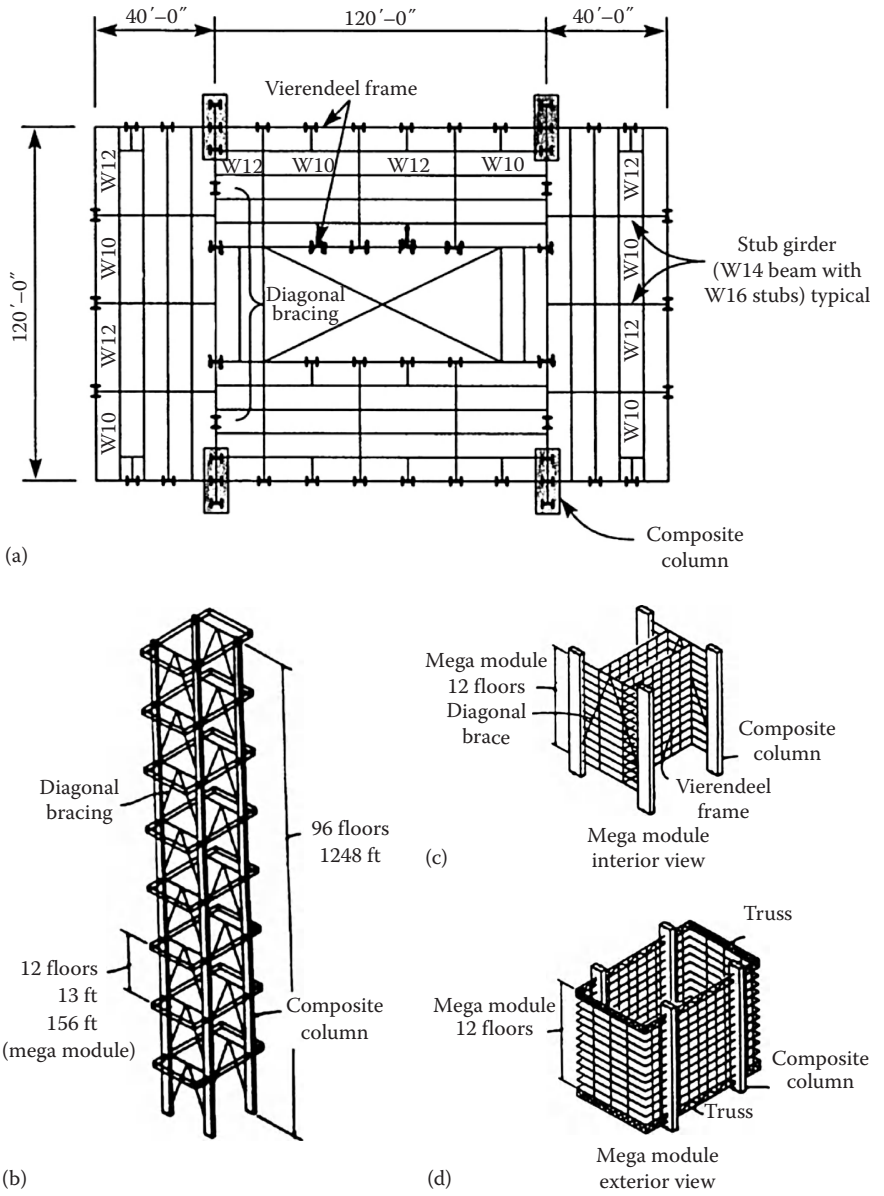


FIGURE 2.26 Structural concept for a super tall building: (a) plan; (b) schematic elevation; (c) interior schematics of mega module; (d) exterior schematics of mega module.

system of 12-story-high chevron composite braces while in the longitudinal direction the shear resistance is primarily provided by the full height Vierendeel frames located on the long faces. A system of story-high trusses located at say, every 12th floor permit cantilevering of the floors, thus acknowledging the current trend in architectural modulation of the building perimeter. The primary function of the interior Vierendeel frame is to transfer the gravity loads of the interior columns to the exterior composite columns via chevron braces. However, because of its geometry, it also resists lateral shear in the long direction.

Any number of floor framing techniques such as composite beams, haunch girders, or stub girders may be used to span the distance from the core to the building perimeter. The scheme in Figure 2.26 shows stub girders consisting of a W14 wide flange beams with W16 stubs welded to the top

flange of W14 beams. This system discussed in Chapter 4 has the advantage of reducing both the floor-to-floor height and the quantity of structural steel required for floor framing.

The author believes that the scheme shown in Figure 2.26 can be modified readily for a variety of architectural building shapes. Any desired slicing and dicing of the building on the short faces may be accommodated without inflicting an undue penalty on systems efficiency. The structural concept is believed to be complete for preliminary designs. However, it has to await its time before finding application to a real building.

2.4 SEISMIC DESIGN OF COMPOSITE BUILDINGS

The progress of composite systems in regions of high seismicity has been the least because of the difficulties associated with the detailing of joints. Until recently, joints for composite members were designed as for steel structures without regard to the structural concrete encasing the steel sections. However, studies reported over the years from Japan, concerned principally with the resistance of joints to earthquake forces, have given new impetus to composite construction.

With proper detailing, a composite building may be designed to perform in a ductile manner similar to an all-steel or reinforced concrete building. This characteristic has made composite design quite popular in Japan. The acceptance of composite construction in California, another earthquake country, has been less enthusiastic. However, in the rest of the United States and around the world, composite construction has in the past two decades gotten a strong foot hold. Many ultra-tall buildings constructed recently in Shanghai, China are of composite construction.

Because composite systems are assemblies of steel and concrete components, their design is governed by both AISC and ACI specifications. The available research and experience has demonstrated that properly detailed composite members and connections can perform reliably when subjected to seismic ground motions. So much is the confidence in composite construction, that as many as nine distinct composite systems are codified in the AISC 341-05 and 341-10. However, since experience with composite buildings subjected to extreme seismic forces is limited, careful attention to all design aspects is necessary, particularly in the detailing of the connections.

When discussing composite systems, it is worthwhile to revisit the frequently used phrase "design for wind and seismic forces." A clear understanding of the difference between the two is critical for proper implementation of design.

It is quite evident that the greater of the two forces calculated for wind or seismic is used for the design of lateral-force-resisting system. However, in seismic design, it is recognized that actual seismic forces occuring during a major earthquake are many times more than code-prescribed values. Thus, seismic design must include not only strength requirements but also material and system limitations and special provision for member proportioning and detailing. The purpose is to ensure that the member and joints do not snap in a large seismic event, but have the necessary ductility to ride out the forces. Therefore, when designing a building assigned to a high SDC, even when wind forces govern the design, the detailing and proportioning requirements for seismic resistance must also be satisfied.

The seismic provisions governing the design, fabrication, and erection of seismic composite systems and connections are given in the AISC 341-05, commonly referred to as AISC Seismic. These provisions need only be applied when the seismic response modification factor, R, is taken greater than 3 or when the building is assigned to a Seismic Design Category D or higher. Such composite systems in addition to meeting the requirements of AISC 360-05/10 and AISC 341-05/10 shall also meet the requirements of ACI 318-08 for reinforced concrete members.

3 Gravity Systems for Steel Buildings

PREVIEW

We start this chapter with a brief historical review of cast iron and steel components of building Structures. Then, design of compression members, the most common of which are the vertical members supporting floors in buildings, is considered, followed by a review of their effective length and flexural buckling stress. We then discuss design of wide flange sections for flexure including floor beams acting compositely with metal deck and concrete topping. For these beams with continuously braced top flange, the nominal moment capacity, $M_p = ZF_y$. This need not be reduced for unbraced length effects because the top flange is laterally braced for the entire length of the beam by metal deck and concrete topping. Serviceability considerations such as deflection limitations are also discussed. Next, we discuss design of members for tension highlighting the concepts of tensile yield and tensile rupture strengths by noting that it is not unusual for tensile rupture strength to govern the design of a tension member, particularly for small members with holes of heavier sections with multiple rows of holes. The next discussion centers on design of members for shear with and without consideration of post-buckling strength of web. This is followed by a presentation of design for combined forces and torsion. The emphasis is on linear interaction equation commonly called the unity equation, in which the ratios of calculated forces are compared to available capacities. The chapter concludes with an in-depth discussion of design for stability.

3.1 GENERAL CONSIDERATIONS

3.1.1 Steel and Cast Iron: Historical Perspective

Iron and steel have been used in the construction of buildings for centuries. Cast iron first developed as early as 200 BC, was produced in significant quantities in the United States during the late eighteenth century and throughout the nineteenth century. It has a relatively high carbon content (more than 1.5%) along with silicon and sulfur. As a result, cast iron is hard and brittle, with limited tensile strength. It is difficult to work, so it must normally be used in cast assemblies. Because of its availability and good compressive strength, it was used quite extensively for columns in buildings built in the early to middle nineteenth century. Engineers preferred not to use cast iron in components that were either part of a lateral load system or developed significant bending or tension, because of brittle and dramatic failures of cast iron components in bridges and other similar structures. Cast iron continued to be used into the early twentieth century, but wrought iron became the more dominant material in the late nineteenth century, and steel overtook both in the early 1900s.

Wrought iron was first developed through the hand puddle process in 1613. The metal produced by this process was somewhat variable, depending upon the skill of the producer, and only relatively small quantities of metal could be produced. As a result, this early wrought iron was used in buildings built before 1850, but not as major structural elements because of the small volume that could be produced during that time. Mechanical methods for producing larger quantities of wrought iron were developed in the mid-1800s, resulting in its use in structural systems of a substantial number of buildings in the late 1800s and early 1900s.

Wrought iron is much more workable than cast iron; it is more ductile and has better tensile capacity. As a result, it was a more versatile construction material than the cast iron that preceded it. For columns, however, cast iron was still viewed as the most economical material until very late in the 1800s.

Steel was largely made possible by the development of the Bessemer process combined with the open hearth furnace. The Bessemer process was patented in 1856, but steel does not appear to have become commonly available until about 1880. This delay was partly due to some legal disputes, as well as fundamental concerns about the properties and quality of the material. In 1880, wrought iron still dominated the structural market, and buildings built in the mid-1890s were still most likely to be built of wrought iron (possibly with cast iron columns) rather than steel, but most engineers of that period believed that low carbon structural steel was the superior material and would dominate future building construction.

In 1894–1895, the first specification for structural steel was published. This document did not address building design, but established quality control and standardization requirements for the material. In 1896, the steel manufacturers agreed to establish standardization in the shapes that they produced, and steel proceeded to totally dominate the structural market during the next 10 years.

A number of tests for steel and structural steel components are reported during the 1890s. Examination of the reported test results suggests that the properties of this early steel were not very different from the ASTM-A36 steel used in the 1950s and 1960s. The yield stress may have been somewhat lower, and the early standard designation for this mild steel was A9 with a nominal yield stress of 30 ksi. In the late 1890s, as a result of fire tests on steel members, engineers became concerned about fire protection. Masonry therefore, was used to enclose the steel to provide fire protection in some early buildings, but concrete encasement became the predominant form of fire protection at about the start of the twentieth century. Riveted connections were the primary method for connecting both wrought iron and steel members during that period.

Steel construction proceeded in a fairly continuous manner in the following years. There was however, quite a wide variation in the structures and the materials because of particular requirements of the designer. Welding techniques were first developed around 1915 and used in a few structures in the 1920s and 1930s, but usage was limited due to poor quality. Mild steel bolts also had limited usage during this period, and A7 steel with a nominal yield stress of 33 ksi arrived on the scene, essentially replacing ASTM-A9 by 1940. Further standards for steel and steel products were developed, largely due to the efforts of the American Institute of Steel Construction (AISC), established in the 1920s. This second wave of standardization, with the structural designer involved in the process, resulted in greater uniformity in both the steel and structural steel shapes as well as the structural designs themselves.

Some of the early welding techniques employed gas welding, but electric arc welding was also developed in the very early 1900s. During the 1930s, the use of flux and shielding of the arc began. Some structural tests on welded components were performed starting in the 1930s, and electric arc welding became common in the 1940s and 1950s. By the mid-1960s, the use of riveted connections was abandoned as high-strength bolts and electric arc welding became the standard connection technique.

Around this time, concrete encasement for fire protection was also disappearing in favor of lighter insulation methods, and A36 steel with a yield stress of 36 ksi became the standard steel. Higher-strength steels were also introduced during this period.

3.1.1.1 Chronology of Steel Buildings

Due to the brittle nature of iron, it was not possible to produce shapes by hot or cold working. As a result, iron shapes for columns were cast and often patented.

Due to lack of good quality control, cast pieces often had inclusions; this greatly reduced the allowable stress for cast iron columns.

Cast iron was used extensively throughout the nineteenth century primarily for columns to carry compression with no significant tension or bending. It performed poorly when subjected to these

alternate stress states. Therefore, wrought iron filled in as an alternate construction material for these other applications in the second half of the 1800s. However, both wrought iron and cast iron were largely replaced by steel at the turn of the century.

Wrought iron and steel were more ductile than cast iron and more easily worked. It made a wide range of field and shop modifications possible.

These wrought iron and steel buildings had some common attributes, but in general, the members and connections were unique. Engineers made extensive use of riveted built-up steel and wrought iron members with riveted connections. The members were commonly built up from plates, angles, and channels. These built-up members used tie plates and lacing and the resulting large number of rivets made them labor intensive. Connections were formed with haunches, knee braces, and large gusset plates. The first effort to standardize steel materials and shapes was made in 1895, but there was relatively little standardization in design. Engineers would use their own unique member and connection configurations. Further, the design was controlled by local practice and city building codes. As a result, the predicted strength of the member varied widely.

The first proposed structural design specification for steel buildings was published by ASCE in an article in the 1920s. This article examined the wide variation in design loads and stress limits, and proposed a standard design procedure for the first time. This pioneering article led to the development of the AISC specification and design manual in the 1920s.

While members and connections were quite variable, there was a lot of similarity in the general structural aspects of these older buildings. First, they usually had massive fire protection. Massive—but lightly reinforced—concrete was used in most building constructed after 1900. In addition, these buildings usually had unreinforced masonry for outside walls and unreinforced clay tile or masonry partitions throughout the interior. These walls and partitions provided the bulk of the strength and stiffness of these older buildings for resisting lateral loads. These buildings were normally designed for wind but not seismic. They were designed as moment frames, with the tacit understanding that infilled walls help to resist lateral loads.

It should be noted that engineers in this era readily shifted designs from one material to another. Concrete encasement was not considered in the evaluation of the strength of steel structures, but it was extensively used as a transition between steel and concrete construction.

3.1.1.2 1920 through 1950

In the 1920s, the use of the unique, complex built-up, members began to be phased out, and standard I and H shapes replaced them as the standard for member design. Partially restrained (PR) connections, such as the riveted T-stub and clip angle connections dominated the construction scene. Because clip angle connections were weaker and more flexible, they were used as beam column connections in shorter buildings or in the top stories of taller buildings. The T-stub connection was stiffer and stronger, and was used in the lower floors of taller buildings where the connection moments were larger. Stiffened angle or T-stub connections were often used to provide beam connections to the weak axis of the column.

Buildings constructed in regions regarded as seismically active were designed for seismic loads, but the design forces were invariably lower than those required today. However, the walls and partitions were not included in the design calculations, although they provided the bulk of the strength and stiffness of these buildings. Buildings outside of regions of known seismic activity were designed for wind load only.

It should be noted that all buildings constructed during this era used relatively simple design calculations compared to modern buildings. Engineers frequently resorted to observations from past building performance and standard practice; the sophisticated computer calculations used in modern structures were unknown.

Bolts and welding were sometimes used, but rivets were clearly the dominant connection. They were designed as moment frames, although actual structural behavior was strongly influenced by masonry infills and partitions.

3.1.1.3 1950 through 1970

Significant changes began to appear during this period. The use of rivets was discontinued in favor of high-strength bolts and welding. In the very first structures, bolts were merely used to replace the rivets in connections such as the clip angle and T-stub connection. However, flange plate and end plate connections were used more frequently. Increased use of and confidence in welding made these connections possible. By using these connections, engineers were often able to develop greater connection strength and stiffness with less labor. Another important change was the replacement of standard concrete fire protection by more modern lightweight materials.

Two more changes are notable. First, the masonry and clay tile walls were less frequently used for cladding and partitions, reducing building weight, although the architectural elements were still significantly heavier and stiffer than those used in steel frames today. However, these panels and finishes were more likely to be attached to the structure rather than being used as an infill to the frame. As a result, buildings built during this era are less able to utilize this added strength and stiffness than are the older structures. Secondly, significant differences began to evolve in the way buildings were designed in regions of high seismic activity. These regional differences were developed because high seismic zones had to deal with larger lateral forces, and also because of the increased emphasis on ductility in seismic design procedures. In less seismically active zones, the weaker, more flexible connections were retained for a longer period of time, while in the seismically active zones the fully restrained connection began to evolve. Also, braced frames and alternate structural systems were used because they could often achieve much greater strength with less steel and more economical connections.

3.1.1.4 1970 to Present

The trends established in the 1960s continued into the following period. First, there was increased emphasis on lightweight fire protection and slender architectural elements. As a result, the reserve strength and stiffness provided by these elements was reduced.

Second, there was increased emphasis on ductility in seismic design; and extensive rules intended to assure ductility for moment frames, braced frames, and other structural systems were established. These rules undoubtedly had some substantial benefit, but compliance was often expensive, and there was a distinct tendency toward using structures with less redundancy, since these less-redundant structures required satisfaction of the ductility criteria at fewer locations. This reduced redundancy also resulted in larger member and connection sizes. This separation of the practice between regions with significant seismic design requirements, and those with little or no seismic design requirements, continued to widen. The less seismically active regions sometimes retained more flexible connections with great redundancy in the overall structure.

Third, seismic design forces were appearing for the first time in many parts of the United States, and they increased significantly for all parts of the country for some structural systems. Finally, the steel and construction processes themselves were also changing. There was a significant increase in steel produced by reprocessing scrap metal in an electric furnace. As a result, the yield stress of steels increased, while tensile stress remained relatively stable. Welding evolved from the relatively expensive stick welding shielded arc process to the quicker and more economical flux core, gas shield, and dual shield processes. High-strength bolts were increasingly used as slip-critical friction bolts; however, quality control variations caused by tightening and installation became a major concern. These changes in turn produced changes in the ductility and behavior of many steel structures.

Today, in 2011, we basically have three groups of structural steel available for use in bridges and buildings:

1. Carbon steel: American Society for Testing and Materials (ASTM) A36, A500, and A529
2. High-strength, low-alloy steels: ASTM-A572, A618, A913, and A992
3. Corrosion-resistant, high-strength, low-alloy steels: ASTM-A242, A588, and A847

In the A572 category, five grades of steel—42, 50, 55, 60, and 65—are available for structural use. The grade numbers correspond to the minimum yield point in ksi, kilo pounds (kilos) per square inch of the specified steel. Carbon steel is available in grades 35 to 55.

Steel buildings in the United States are designed per AISC specifications, which were first published in 1923. The specifications are revised periodically to keep pace with new research findings and the availability of new materials. Steel construction for buildings is commonly referred to as steel skeleton framing, signifying that a majority of the members consist of linear structural elements such as beams and columns.

The rules for the design of structural steel members subject to any one or a combination of stress conditions due to bending, shear, axial tension, axial compression, and web crippling are given in the AISC specifications. Members may be designed by the Allowable Stress Method (ASD) or by the Load Resistance Factor Design (LRFD). Only LRFD is considered in this book using ANSI/AISC 360-05, Specification for Structural Steel Buildings, dated March 9, 2005.

3.1.2 Gravity Loads

Gravity loads on buildings are of two kinds: (1) static and (2) dynamic. Static load is considered permanent, whereas the dynamic load as time dependent. The weight of every element within the structure is a static load and includes weights of load-bearing elements, beams, slabs, columns, walls, ceiling, floor and wall finishes, sprinkler systems, light fixtures, sheet metal ducts, permanent partitions, exterior cladding, cooling towers, central plants, pump rooms, thermal storage tanks, and other mechanical equipment.

Although their effect is similar to dead loads and essentially static, live loads are less accurately predictable because they are subject to greater variation. Most often in North America a minimum value of 50 psf (2394 Pa) for live load plus 20 psf (958 Pa) for partition allowance is used for lease space in the design of speculative office buildings. Often times if tenants' requirements are known prior to the design of the building, allowance is made for expected usage of heavier loads such as book stacks, filing cabinets, computers, and business machines. However, it is becoming increasingly common to design a lease depth of 20 ft (6.0 m) from the building exterior for the minimum load, while the remainder of lease depth is designed for a heavier load of say, 100 psf (4788 Pa). The rationale is that the 20 ft (60 m) deep space adjacent to the building exterior invariably consists of office space with light furniture, whereas heavily loaded areas such as storage and computer rooms are tucked away in an area closer to the building core.

The live loads given in building codes have built-in empirical safety factors to account for maximum possible loading conditions and given in the form of equivalent uniform or concentrated loads. Although there is a common understanding among engineers that the code live loads are rather conservative, invariably the design is done to comply with the codes. However, when the actual layout and height of partitions are known, an attempt is made to justify a lower partition load. This is done more to justify the load-carrying capacity of an existing framing than to take advantage of load reduction in the design of a new building.

Concentrated loads indicate possible single-load action at critical locations and are in addition to the uniform distributed load. It is perhaps obvious that the chances of having the full occupancy load simultaneously on every square foot of a large area of a building are next to zero. The larger the area under consideration, the less is the potential for having full occupancy load. This probability aspect is taken into consideration in the codes by the use of live load reduction factors. However, past failures of hung, long-span structures have brought to light the necessity of taking extra precautions, especially in the detailing of connections to account for extraordinary situations, such as people crowding because of ceremonies, parties, and fire drills.

3.1.3 Design Load Combinations

The loads and load combinations are typically taken from the applicable building code. In the absence of a specific local, regional, or national code, the load combinations and the nominal loads specified in SEI/ASCE 7-05 *Minimum Design Loads for Buildings and Other Structures* are used as a basis of design.

The load combinations of Section 2.3, *ASCE 7* are based on probabilistic load modeling and a comprehensive survey of reliabilities inherent in traditional design practice. These load combinations utilize a "principal action-companion action format," which is based on the idea that the maximum combined load effect occurs when one of the loads takes on its maximum value while the other *variable loads* are an "arbitrary point-in-time" values. The dead load, which is considered to be permanent, is the same for all combinations in which the load effects are additive. The load factors reflect uncertainty in individual load magnitudes. The nominal loads of ASCE 7 are substantially in excess of the arbitrary point-in-time values. The nominal live, wind and snow loads historically have been associated with mean return periods of approximately 50 years, while the nominal earthquake effect is associated with a mean return period of approximately 2500 years. To avoid having to specify both a maximum and an arbitrary point-in-time value for each load type, some of the specified load factors are less than unity.

The load combinations given in ASCE 7, Section 2.3 apply only to design for strength limit states. These do not account for gross error or negligence.

Serviceability load combinations. Serviceability limit states and associated load factors are covered in Appendix B of ASCE 7-05. That Appendix contains a number of suggested load combinations for checking serviceability. While the nominal loads appearing in those equations are defined in Sections 3 through 7 of ASCE 7, the performance objectives for serviceability checking are different from those for checking strength, and, thus, the combinations and load factors are different.

3.1.4 Required Strength

The AISC specification permits the use of elastic, inelastic, or plastic structural analysis. Generally, design is performed by elastic analysis to determine the required strength.

In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces, the required strength is explicitly stated in this Specification.

3.1.5 Limit States

A limit state is a condition in which a structural system or component becomes unfit for its intended purpose, when it is exceeded. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be related to structural behavior, such as the formation of a plastic hinge or mechanism; or they may represent the collapse of the whole or part of the structure, such as by instability or fracture. The design provisions provided make certain that the probability of reaching a limit state is acceptably small by stipulating the combination of load factors, nominal strengths consistent with the design assumptions.

Two kinds of limit states apply to structures: (1) strength limit states define safety against local or overall failure conditions during the intended life of the structure; and (2) serviceability limit states define functional requirements. The primary focus is on strength limit states because of overriding considerations of public safety. This does not mean that limit states of serviceability are not important to the designer, who must provide for functional performance. However, serviceability considerations permit more exercise of judgment on the part of the designer.

Strength limit states vary from element to element, and several limit states may apply to a given element. The following strength limit states are the most common: yielding, formation of a plastic hinge, member or overall frame instability, lateral-torsional buckling, local buckling,

rupture, and fatigue. The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

3.1.6 Design for Strength Using Load and Resistance Factor Design

Design for strength by LRFD is performed in accordance with the equation $R_u \leq \phi R_n$. The left side of the equation, R_u represents the required strength computed by structural analysis based on loads stipulated in SEI/ASCE 7, while the right side, ϕR_n, represents the limiting structural resistance, or *design strength*, provided by the member.

The resistance factor ϕ is equal to or less than 1.0. When compared to the nominal strength, R_n, a ϕ-value of less than 1.0 accounts for inaccuracies of the theory and variations in mechanical properties and dimensions of members and frames. For limit states where $\phi = 1.0$, the nominal strength is judged to be sufficiently conservative when compared to the actual strength that no further reduction is needed.

The LRFD provisions are based on (1) probabilistic models of loads and resistance, (2) a calibration of the LRFD provisions to the ASD Specification for selected members, and (3) the evaluation of the resulting provisions by judgment and past experience.

3.1.7 Serviceability Concerns

Serviceability as referred to performance of a building is a state in which the functions of the building, its appearance, maintainability, durability, and comfort of its occupant are evaluated under normal usage. While serviceability generally does not involve collapse of a building, loss of life or injury, it can seriously impair the building's usefulness and lead to costly repairs and other economic consequences. Serviceability provisions are essential to provide satisfactory performance of building structural systems. Neglect of serviceability may result in structures that are excessively flexible or otherwise perform unacceptably in service.

The three general types of structural behavior that are indicative of impaired serviceability in steel structures are as follows:

1. Excessive deflections that may affect the appearance, function, or drainage of the building or may cause damaging transfer of load to nonstructural components and attachments
2. Excessive vibrations produced by the activities of the building occupants, mechanical equipment, or wind effects, which may cause occupant discomfort or malfunction of building service equipment
3. Excessive local damage (local yielding, buckling, slip, or cracking) or deterioration (weathering, corrosion, and discoloration) during the service life of the structure

Serviceability limit states depend on the occupancy or function of the building, the perceptions of its occupants, and the type of structural system. Limiting values of structural behavior intended to provide adequate levels of serviceability are not easily quantifiable. It depends on how it is perceived by an individual and therefore should be determined by a team consisting of the building owner/developer, the architect, and the structural engineer after a careful analysis of all functional and economic requirements and constraints. In arriving at acceptable serviceability limits, the team should recognize that building occupants are able to perceive structural deformations, motions, cracking, or other signs of distress at levels that are much lower than those that would indicate impending structural damage or failure. Such signs of distress may be viewed as an indication that the building is unsafe and diminish its economic value, and therefore must be considered at the time of design.

Service loads that may require consideration in checking serviceability include the following: (1) static loads from the occupants, snow or rain on the roof, or temperature fluctuations; and (2) dynamic loads from human activities, wind effects, the operation of mechanical or building

service equipment, or traffic near the building. Service loads are loads that act on the structure at an arbitrary point of time, and may be only a fraction of the corresponding nominal load. The response of the structure to service loads is generally analyzed assuming elastic behavior. Members that accumulate residual deformations under service loads also may require examination with respect to this long-term behavior.

Camber in floor framing members is frequently specified in order to provide a level surface under *permanent loads*, for reasons of appearance or for alignment with other work. In normal circumstances camber does nothing to prevent excessive vibration. It is normally introduced in beams by controlled heating of selected portions of the beam or by cold bending, or both. The *Code of Standard Practice for Steel Buildings and Bridges* provides tolerances on actual camber and recommends that all cambers be measured in the fabricating shop on unstressed members.

3.1.8 Deflections

Excessive vertical deflections and misalignment arise primarily from three sources: (1) gravity loads, such as dead, live and snow loads; (2) effects of temperature creep and differential settlements; and (3) construction tolerances and errors. Such deformations may be visually objectionable; cause separation, cracking, or leakage of exterior cladding, doors, windows, and seals; and cause damage to interior components and finishes. Appropriate limiting values of deformations depend on the type of structure, detailing and intended use. Historically, common deflection limits for horizontal members have been 1/360 of the span for floors subjected to reduced live load and 1/240 of the span for roof members. Deflections of about 1/300 of the span (for cantilevers, 1/150 of the length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than 1/200 of the span may impair operation of moveable components such as doors, windows and sliding partitions.

Deflection computations for composite beams should include an allowance for creep and shrinkage.

In certain long-span floor systems, it may be necessary to place a limit (independent of span) on the maximum deflection to minimize the possibility of damage of adjacent nonstructural elements. For example, damage to nonload-bearing partitions may occur if vertical deflections exceed more than about 3/8 in. (10 mm) unless special provision is made for differential movement. However, many components can and do accept larger deformations.

The dead load used for calculating deflections is the dead load that occurs following attachment of nonstructural elements. For example, in composite construction, the dead load effects frequently are taken as those imposed after the concrete has cured. For ceiling related calculations, the dead load effects may include only those placed after the ceiling structure is in place.

Camber is a curvature of a beam in its strong direction while *Sweep* is a curvature in the weak direction. The term *camber* refers to the curvature of a beam as it is delivered, or to the curvature induced to compensate for deflection under applied loads. Hot-rolled shapes are air cooled without physical restraint. When a hot-rolled shape cools, it bends in both the strong and weak directions, twists, and distorts locally. ASTM A6 limits the amount of sweep and camber acceptable in a rolled shape, and steel producers straighten the product to meet those tolerances. Straightening is done with a rotary straightener or in a press. The delivered product usually has some camber and sweep. When no camber is specified in rolled shapes fabricated into beams, the shape is fitted in fabrication so that the natural camber is up in the beam as erected. In long-span floors typical of commercial buildings, camber is a design parameter and is intentionally induced in the beam. Camber is typically designed as a percentage of the dead-load deflection or occasionally a percentage of the dead- and live-load deflection of the beam. Natural mill camber is acceptable if the design camber is ¾ in. or less. The decision to camber and how much to camber is made by the designer.

When camber is required, the fabrication shop cambers the shape after it is cut and punched or drilled, and before detail material is attached. Camber can be achieved with local application of heat, and this is typically done for heavy sections. For most beams, however, camber is induced by

cold bending with hydraulic jacks in a cambering machine. Cold camber is achieved by inducing plastic tensile and compressive strain of elements of the shape. The curvature should be limited so that the induced strain is limited to a reasonable percentage, based on the minimum elongation requirements of the material specification. Also the strain should be induced uniformly over a length of the section using multiple jacks.

In girders, webs may be cut thermally with a curve calculated to achieve the required camber when the flanges have been welded to the web. Variations do occur, and camber adjustments are made using local application of heat after the piece is assembled. Large trusses are cambered by detailing and fabricating the elements to calculated lengths such that the desired camber is achieved when the trusses are assembled. In other words, each member is fabricated to its geometric length in the cambered position.

Heat is used to induce camber or sweep in some cases and to adjust camber or straighten pieces in other cases. There are a variety of specific techniques used to heat-camber beams, but in all of them the side to be shortened is heated with a torch. As the part is heated, it tries to elongate. Because it is restrained by unheated material, the heated part with reduced yield stress is forced to upset (increase inelastically in thickness) to relieve its compressive stress. Since the increase in thickness is inelastic, the part will not return to its original thickness upon cooling. When the part is allowed to cool, therefore, it must shorten to return to its original volume. The heated flange thus experiences a new shortening that produces the camber. Heat cambering is generally slow and expensive and is typically used in sections larger than the capacity of available equipment. Heat can also be used to straighten or eliminate warping of parts. Some of these procedures are quite complex and intuitive, demanding experience on the part of the operator.

3.2 DESIGN OF MEMBERS SUBJECT TO COMPRESSION

3.2.1 Buckling of Columns, Fundamentals

The first problems of elastic instability, concerning lateral buckling of compressed members, were solved about 280 years ago by Leonhard Euler. At that time, the principal structural materials were wood and stone. The relatively low strength of these materials necessitated stout structural members for which the question of elastic stability is not of primary importance. Thus, Euler's theoretical solution, developed for slender bars, remained for a long time without practical application. Only with the beginning of extensive construction of steel during the latter half of the past century did the question of buckling of compression members become of practical importance. The use of steel led naturally to types of structures embodying slender compression members. Experience showed that such structures may fail in some cases not on account of high stresses, surpassing the strength of material, but owing to insufficient elastic stability of slender members.

Under pressure of practical requirements, the problem of lateral buckling of columns, originated by Euler, has been extensively investigated theoretically and experimentally and the limits within which the theoretical formulas can be applied have been established. However, lateral buckling of compressed members is only a particular case of elastic instability. In the modern design of buildings, we encounter not only solid struts, but built-up or "lattice-work" columns, and tubular members, where there is the possibility of local buckling, as well as buckling as a whole. In the use of thin sheet material, as in plate girders, we have to keep in mind that thin plates may prove unstable under the action of forces in their own planes, and fail by buckling sideways.

In discussing the design of compression members, it has not been deemed necessary to include an account of the general theory of elastic stability, which finds its appropriate place in books on the theory of elasticity. This section proceeds directly to particular problems showing in each case under what conditions the question of stability calls for consideration. While all available information relevant to a prescribed problem has been given, no attempt has been made to go beyond this into other considerations besides rational theory.

The preliminary knowledge of mathematics and strength of materials taken for granted is that usually covered by our schools of engineering.

3.2.1.1 Euler's Formula

The critical load on a compressed column may be obtained by assuming initially that the column is perfectly straight and compressed by a centrally applied load. Let us consider the case of a slender column, built-in at the bottom and subjected to load P at the top (Figure 3.1a).

If the load P is less than its critical value, the column remains straight and undergoes only axial compression. This straight form of elastic equilibrium is *stable*, that is, if a lateral force is applied and a small deflection produced, this deflection disappears when the lateral force is removed and the column becomes straight again. By gradually increasing P, a condition is obtained in which the straight form of equilibrium becomes unstable and a slight lateral force may produce a lateral deflection that does not disappear when the lateral force is removed. The *critical load* is then defined as the axial load, which is sufficient to keep the bar in a slightly bent form.

This load can be calculated by using the differential equation of the deflection curve. Taking the coordinate axes as indicated in Figure 3.1b, the bending moment at any cross section mn is $P(\delta - y)$ and the differential equation of the deflection curve becomes

$$EI\frac{d^2y}{dx^2} = P(\delta - y) \tag{a}$$

It is clear that, with the upper end free, buckling of the bar will occur in the plane of *smallest flexural rigidity*. This smallest rigidity EI is used in the above Equation (a). Using the notation

$$k^2 = \frac{P}{EI} \tag{b}$$

Equation (a) becomes

$$\frac{d^2y}{dx^2} + k^2y = k^2\delta \tag{c}$$

The general solution of this equation is

$$y = \delta + A\cos kx + B\sin kx \tag{d}$$

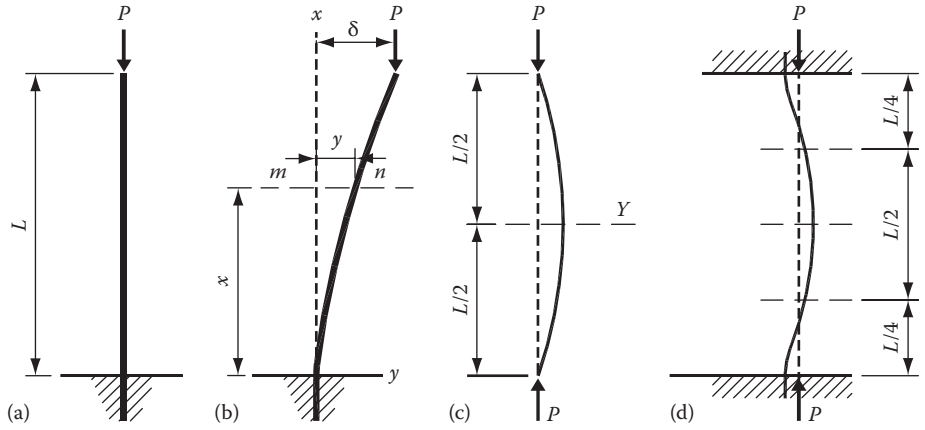


FIGURE 3.1 Buckling of columns: (a) cantilever column; (b) bending of cantilever column; (c) column with hinged ends; (d) column with built-in (or fixed) ends.

in which A and B are constants of integration that must be adjusted to satisfy the conditions at the built-in end:

In our case,

$$(y) = 0, \quad \text{at } x = 0$$

$$\left(\frac{dy}{dx}\right) = 0, \quad \text{at } x = 0$$

These conditions are fulfilled if

$$A = -\delta \quad \text{and} \quad B = 0$$

Then

$$y = \delta(1 - \cos kx) \tag{e}$$

The condition at the upper end requires

$$(y)_{x=1} = \delta$$

This will be satisfied if $\delta \cos kl = 0$, which requires either $\delta = 0$, in which case there is no deflection, or $\cos kl = 0$, that is,

$$kl = (2n + 1)\frac{\pi}{2} \tag{f}$$

where n is any integer. In this case δ is apparently indeterminate.

The smallest value of kl and therefore of P [see Equation (b)], which satisfies Equation (f), is obtained by taking $n = 0$. Then, by using Equation (b),

$$kl = l\sqrt{\frac{P}{EI}} = \frac{\pi}{2} \tag{g}$$

From which

$$P_{cr} = \frac{\pi^2 EI}{4l^2} \tag{h}$$

This is the *critical load* for the cantilever column. It is the smallest axial load that can keep the column in a slightly bent shape.

The critical loads for other cases can be obtained from the solution of the foregoing case. For example, in the case of a column with hinged ends, Figure 3.1c, it is evident from symmetry that each half of the column is in the same condition as the entire cantilever column shown in Figure 3.1b. Hence, the critical load for this case is obtained by using $l/2$, instead of l, in Equation (h).

Then,

$$P_{cr} = \frac{\pi^2 EI}{4(l/2)^2}$$

$$P_{cr} = \frac{\pi^2 EI}{l^2}$$

This case is very often encountered in the design of compression members and is called the *fundamental* case of buckling of a column. The critical load is called *Euler's load*, typically denoted by P_{cr}.

In the case of a column with both ends built-in (Figure 3.1d), there are reactive moments that keep the ends from rotating during buckling. The combination of the axial compressive forces with these moments is equivalent to the compressive forces P applied eccentrically. Inflection points occur where the line of action of P intersects the deflection curve, because the bending moments at these points are zero. These points at the mid-point of the column length divide the column into four equal portions, each of which is in the same condition as the column represented in Figure 3.1b. Hence, the critical load for a column with built-in ends is found from Equation (h) by using $l/4$ instead of l, which gives

$$P_{cr} = \frac{\pi^2 EI}{4(l/4)^2}$$

$$= \frac{4\pi^2 EI}{l^2}$$

3.2.1.2 Energy Method of Calculating Critical Loads

Let us consider the case of lateral buckling of a compressed column shown in Figure 3.2. Assume that a lateral deflection is produced in the column shown therein. When load P is applied to the column, the strain energy of deformation increases, because in addition to the energy of compression, some energy due to bending of the bar is added to the system. At the same time, the potential energy of the load diminishes, owing to the lowering of its point of application. This decrease in potential energy is simply the work done by the load during the lowering of the top of the column. If ΔV denotes the strain energy of bending of the column and ΔT the work done by the load during bending, then, applying the law of conservation of energy we get

$$\Delta V = \Delta T \tag{i}$$

For the case represented in Figure 3.2, the deflection curve of the slightly buckled column is given by the equation

$$y = \delta\left(1 - \cos\frac{nx}{2l}\right)$$

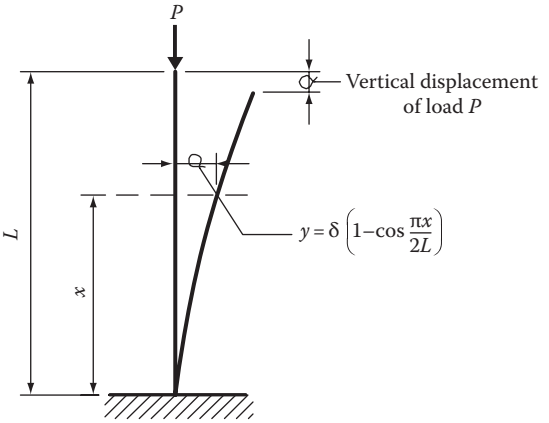


FIGURE 3.2 Buckling of cantilever column; energy method.

The bending moment at any cross section is

$$M = P(\delta - y) = P\delta \cos \frac{\pi x}{2l}$$

and the corresponding bending energy is

$$\Delta V = \int_0^l \frac{M^2 dx}{2EI} = \frac{P^2 \delta^2 l}{4EI}$$

The vertical movement of the load P during buckling is equal to

$$\lambda = \frac{1}{2}\int_0^l \left(\frac{dy}{dx}\right)^2 dx = \frac{\delta^2 \pi^2}{16l}$$

And the corresponding work done by P is

$$\Delta T = \frac{P\delta^2 \pi^2}{16l}$$

Substituting the derived expressions for ΔV and ΔT in Equation (i), we obtain for the critical load the same value as given by Equation (h).

3.2.2 Behavior of Compression Members

The primary difference between tension and compression action is steel members does not arise from differences in stress–strain relationships. The stress–strain curves of structural steels in compression are almost identical in the elastic and ductile regions to those for tension. The difference arises due to the proportions of the members and the phenomenon of buckling, which often takes place at stresses far below the yield stress. As shown in the previous section, the failure load for long, slender columns is governed by the parameter, EI/L_{eff}^2, where E is a material property, the elastic modulus, I is a geometric property of the cross section; L_{eff} is the length of the column modified by a factor dependent on the nature of end restraints. The yield strength appears nowhere in the equation for the critical load. It only becomes relavent when, for relatively short, stiff columns, the Euler equation leads to critical stress higher than yield stress.

A second difference arises from the effect of bending moment when associated with axial load. An axial tension tends to reduce member deflections caused by bending moment, thus tending to stabilize the member, while axial compression associated with end moment is destabilizing. Bending moment may arise in axially loaded members from a number of practical considerations. These moments result in deflection of the member along its length. Such moments are destabilizing and reduce the available resistance to compressive stress.

The most easily recognizable compression member is a *column* in a building frame, which resists the gravity effect of dead and live loads on the various floors. However, compression members are not limited to columns and may be oriented in any direction. They may resist loads that result from gravity or other forces acting in any direction.

In considering the failure of compression members, it is possible to define three different classes of columns:

1. Short columns
2. Intermediate columns
3. Long columns (long here means $KL/r > C_c$, not great length)

Short columns are defined by very low values of the ratio KL/r (see Figure 3.3). In this range, the Euler curve of critical load is approaching infinity. However, when the load becomes sufficient to cause yield stress, *failure* occurs by compression yielding. Actually, collapse is not likely. As the column shortens in the yield range, the elements of its cross section squash and thicken, increasing its resistance. Short columns are not common in steel structures, but short special-purpose compression struts are.

Failure of intermediate length *columns* is affected by the tendency for buckling instability, the limit set by yield stress, and unmeasured effects such as eccentricity of end connections and lack of straightness. The curves in Figure 3.3 show a smooth, gradual transition between the Euler curve and the yield condition. The two curves become tangent at a value KL/r, *dependent on* F_y, that is called C_c. The value is somewhat arbitrarily chosen in the AISC Specification as

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

when $KL > C_c$ the Euler critical stress becomes the failure stress.

As shown in Figure 3.3, C_c varies from 76 to 126 for most common structural steels. The intermediate column range is defined by

$$20 < \frac{KL}{r} < C_c$$

Although we show building columns as being perfectly straight on structural drawings, in practice, they are rarely so. Observe that usual mill practice permits a *column-sweep* of as much as an inch for long columns. This is essentially not controllable by the designer. Moments caused

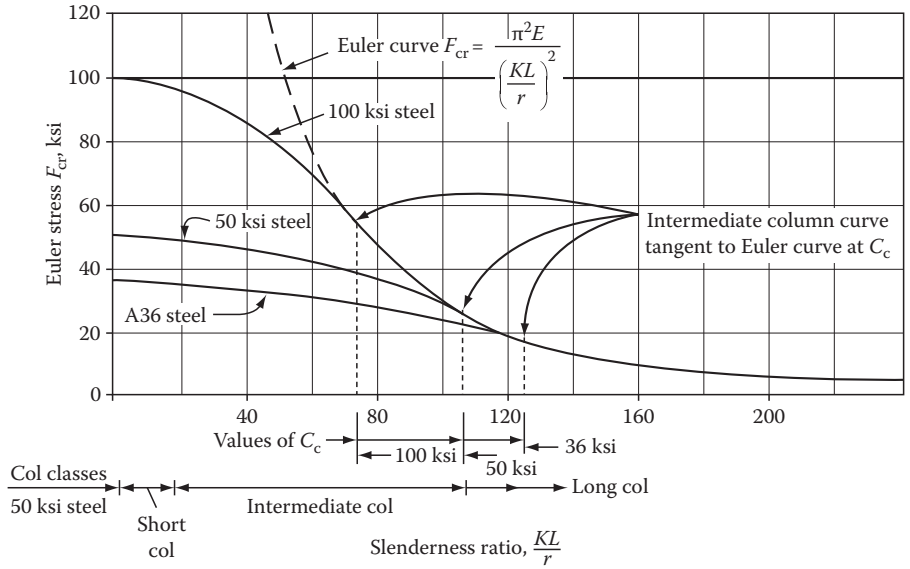


FIGURE 3.3 Euler stress P_{cr}/A_g versus KL/r.

by fabrication and erection error are somewhat controllable by the designer through strict specifications and inspection. But even then, perfection is costly and not usually achieved. The AISC specifications identify these and other common imperfections that increase effective compressive stresses, thus reducing the portion of F_y available for axial compression. These aspects along with the so-called PΔ effects are addressed later in this chapter.

It is seen from the previous discussions that the vertical load that causes buckling of columns does not depend upon the strength of the material but only the dimensions of the column and the modulus of elasticity, E, of the material. For example, two equal slender columns, one of high strength, ASTM A913, 70 ksi steel, and the other ASTM A53 Grade B, 35 ksi steel, will buckle at the same compressive force. This is because of the moduli of elasticity are the same for both steels although the strengths of the columns working as short columns are very different.

3.2.2.1 Element Instability

Figure 3.4a shows an I-shaped compression member where a portion of one flange has buckled locally while the rest of the member remains straight. The reduction in compressive capacity should be obvious as the buckled portion rejects load, transferring it to other elements.

Similar buckling potential exists in other elements of cross sections shown in Figure 3.4b through e, governed by the ratios of b/t or, in the case of round tubes, r/t.

Much of the attention of designers and design rules is directed to making individual elements of compression members stiff enough to eliminate local buckling as a control on failure load. If that is not done, the curves of Figure 3.3, no longer describe the failure loads. Failure will be initiated at lower stress, being triggered by the instability of the vulnerable element. The controlling parameter is the ratio b/t, or its equivalent, r/t, in the case of thin-walled round tubes and pipes.

3.2.3 Limits on Slenderness Ratio, *KL/r*

Engineers of bygone years were mandated to limit the slenderness ratio, KL/r, of compression members to 200 for primary members, and to 300 for secondary members. There never was a clear-cut distinction between what constituted a primary or secondary member; the classification was entirely up to the design engineers. However, the then prevailing opinion was to consider columns as primary, and other building members, such as truss diagonals and bracing members, as secondary members.

With the publication of the 13th edition of the ASCE Manual, all of that has changed. There is no restriction on the slenderness ratio of compression members.

In practice, however, it is a good idea to limit KL/r of columns to 200, not so much for analytical reasons but for other practical considerations such as construction economy, ease of handling, minimize inadvertent damage during fabrication, transportation and erection.

3.2.4 Column Curves: Compressive Strength of Members without Slender Elements

To understand the performance of compression members, consider the curves in Figure 3.5 that show the failure stress versus the slenderness ratio KL/r for three grades of steel. Three things are clear from the figure: the yield strength of steel is very significant for short columns, of decreasing significance through the intermediate range, and of no consequence in the performance of long columns. The most efficient use of the strength of steel is made by selecting columns in the intermediate range. To achieve large values of I and r for a given area, a section that has the area distributed as far from its centroid as possible offers the best choice, other things being equal. The most efficient sections are those with $r_u/r_y = 1$. Of the available wide-flange sections, those with $bld = 1$ are most efficient for columns.

Compression members are divided into three classes by their values of $\frac{KL}{r}$. See Figure 3.3. Short columns are defined by very low values of $\frac{KL}{r}$. In this range, the Euler curve for critical load is

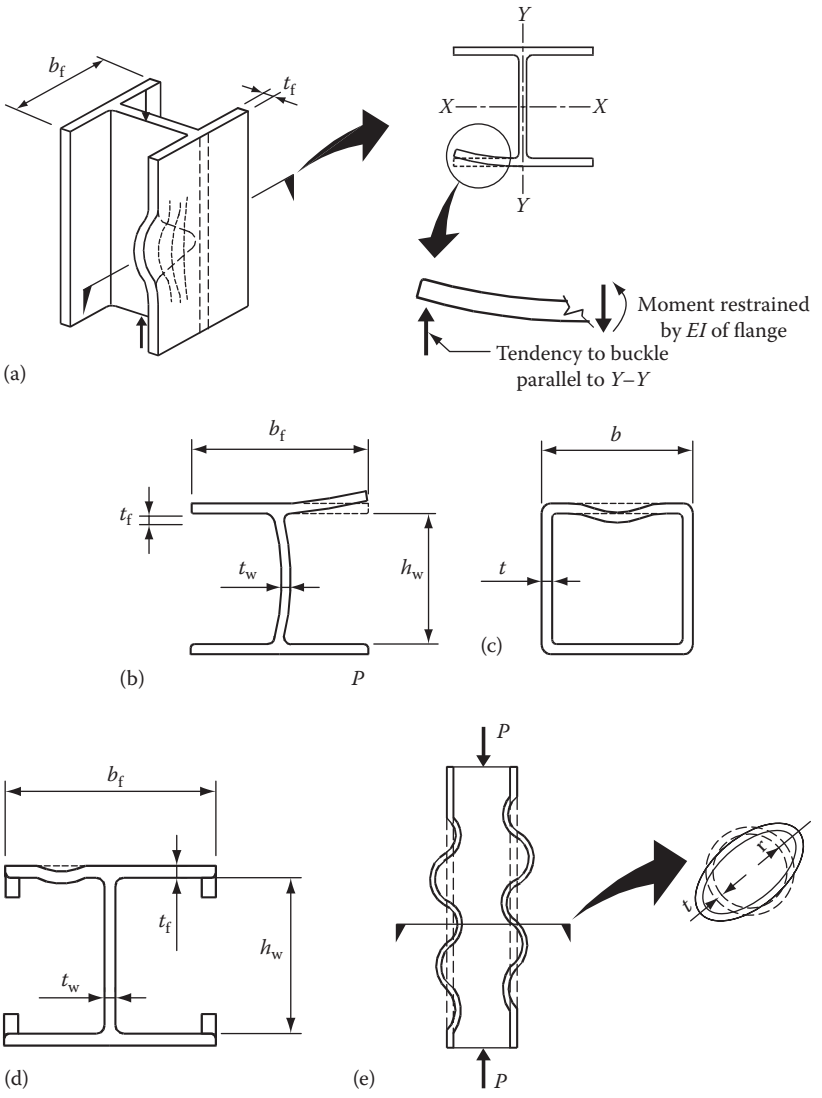


FIGURE 3.4 Local buckling of steel sections. Notes: Steel section subjected to compression from direct forces or flexure may be classified as compact, noncompact, or slender–element sections. For a section (such as an I or a box section) to qualify as compact, the flanges must be continuously connected to the web or webs and the width-thickness ratios of compression elements must not exceed a certain limiting width–thickness ratio, γ_p. If the width–thickness ratio of any compression element exceeds γ_p, but does not exceed the limit γ_r, the section is termed noncompact. Further, if the width-thickness ratio of any elements exceed γ_r, the section is referred to as slender–element compression section. Values of λ_p and λ_r for numerous cases are given in Table B4.1 and AISC specifications 2005.

approaching infinity. However, when the axial load becomes sufficient to cause yield stress, failure occurs by compression yielding although collapse is unlikely.

Failure for intermediate length columns is initiated by the tendency for buckling instability. The failure curve shows a smooth transition between the yield and the buckling conditions. The two curves become tangent at a value of $\frac{KL}{r} = 4.71\sqrt{\frac{E}{F_y}}$ somewhat arbitrarily chosen in the AISC specifications.

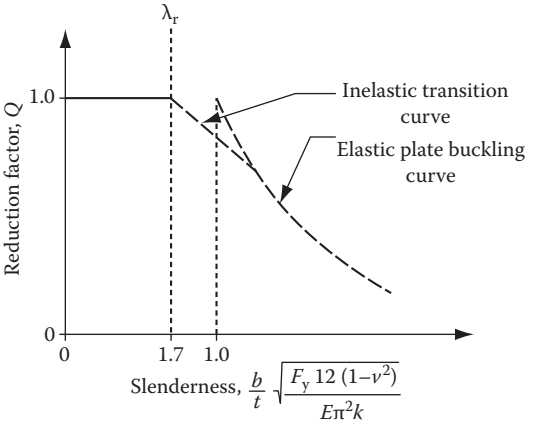


FIGURE 3.5 Stress reduction factor Q versus slenderness.

The nominal compressive strength, P_n, is determined based on the limit state of flexural buckling.

$$P_n = F_{cr}A_g$$

The flexural buckling stress, F_{cr}, is determined as follows:

(a) When $\dfrac{KL}{r} < 4.71\sqrt{\dfrac{E}{F_y}}$ (or $F_e > 0.44F_y$)

$$F_{cr} = \left[0.658\frac{F_y}{F_e}\right]F_y$$

(b) When $\dfrac{KL}{4} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $F_e < 0.44F_y$)

$$F_{cr} = 0.877F_e$$

where F_e is the elastic critical buckling stress given by

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2}$$

It should be noted that the underlying ASCE formulas for F_{cr} are based on the assumption that individual elements of sections will not buckle at lower stress than that which causes general failure. If the individual elements are prone to buckling as determined by the ratio b/t, then a stress reduction factor Q (for slender elements) or Q_a (for slender stiffened elements) must be taken into account.

3.2.5 Columns with Slender Unstiffened Elements: Yield Stress Reduction Factor, Q

In their every day practice, structural engineers designing columns with hot-rolled shapes will seldom find occasion to turn to Section E 7.1 of AISC 360-05 Specification. This section talks about a slenderness reduction factor, Q, which in essence reduces the yield stress, F_y, of the column, and is

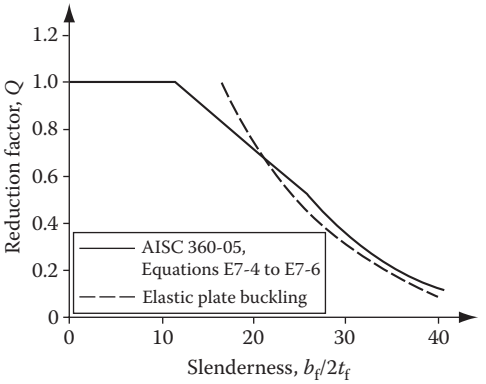


FIGURE 3.6 Stress reduction factor Q versus $b_f/2t_f$ for rolled shapes.

only applicable typically to angles with thin legs and tee-shaped columns having slender stems. The reduction factor, Q as a function of slenderness ratio is given by the relation:

$$Q = \frac{b}{t}\sqrt{\frac{F_y 12(1-\nu^2)}{E\pi^2 k}}$$

This is shown in Figure 3.5. In this equation, $\nu = 0.3$ (Poisson's ratio), and k is the plate buckling coefficient. $k = 0.425$ for single angles, and for T-sections, $k = 0.7$.

The curve shown in Figure 3.5 relating Q to the plate slenderness ratio has three components:

1. The top part where $Q = 1$ (i.e., no reduction in the yield stress) when slenderness factor is less than or equal to 0.7
2. The elastic buckling portion
3. A transition range between the $Q = 1$ range, and the elastic buckling portion

It should be noted that the stress reduction factor also applies to rolled shapes with slender flanges. See Figure 3.6.

In conclusion, the design of a gravity column primarily carrying vertical loads reverts to selecting a steel section with a calculated design compressive strength $\phi_c P_n$ greater than or equal to the ultimate load P_u. For buildings taller than 40 stories or so, because of high axial loads it becomes necessary to use built-up columns or cover-plated wide flange columns. Examples are shown in Figure 3.7.

3.2.6 Design Examples: Compression Members

Steel members in buildings subject to compressive loads typically include columns, truss top chords, struts, and diagonal braces. Slenderness is a major factor in design of compression members. Most suitable steel shapes are pipes, tubes, or wide-flange sections. Double angles, however, are commonly used for diagonal braces of truss members because they can be easily connected to truss chords with gusset plates and bolts or welds.

The AISC Specification gives the nominal strength P_n (kip) for a steel compact or noncompact section without slender elements in compression as

$$P_n = A_g F_{cr}$$

The factored load for LRFD, $P_u = \phi P_n$, $\phi = 0.90$.

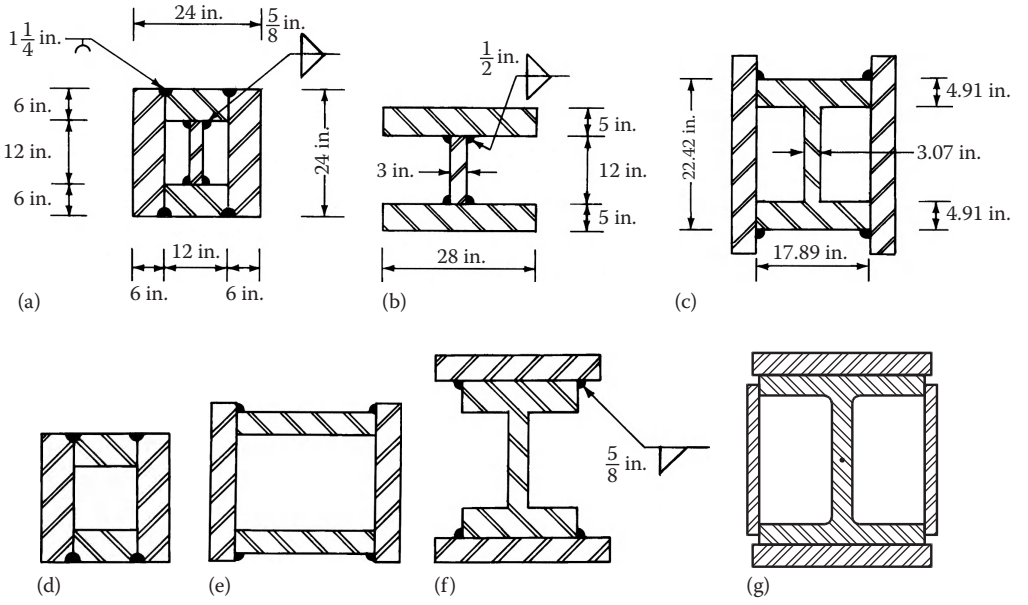


FIGURE 3.7 Examples of built-up columns.

The flexural buckling stress, F_{cr}, is a function of the material strength and slenderness. First the elastic critical buckling stress F_e (ksi) is determined as follows:

$$F_e = \frac{\pi^2 E}{(KL/r)^2}$$

When $(KL/r) < 4.71\sqrt{E/F_y}$ or $(F_e \geq 0.44F_y)$

$$F_{cr} = (0.658^{F_y/F_e})F_y$$

When $(KL/r) < 4.71\sqrt{E/F_y}$ or $(F_e < 0.44F_y)$

$$F_{cr} = 0.877F_e$$

where

A_g is the gross area of the member, in.^{2}
K is the effective length factor
L is the unbraced length of the member, in.
F_y is the yield stress of steel, ksi
E is the modulus of elasticity of steel, ksi
r is the radius of gyration corresponding to the plane of buckling, in.

3.2.6.1 Wide Flange Column, Design Example

A wide-flange section is to be used for columns in a 10-story steel building. A typical interior column in the lowest story will be designed to support gravity loads. (In this example, no eccentricity will be assumed for the load.) The effective height of the column is 18 ft. The axial loads on the column from the column above and from the steel girders supporting the second level are dead load of 840 kip and live load of (reduced according to the applicable building code) 240 kip.

The factored axial load is the larger of the following:

$$P_u = 840 \times 1.4 = 1176 \text{ kip}$$

$$P_u = 840 \times 1.2 + 240 \times 1.6 = 1392 \text{ kip (governs)}$$

The use of the column design tables given on pages 4–13 and 4–16 of the AISC "Steel Construction Manual" 13th edition presents the following options:

1. W14 × 145 with a design strength $\Phi P_n = 1550$ kip@$KL = 18$ ft.
2. W12 × 152 with a design strength $\Phi P_n = 1440$ kip@$KL = 18$ ft.

Both columns are ASTM $F_y = 50$ ksi.

Parts of Tables 4-13 and 4-16 relavent to the design example are reproduced here as Figures 3.8a and 3.8b.

3.2.6.2 HSS Column, Design Example

A square HSS section of $F_y = 42$ ksi steel is to be used to support framing for the flat roof of a one-story factory building. The roof height is 18 ft. from the top of the steel roof beams to the finish of the floor. The steel roof beams are 16 in. deep, and the bases of the steel-pipe columns are 1.5 ft. below the finished floor. A separation square joint is provided in the slab at the steel column. Therefore, the concrete slab does not provide lateral bracing. The effective height of the column, from the base to the center line of the steel roof beam is

$$h = 18 + 1.5 - \frac{16}{2 \times 12} = 18.83 \text{ ft}$$

The dead load on the column is 60 kip. The live load due to snow at the roof is 65 kip. The factored axial load is the larger of

$$P_u = 60 \times 1.4 = 84 \text{ kip}$$

$$P_u = 60 \times 1.2 + 65 \times 1.6 = 176 \text{ kip (governs)}$$

With the factored load known, the required HSS may be obtained from Axial Compression Tables given in the AISC "Manual of Steel Construction." For $KL = 19$ ft., a 7 × 7 HSS (weight 27.5 lb per linear ft) offers a compression-load capacity of 196 kip—See Table 4-4 of the AISC Steel Construction Manual (Figure 3.9 of this text).

For verification of this selection, the following computations for the column capacity are made based on a radius gyration $r = 2.72$ in.

Because

$$\left(\frac{KL}{r} = \frac{19 \times 12}{2.72} = 83.82\right) \le \left(4.71\sqrt{\frac{E}{F_y}} = \sqrt{\frac{29{,}000}{42}} = 123.75\right)$$

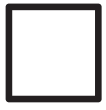
F_y = 50 ksi		Available Strength in Axial Compression, kip W Shapes								W14			
Shape		W14×											
wt/ft		145		132		120		109		99		90	
		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	1,280	1,920	1,160	1,740	1,060	1,590	959	1,440	872	1,310	792	1,190
	6	1,250	1,870	1,130	1,700	1,030	1,550	934	1,400	849	1,280	771	1,160
	7	1,240	1,860	1,120	1,680	1,020	1,530	924	1,390	840	1,260	763	1,150
	8	1,220	1,840	1,110	1,660	1,010	1,510	914	1,370	831	1,250	754	1,130
	9	1,210	1,820	1,090	1,640	995	1,500	902	1,360	820	1,230	746	1,120
	10	1,200	1,800	1,080	1,620	981	1,470	889	1,340	808	1,210	734	1,100
	11	1,180	1,770	1,060	1,590	965	1,450	875	1,320	795	1,200	722	1,090
	12	1,160	1,740	1,040	1,570	949	1,430	860	1,290	781	1170	709	1070
	13	1,140	1,720	1,020	1,540	931	1,400	844	1,270	767	1,150	696	1,050
	14	1,120	1,690	1,000	1,510	912	1,370	827	1,240	751	1,130	682	1,020
	15	1,100	1,650	982	1,480	893	1,340	809	1,220	734	1,100	667	1,000
	16	1,080	1,620	959	1,440	872	1,310	790	1,190	717	1,080	651	978
	17	1,050	1,580	936	1,410	851	1,280	771	1,160	699	1,050	635	954
	18	1,030	1,550	912	1,370	829	1,250	751	1,130	681	1,020	618	928
	19	1,000	1,510	887	1,330	806	1,210	730	1,100	662	995	600	902
	20	979	1,470	862	1,300	788	1,180	709	1,070	642	966	583	876
	22	926	1,390	809	1,220	736	1,100	665	1,000	602	906	546	821
	24	871	1,310	756	1,140	685	1,030	620	932	562	844	509	765
	26	815	1,230	702	1,050	636	956	575	864	520	782	470	708
	28	759	1,140	647	973	586	881	530	797	479	720	434	652
	30	702	1,060	594	892	537	807	485	730	438	659	397	596

FIGURE 3.8 (a) Axial compression strength of selected W14 columns.

(*continued*)

W12		Available Strength in Axial Compression, kip W Shapes								F_y = 50 ksi			
Shape		W12×											
Wt/ft		190		170		152		136		120		106	
		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	1,670	2,510	1,500	2,250	1,340	2,010	1,200	1,800	1,060	1,590	933	1,400
	6	1,610	2,420	1,440	2,170	1,290	1,940	1,150	1,730	1,020	1,530	897	1,350
	7	1,590	2,390	1,420	2,140	1,270	1,910	1,140	1,710	1,000	1,510	884	1,330
	8	1,570	2,360	1,400	2,110	1,250	1,880	1,120	1,680	986	1,480	870	1,310
	9	1,540	2,320	1,380	2,070	1,230	1,850	1,100	1,650	968	1,460	854	1,280
	10	1,510	2,270	1,350	2,030	1,210	1,820	1,080	1,620	949	1,430	837	1,260
	11	1,480	2,230	1,320	1,990	1,180	1,780	1,050	1,580	927	1,390	818	1,230
	12	1,450	2,180	1,290	1,940	1,150	1,730	1,030	1,540	905	1,360	798	1,200
	13	1,410	2,120	1,260	1,900	1,120	1,690	1,000	1,500	881	1,320	776	1,170
	14	1,380	2,070	1,230	1,840	1,090	1,640	972	1,460	856	1,290	754	1,130
	15	1,340	2,010	1,190	1,790	1,060	1,600	943	1,420	829	1,250	730	1,100
	16	1,300	1,950	1,150	1,730	1,030	1,540	913	1,370	802	1,210	706	1,060
	17	1,250	1,880	1,120	1,680	996	1,490	881	1,320	774	1,160	681	1,020
	18	1,210	1,820	1,080	1,620	958	1,440	849	1,280	746	1,120	656	985
	19	1,170	1,750	1,040	1,560	922	1,390	817	1,230	717	1,080	630	946
	20	1,120	1,690	997	1,500	886	1,330	784	1,180	687	1,030	604	907
	22	1,030	1,550	916	1,380	812	1,220	718	1,080	628	944	551	828
	24	941	1,420	834	1,250	738	1,110	661	979	569	855	498	749
	26	852	1,280	754	1,130	666	1,000	586	881	511	768	447	672
	28	765	1,150	675	1,010	595	895	523	786	455	684	397	597
	30	682	1,020	600	902	528	793	462	695	401	602	350	525

FIGURE 3.8 (continued) (b) Axial compression strength of selected W12 columns.



HSS7–HSS6

Available Strength in Axial Compression, kip Square HSS									F_y = 46 ksi			
		HSS7X7X							HSS6X6X			
Shape		5/16		1/4		3/16[a]		1/8[c]		5/8		1/2
t_{design}, in.		0.291		0.233		0.174		0.116		0.581		0.465
wt/ft		27.5		22.4		17.1		11.6		42.1		35.1
		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_y	0	209	314	170	255	124	187	61.6	92.6	323	485	268	403
	6	199	300	162	244	120	181	60.4	90.8	300	451	250	376
	7	196	295	160	240	119	179	60.0	90.1	292	439	244	366
	8	192	289	156	235	118	177	59.5	89.4	283	426	237	356
	9	188	283	153	230	116	174	58.9	88.5	273	411	229	344
	10	183	276	149	225	114	171	58.3	87.6	263	395	221	332
	11	178	268	145	219	111	166	57.6	86.5	252	378	212	318
	12	173	260	141	212	107	161	56.8	85.3	240	361	202	304
	13	168	252	137	206	104	156	55.9	84.0	228	343	193	290
	14	162	243	132	199	101	151	55.0	82.6	216	324	18.6	275
	15	156	234	127	191	97.0	146	54.0	81.1	203	306	173	260
	16	150	225	122	184	93.3	140	52.9	79.5	191	287	163	244
	17	143	215	117	176	89.5	134	517	77.8	178	268	162	229
	18	137	206	112	168	85.6	129	50.5	75.9	166	249	142	214
	19	130	196	107	161	81.7	123	49.2	73.9	154	231	132	199
	20	124	186	102	153	77.8	117	47.8	71.8	142	213	123	184

FIGURE 3.9 Axial compression strength of selected HSS 7 × 7 and HSS 6 × 6 columns.

The elastic critical stress F_e from Equation E 3-4 of ANSI/AISC 360-05 Specification is

$$F_e = \frac{\pi^2 \times 29{,}000}{(KL/r)^2} = \frac{\pi^2 \times 29{,}000}{83.82^2} = 40.74 \text{ ksi}$$

Equation (E3.2) yields the critical stress

$$F_{cr} = (0.658^{42/40.74}) \times 42 = 27.28 \text{ ksi}$$

The nominal strength P_n of the HSS 7 × 7 is given by

$$P_n = F_{cr}A_g = 27.28 \times 7.59 = 207 \text{ kip}$$

The design strength = $\phi_c P_n = 0.9 \times 207 = 186 \text{ kip} > 17.6 \text{ kip}$.

3.3 DESIGN OF MEMBERS SUBJECT TO BENDING

It should be noted that compression portions of beams subjected to bending may have buckling problems similar to compression members. Referring to Figures 3.10 through 3.12, we note that the segment of the beam above the neutral axis in the positive member region, acts in a manner similar to a column. Unless there are closely spaced lateral restraints, we can expect that there is some critical moment, M_{cr}, analogous to the critical load, P_{er}, at which compression members buckle laterally.

The characteristic mode of buckling for an I-shaped section consists in part due to St. Venant's torsion characterized by the polar moment of inertia, J, and warping of the beam flanges as delineated by the warping moment of inertia C_w. The polar moment of inertia also called St. Venant torsion constant, J, is typically expressed in units of in.^{4}, while C_w, in units of in.^{6}

Shapes that are intended to be used primarily as beams are generally proportioned so that the moment of inertia about the major principal axis is considerably larger than that about the minor principal axis. This is done to produce shapes that make economical beams. As a result, they are relatively weak in resistance to torsion and to bending about the minor axis, and if not held in-line by floor

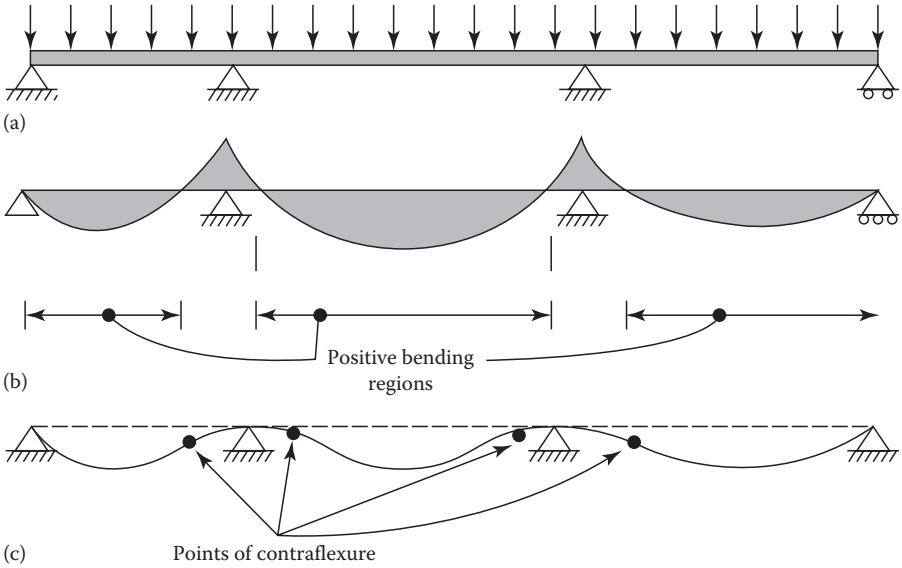


FIGURE 3.10 Lateral torsional buckling of beams: (a) continuous beam with a uniformly distributed load; (b) bending moment diagram showing positive bending regions; (c) point of contraflexure.

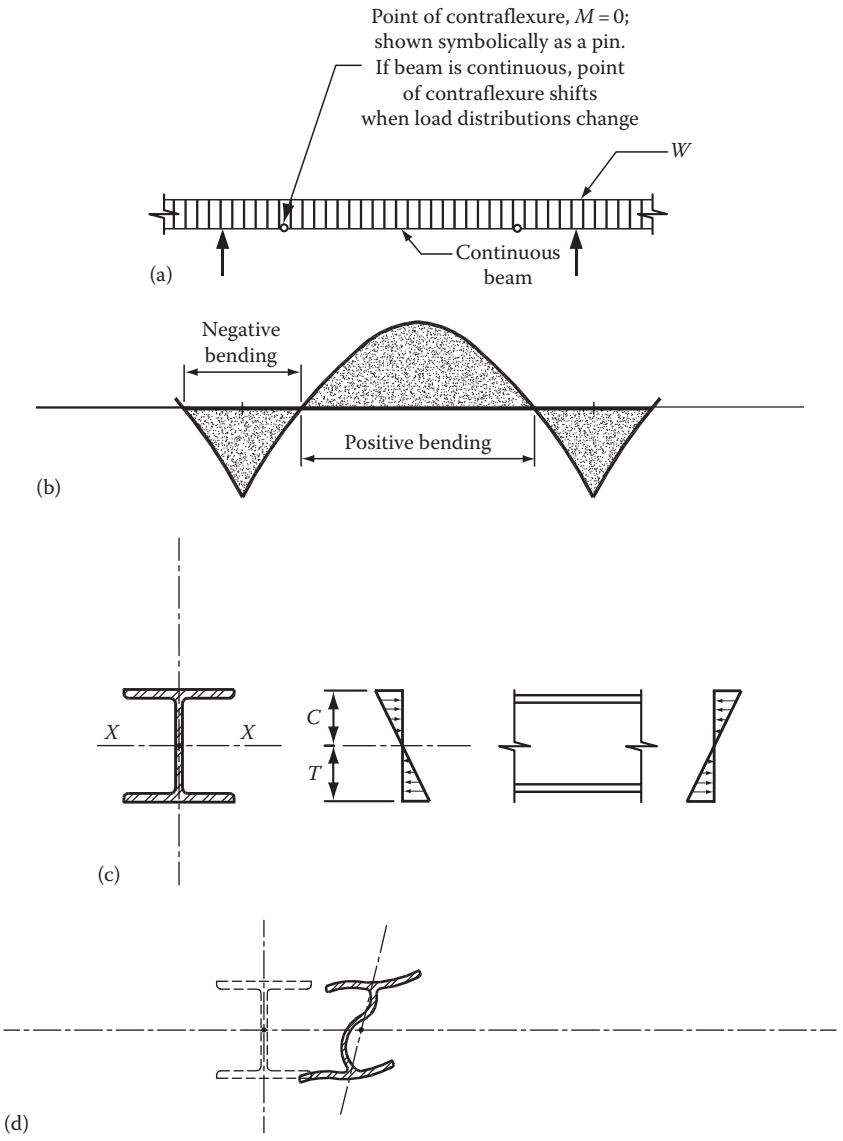


FIGURE 3.11 Lateral buckling of beams: (a) continuous beam, uniformly distributed load; (b) bending moment diagram; (c) portion of positive bending region; (d) lateral-torsional buckling of I beam.

construction or by bracing, they may become unstable under load. The instability manifests itself as a sidewise bending accompanied by twist and is called *lateral buckling or later-torsional buckling.*

It is not always easy to decide whether a beam has adequate support against lateral buckling. Embedment of the top flange in a concrete slab provides support except when the beam is a cantilever in which the compression flange is at the bottom. A completely encased beam is laterally braced no matter which flange is in compression. Wood flooring spiked to nailing strips fastened to the top flange should furnish lateral support. Corrugated sheet-metal roofs are sometimes attached to purlins by metal straps or clips. It is questionable whether such connections provide dependable lateral support.

Lateral bracing must be adequate to hold the braced beam in position. Thus, stiffness as well as strength is required. As a general rule, bracing will be adequate if each lateral brace is designed for 2% of the compressive force in the flanges of the beam it braces. This rule of thumb is based on observations from laboratory test.

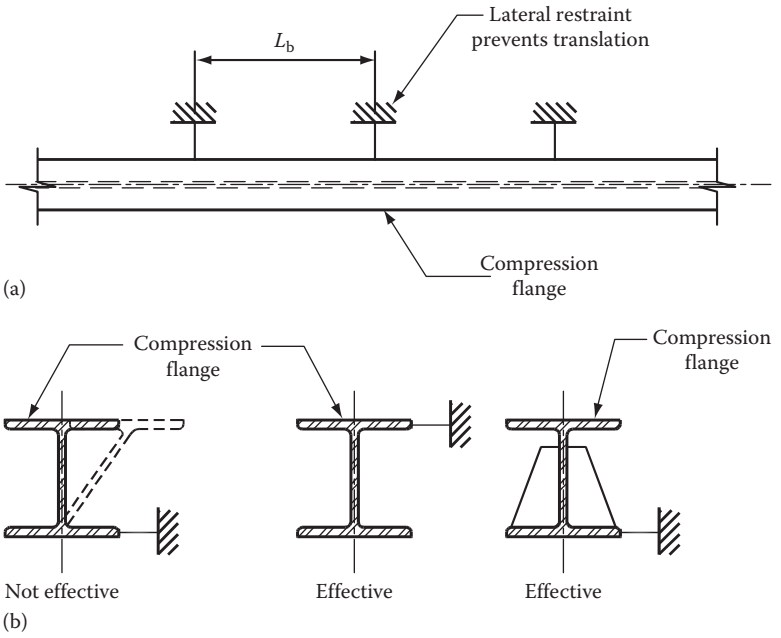


FIGURE 3.12 Concept of L_b, Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section: (a) lateral restraints limit effective length of compression flange; (b) effective lateral restraint must prevent translation of compression flange.

Conceptually, a wide flange beam with a moment of inertia about the principal axis considerably larger than about the minor axis is similar to a strip with a narrow rectangular cross section. If bent in its plane by couples M applied at the ends (Figure 3.13), the beam may become unstable and at a certain critical value of M buckle sideways as shown in the figure. Such lateral buckling accompanied by twist may be of practical importance and should be considered in the case of beams without lateral support if the flexural rigidity of the beam in the plane of bending is very large in comparison with its lateral rigidity.

3.3.1 Compact, Noncompact, and Slender Sections

There are two main categories of beams, compact and noncompact. Compact beams by virtue for their special controls on their geometry are particularly stable. Compact section criteria are based on the yield strength of steel, the type of cross section, the ratios of width to thickness of the elements of cross section. Members, whether they are rolled shapes or shapes made from plates, are compact if they fulfill certain criteria. To meet all criteria, the member must cope with certain stringent limits, of b_f/t_f and h/t_w ratios, have unsupported length of compression flange less than L_c and be bent about its major axis. Nonfulfillment of any of the compact criteria will degrade the member to noncompact. If the width to thickness ratios exceed the limiting width to thickness ratios of noncompact elements, then it is classified as a slender element section.

3.3.2 Flexural Design of Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis

The design flexural strength, $\phi_b M_n$ shall be determined using plastic section modulus M_n and $\phi_b = 0.9$.

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling.

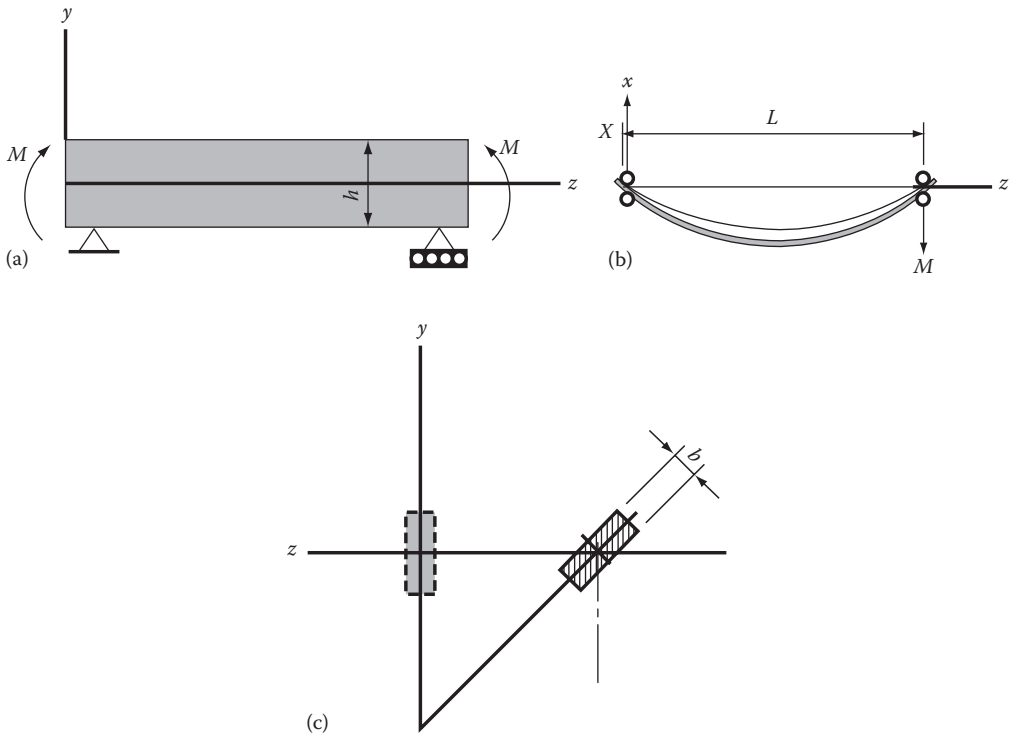


FIGURE 3.13 Buckling of narrow deep beams: (a) simply supported beam subject to moments M in the vertical plane; (b) plan showing lateral displacement of beam; (c) section showing lateral and torsional displacements.

1. Yielding

$$M_n = M_p = F_y Z_x$$

where
F_y is the specified minimum yield stress of the type of steel being used, ksi
Z_x is the plastic section modulus about the x-axis, in.^{3}

2. Laterial-torsional buckling
   (a) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.
   (b) When $L_p < L_b \le L_r$

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_p$$

   (c) When $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p$$

where L_b is the length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in.

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_{ts})^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}} \right)^2}$$

and where

E is the modulus of elasticity of steel = 29,000 ksi
J is the torsional constant, in.^{4}
S_x is the elastic section modulus taken about the x-axis, in.^{3}

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y}{E} \frac{S_x h_o}{J_c} \right)^2}}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}$$

and

For a doubly symmetric I-shape: $c = 1$

For a channel: $c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}}$

where h_o is the distance between the flange centroids, in.

The formulas given above apply to

1. Doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges
   All current ASTM A6 W, S, M, C and MC shapes except W21 × 48, W14 × 99, W14 × 90, W12 × 65, W10 × 12, W8 × 31, W8 × 10, W6 × 15, W6 × 15, W6 × 9, W6 × 8.5, and M4 × 6 have compact flanges for $F_y = 50$ ksi; all current ASTM A6 W, S, M, HP, C, and MC shapes have compact webs at $F_y \leq 65$ ksi.
2. Beams and girders that are restrained against rotation about their longitudinal axis at the supports

The highest possible nominal flexural strength is the plastic moment, $M_n = M_p$. Being able to use this value in design represents the optimum use of the steel. In order to attain M_p the beam cross section must be compact and the member must be laterally braced. Compactness depends on the flange and web plate width-to-thickness ratios. When compact conditions are not met, the available nominal flexural strength diminishes. For laterally braced beams, the plastic moment region extends over the range of plate width-thickness ratios λ terminating at λ_r. This is the range where the section is noncompact. Beyond λ_r the section is a slender-element section.

These three ranges are illustrated in Figure 3.14 for the case of rolled wide-flange members for the limit state of flange local buckling. The curve in that figure shows the relationship between the flange width–thickness ratio $b_f/2t_f$ and the nominal flexural strength M_n.

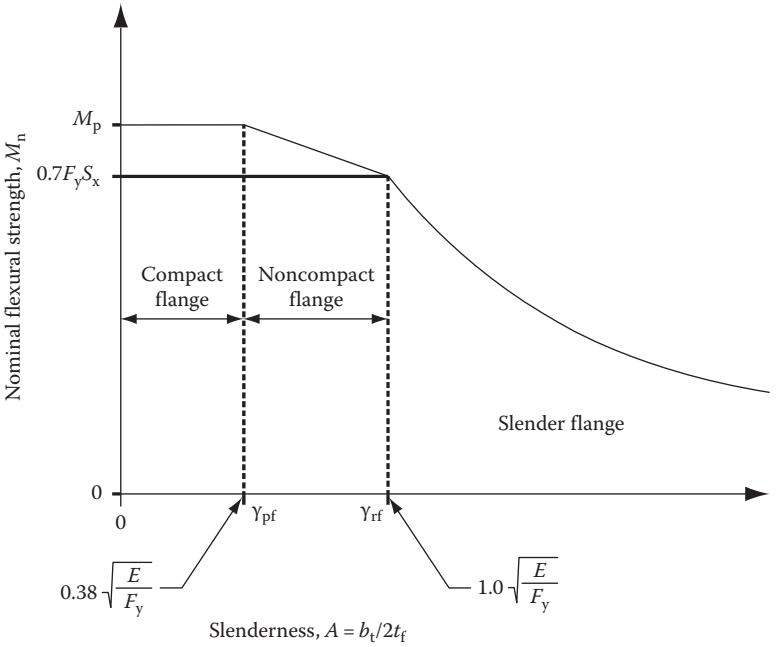


FIGURE 3.14 Nominal flexural strength as a function of flange width–thickness ratio of rolled I-shapes.

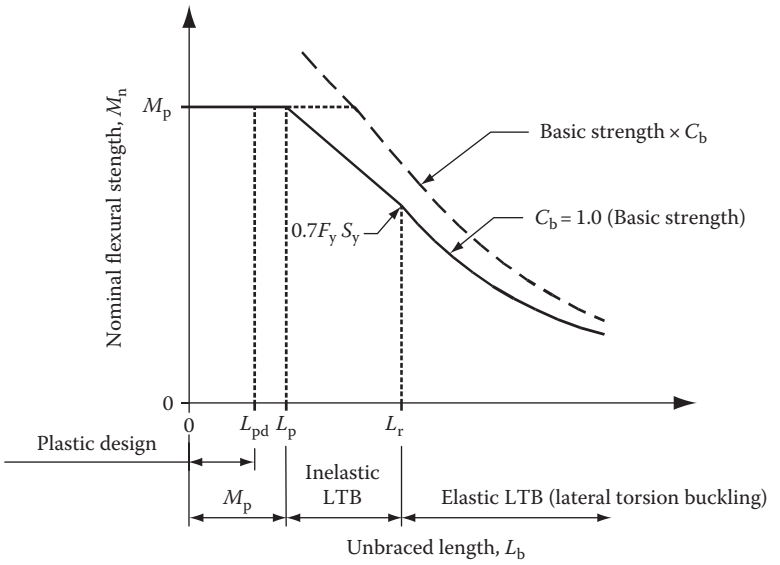


FIGURE 3.15 Nominal flexural strength as a function of unbraced length and moment gradient.

The basic relationship between the nominal flexural strength, M_n, and the unbraced length, L_b, for the limit state of lateral-torsional buckling is shown in Figure 3.15 for a compact section [$W27 \times 84F_y = 50$ ksi] subjected to uniform bending, $C_b = 1.0$.

There are four principal zones defined on the basic curve by the lengths L_{pd}, L_p, and L_r. Note that L_p is the limiting unbraced length needed to reach M_p.

Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r. The range of inelastic lateral-torsional buckling is a straight line between the defined limits M_p at L_p and $0.7F_yS_x$ at L_r. Buckling strength in the elastic region is given by AISC 360-05 Equations F2-3 and F2-4 for I-shaped members. The length L_{pd} is the maximum unbraced length needed for plastic design.

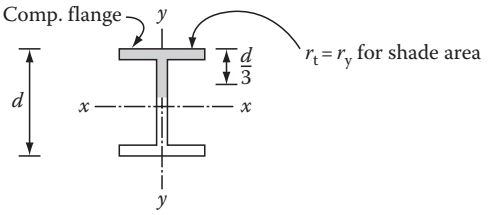


FIGURE 3.16 Beam section showing area for the calculation of r_t, the radius of gyration used in determining L_p and L_r.

Shown in Figure 3.16 is the beam area for the calculation of radius of gyration r_t required for determining L_p and L_r.

3.3.3 Design Examples, Members Subject to Bending and Shear

3.3.3.1 General Comments

According to the AISC Specification, the nominal capacity M_p (in kip) of a steel section in flexure is equal to the plastic moment:

$$M_p = ZF_y$$

where

Z is the plastic section modulus (in.^{3})
F_y is the steel yield strength (ksi)

However, this applies only when local or lateral torsional buckling of the compression flange is not a governing criterion. The nominal capacity M_p is reduced when the compression flange is not braced laterally for a length that exceeds the limiting unbraced length for full plastic bending capacity L_p. Also, the nominal moment capacity is less than M_p when the ratio of the compression-element width to its thickness exceeds limiting slenderness parameters for compact sections. The same is true for the effect of the ratio of web depth to thickness.

In addition to strength requirements for design of beams, serviceability is important. Deflection limitations defined by local codes or standards of practice must be maintained in selecting member sizes. Dynamic properties or the beams are also important design parameters in determining the vibration behavior of floor systems for various uses.

The factored shear strength, V_v of the web of wide-flange sections shall be verified with the design shear strength, especially if large concentrated loads occur near the supports. For LRFD, the AISC Specification requires that the factored shear V_v (kip) not exceed ϕV_n

$$V_v \leq \phi_v V_n$$

Note that $\phi_v = 0.9$ for all cases except for webs of wide flanges with $h/t_w \text{ " } 2.24\sqrt{E/F_y}$.

The nominal shear strength V_n (kip) is calculated as follows:

$$V_n = 0.6F_{yw}A_wC_v$$

where

F_{yw} is the yield strength of the web, ksi
A_w is the web area, in.

C_v is the web shear coefficient determined as follows:

1. For $h/t_w \le 1.10\sqrt{k_v E/F_{yw}}$ or $hlt_w \le 2.34\sqrt{E/F_{yw}}$ for rolled I-shaped members ($k_v = 5$),

$$C_v = 1.0, \quad \phi_v = 1.0$$

2. For $110\sqrt{k_v E/F_{yw}} \le h/t_w \le 1.37\sqrt{k_v E/F_{yw}}$,

$$C_v = \frac{1.10\sqrt{k_v E/F_{yw}}}{h/t_w}, \quad \phi_v = 0.9$$

3. For $h/t_w > 1.37\sqrt{k_v E/F_{yw}}$,

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_{yw}}, \quad \phi_v = 0.9$$

$$k_v = 5 + \frac{5}{(a+h)^2}$$
$$= 5 \text{ when } a/h > 3 \quad \text{or} \quad a/h > [260/(h/t)]^2$$
$$= 5 \text{ if no stiffeners are used}$$

where a is the distance between transverse stiffeners.

The reader is referred to Chapter G, of ASCE 360-05 specifications for further requirements of shear design.

3.3.3.2 Simple-Span Beam, Braced Top Flange

Given

A 40-ft simply supported wide flange beam of Grade 50 steel with $F_y = 50$ ksi. The beam supports open web joists spaced at 4 ft center to center with 2-in. deep metal deck and a 3-in. thick normal weight concrete topping. For purposes of beam design, assume unfactored uniformly distributed dead load of 2.5 kip/ft (including self weight of beam) and a live load of 1.1 kip/ft.

Required

Design the beam for flexure while limiting the live load deflection to $L/360 = \frac{40 \times 12}{360} = 1.34$ in. Comment on cambering of beams. Also, design for shear.

Solution

$$W_u = 1.4 \times 2.5 = 3.5 \text{ kip/ft}$$

$$W_u = 1.2 \times 2.5 + 1.6 \times 1.1$$
$$= 4.76 \text{ kip/ft (governs)}$$

$$\text{Factored moment } M_u = 4.76 \times 40^2/8 = 952 \text{ kft}$$

Although the top flange of the beam is not continually braced, the presence of closely spaced joists attached to the beam, allows us to consider the top flange as fully braced. Therefore, the required plastic modulus Z_x, is determined as follows.

The factored moment M_u may not exceed the design strength of $\phi M_n = \phi Z_x F_y$

Therefore, required $Z_x = \dfrac{952 \times 12}{0.9 \times 50} = 253.87 \text{ in.}^3$

A wide-flange section $W24 \times 94$ with $Z = 278 \text{ in.}^3$ is adequate

The moment of inertia of $W24 \times 94$ is 2700 in.^{4} and the modulus of elasticity for steel is 29,000 ksi. Hence, the deflection due to live load is

$$\Delta_L = \frac{5W_L L^4}{384EI} = \frac{5 \times 1.1 \times 40^4 \times 12^3}{384 \times 29{,}000 \times 2700} = 0.81 \text{ in.}$$

This is less than $L/360 = \dfrac{40 \times 12}{360} = 1.34$ in. Hence, the beam satisfies the given live load deflection criteria.

The immediate dead load deflection due to the weight of concrete topping is important in maintaining reasonably level floors. If the calculated load deflection multiplied by a factor equal to 0.75 (to account for some fixity at simply supported connections) exceeds ¾-in., typical practice in building construction is to camber the beams. The purpose is to achieve a commercially acceptable level floor and to prevent ponding of wet concrete during placement.

In the present example, deflection due to a dead load of 2.5 kip/ft is

$$\Delta_L = \frac{5 \times 2.5 \times 40^4 \times 1{,}732}{384 \times 29{,}000 \times 2{,}700} = 1.84 \text{ in.}$$

This is considerably larger than 0.75 in., the cut of deflection value above which camber is recommended. Therefore, cambering of the example beam at midspan for 75% of the calculated deflection is recommended.

Required camber = 0.75 × 1.84 = 1.38 in., say $1\frac{3}{8}$ in.

3.3.3.2.1 Shear Design

For review of the shear capacity of the section, the depth/thickness ratio of the web for W24 × 94 is

$$\frac{h}{t_w} = 54.6 < 2.45\sqrt{\frac{29{,}000}{50}} = 59.0$$

Therefore $C_v = 1.0$, $\phi_v = 0.9$.

The design shear strength is given by

$$\phi V_n = 0.9 \times 0.6 \times F_y \times A_w C_v$$

Note $C_v = 1.0$

$$\phi V_n = 0.9 \times 0.6 \times 50 \times 23.6 \times 0.395 = 252 \text{ kip}$$

The factored shear force near the support is

$$V_u = 4.76 \times \frac{40}{2} = 95.2 \text{ kip} < 252\text{kip, OK}$$

As illustrated in this example, it usually is not necessary to review shear capacity of typical floor beams, subject to uniformly distributed loads.

3.3.3.3 Simple-Span Beam, Unbraced Top Flange

Given

ASTM A992, Grade 50 steel beam with a simple span of 30 ft carrying a transfer column with $P_u = 70$ kip at midspan. No floor deck is present on either side of the beam. The beam top flange is therefore unbraced for the entire length. At center span, the transfer column is not capable of laterally bracing the top flange; see Figure 3.17 for beam schematics.

Required

Design a W18 wide flange beam to carry the given loads. Neglect self-weight of beam.

Solution

Factored moment at midspan, $M_u = 70 \times \frac{30}{4} = 525$ kip ft. From the Beam Table 3-16 of the Steel Manual, we note that W18 × 71 as the lightest beam with $\phi M_p = 548$ kip ft exceeding the required $M_u = 525$ kip ft. This beam would have worked if the beam had

1. Lateral bracing of top flange at a spacing not exceeding L_p ($L_p = 6.0$ ft for W18 × 71 from Beam Table 3-16)
2. A torsional bracing at a distance not exceeding L_r, the rotation of the beam ($L_r = 19.6$ ft for W18 × 71) from the same Table

For our W18 × 71, $L_p = 30$ ft. is much greater than 19.6 ft, the maximum distance between beam bracings for developing full plastic capacity. Hence, its nominal bending strength, M_n will be less than $M_p = F_y Z_x$.

Therefore, calculations to find the lowest-weight, larger-size beam must be continued. In practice, however, this trial-and-error process can be eliminated by using beam-selector charts in the AISC Manual. These charts give the beam design moment corresponding to unbraced length for various rolled sections. See AISC manual Tables 3-10 through 3-11.

A sample design table for $M_u = \phi M_n$ ranging from 450 kip-ft to 600 kip-ft is shown in Figure 3.18. For $\phi M_n > 525$ kip-ft-and $L = 30$ ft, the figure indicates that a W18 × 106 of A992 steel satisfies the criteria. As a check, the following calculation is made with the properties of the W18 × 106.

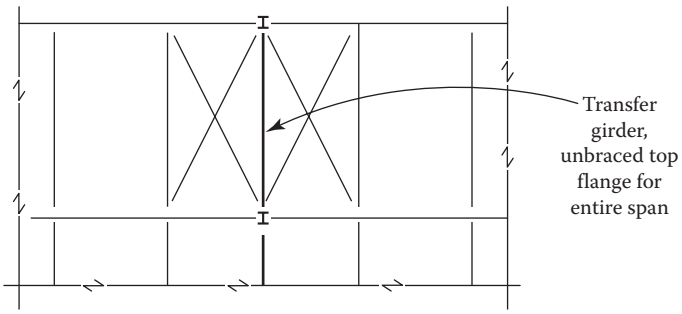


FIGURE 3.17 Design example: Transfer girder, unbraced top flange.

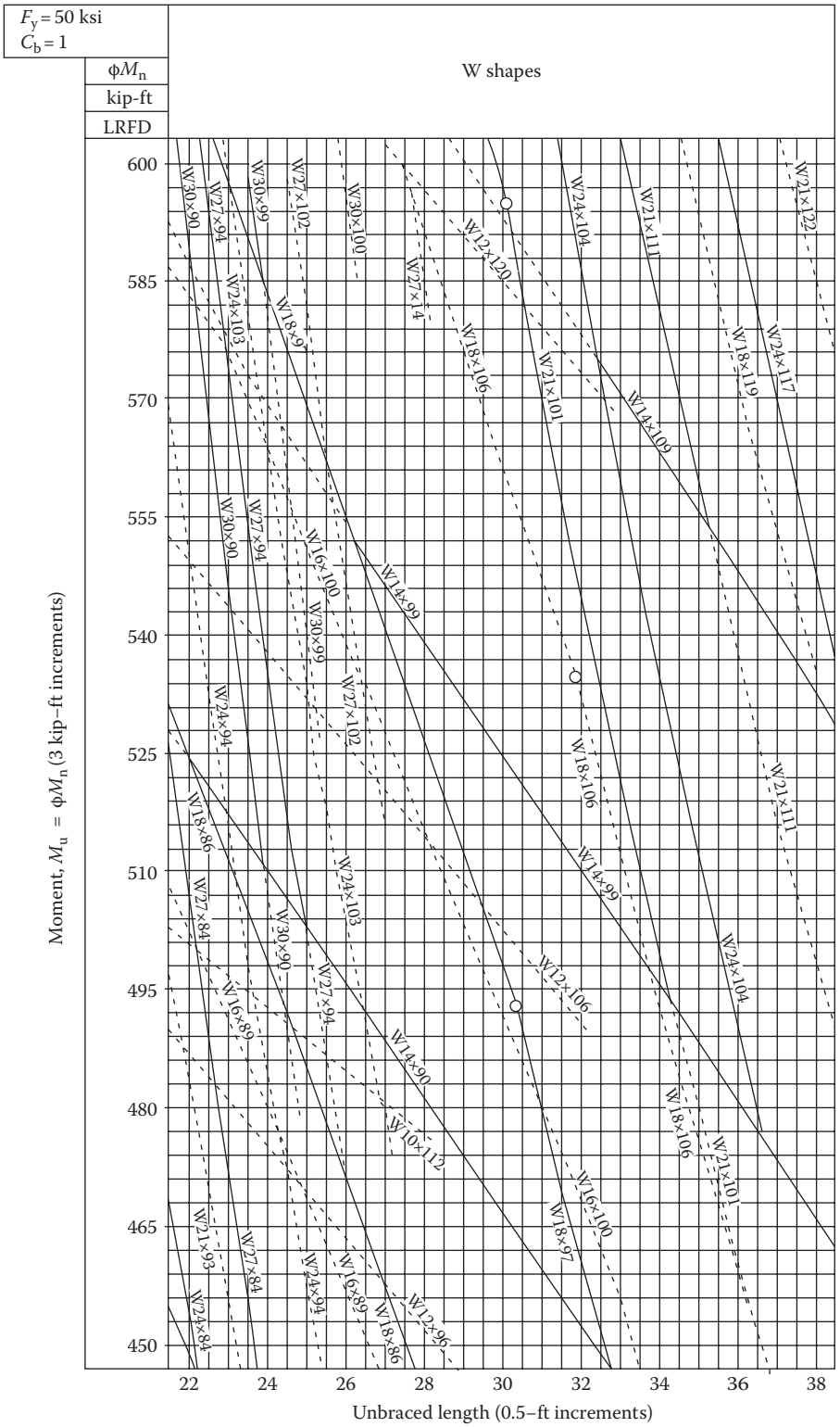


FIGURE 3.18 Moment M_u versus unbraced length.

Properties of $W18 \times 106$

$h_0 = 18.7 - 0.81 = 17.89$
$J = 7.48\,\text{in.}^4$
$C_w = 17{,}400\,\text{in.}^6$
$C_b = 1.0$ (conservative)
$C = 1.0$ for doubly symmetric I-shape
$S_x = 204\,\text{in.}^3$
$I_y = 220\,\text{in.}^4$

$$r_{tn}^2 = \sqrt{\frac{220 \times 17{,}400}{204}} = 9.59$$

$$r_{tn} = \sqrt{9.59} = 3.1\ \text{in.}$$

$$F_{pn} = \frac{C_b \pi^2 E}{(L_b/r_{tn})^2} \cdot \sqrt{1 + 0.078 \frac{Jc}{S_x h_0} \left(\frac{L_b}{r_{tn}}\right)^2}$$

$$= \frac{1 \times \pi^2 \times 29{,}000}{\left(\frac{30 \times 12}{3.1}\right)^2} \sqrt{1 + 0.078 \frac{7.48}{204 \times 17.89} \left(\frac{30 \times 12}{3.1}\right)^2}$$

$$= 37.76 \cdot \text{ksi}$$

$$\phi M_n = 0.9 \times 204 \times 37.76 = 6934.3\ \text{kip-in.}$$

$$= 577.85\ \text{kip-ft}$$

$$> 525\ \text{kip-ft O.K.}$$

Observe, for convenience we assumed $C_b = 1.0$ in the calculations above.
In fact, exact calculations will show $C_b = 1.14$.
Therefore, improved value of $\phi M_n = 1.14 \times 577.85$

$$= 658.75\ \text{kip-ft}$$

$$\gg 525\ \text{kip-ft O.K.}$$

3.4 TENSION MEMBERS

Although any type of steel cross section may be used as a tension member, in practice the selection is influenced by the type of connections required at the ends. The nominal tensile strength P_n is based on both the yield criteria over the gross section, and the fracture criteria based on effective net area. Thus,

$$P_n = F_y A_g \quad \text{(yield criteria, based on gross area)}$$

$$P_n = F_u A_e \quad \text{(fracture criteria, based on net area)}$$

In the above equations, F_y is the specified minimum yield stress of steel and F_u is the minimum tensile strength, both in the units of kips per square inch.

A_g is the gross area of the section and A_e is the effective net area at the connection. The effective area is given by

$$A_e = UA_n$$

where

A_n is the net area

U is the reduction coefficient (see Table D3.1 of ANSI/AISC 360-05 Specification)

If the end connections are welded and the area of the tension member is not otherwise reduced, $A_g = A_e$.

For bolted members, however, determination of the reduced cross-section area A_n is required by searching for different possible chains of holes. When a chain of holes is straight and perpendicular to the member axis,

$$A_n = A_g - \phi Nt$$

where

ϕ is the effective diameter of the hold = ϕ_b + 1/8 in.: (ϕ_b is the bolt diameter)

N is the number of bolts in the chain

t is the thickness of material being connected

Although the slenderness ratio l/r for tension members is not critical, AISC 360-05 recommends a non-mandatory limit of $l/r \leq 300$. Flexible members such as cables, round, or square rods, or thin wide bars are excluded from this requirement.

3.4.1 Design Examples

3.4.1.1 Plate in Tension, Bolted Connection

Given

A bolted joint connecting two 12 in. wide × ¾ "thick plates with 1 in. ϕ bolts in standard holes (see Figure 3.19). The plate material is F_y = 50 ksi and F_u = 65 ksi. Assume block shear does not govern and that the bolts are satisfactory.

Required

Maximum tensile force that can be applied to the plates. Use Working Stress Design, WSD.

Solution

Diameter of hole d_h = 1 + 1/8 = 1.125 in.

Gross area of plate $A_g = 12 \times 0.75 = 9\,\text{in.}^2$

$$0.85A_g = A_{e(\max.)} = 0.85 \times 9 = 7.65\,\text{in.}^2$$

For fracture line A-A, see Figure 3.19, the effective net are of the plate is

$$A_e = t(w - 2d_h) = 0.75(12 - 2 \times 1.125) = 7.31\,\text{in.}^2$$

$$\ll 0.85A_g = 7.65\,\text{in.}^2\ \text{OK}$$

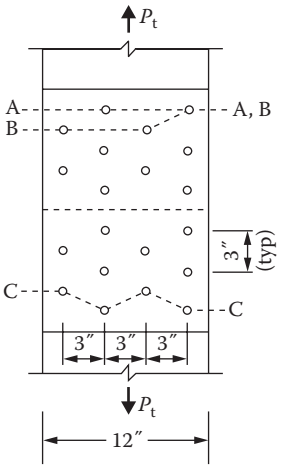


FIGURE 3.19 Design example: plates in tension, bolted splice.

The tensile capacity for this fracture condition along line A-A is

$$P_1 = 0.5F_uA_e$$
$$= 0.5 \times 65 \times 7.31$$
$$= 237.6 \text{ kip.}$$

For the staggered fracture line B-B, the effective net area of the plate is

$$A_e = t\left(w - 3d_h + \frac{S^2}{4g}\right)$$

Noting that $S = 1.5$ in., and $g = 3$ in.

$$A_e = 0.75\left(12 - 3 \times 1.125 + \frac{1.5^2}{4 \times 3}\right)$$
$$= 6.61 \text{ in.}^2 < 0.85A_g = 7.65 \text{ in.}$$

Assuming equal distribution of tensile load among the given number of 10 bolts, each bolt resists 10% of the applied tensile force. Therefore, the fracture plane B-B is required to resist only 90% of the applied force. Expressed in another way, the equivalent net area along B-B is

$$A_{e(equiv)} = \frac{A_e}{0.9} = \frac{6.61}{0.9} = 7.35 \text{ in.}^2$$

$$P_t = 0.5F_u \times A_e$$
$$= 0.5 \times 65 \times 7.35 = 238.9 \text{ kip}$$

For the staggered fracture line C-C,

$$A_e = t\left(w - 4d_h + \frac{3S^2}{4g}\right)$$

$$= 0.75\left(12 - 4 \times 1.125 + \frac{3 \times 1.5^2}{4 \times 3}\right)$$

$$= 6.04 \text{ in.}^2,\ A_{e(\max.)} = 7.65 \text{ in.}^2 \text{ OK}$$

The tensile capacity is

$$P_t = 0.5\, F_h A_e$$

$$= 0.5 \times 65 \times 6.04$$

$$= 196.5 \text{ kip} \leftarrow \text{governs}$$

The tensile capacity based on yielding condition is

$$P_t = 0.6\, F_y A_g$$

$$= 0.6 \times 50 \times 9$$

$$= 270 \text{ kip}$$

Therefore, tension capacity of 196.5 kip, as calculated for fracture condition along C-C controls the design.

3.4.1.2 Plate in Tension, Welded Connection

Given

A ¾ in. thick × 4 in. wide plate welded to a gusset plate with two longitudinal welds (Figure 3.20). The plate material is $F_y = 36$ ksi and $F_u = 58$ ksi. Assume that block shear does not control the design and that the welds are adequate.

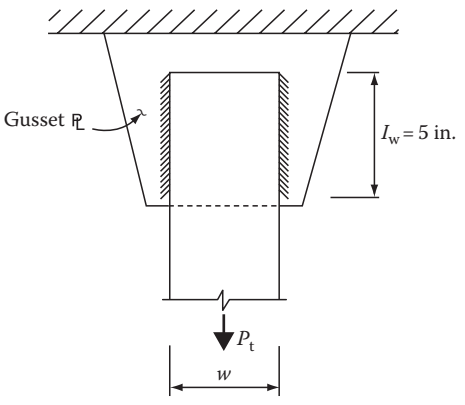


FIGURE 3.20 Design example: tension member—welded connection.

Required
Maximum tensile force that can be applied to the plate. Use Working Stress Design, WSD.

Solution
The design parameters are width of plate $w = 4$ in., thickness $t = 0.75$ in., and weld length $l = 5$ in.

$$\frac{l}{w} = \frac{5}{4} = 1.25$$

The reduction coefficient U that accounts for the effects of eccentricity and shear lag is

$$U = 0.87$$

The gross are of the plate is

$$A_g = 0.75 \times 4 = 3 \text{ in.}^2$$

The effective net area is

$$A_e = UA_g$$
$$= 0.87 \times 3 = 2.61 \text{ in.}^2$$

The corresponding tensile capacity is

$$P_t = 0.5F_uA_e$$
$$= 0.5 \times 58 \times 2.61$$
$$= 75.69 \text{ kip}$$

The tension capacity for yielding condition is

$$P_t = 0.6F_yA_g$$
$$= 0.6 \times 36 \times 3$$
$$= 64.80 \text{ kip} \leftarrow \text{governs}$$

3.4.1.3 Double-Angle Hanger

Given
The ultimate tension load, $P_u = 200$ kip.

Required
Design a double-angle member of ASTM A36, $F_y = 36$ ksi and $F_u = 58$ ksi. The connection of the hanger consists of one row of 1in. diameter, A325 bolts in with the hole size $1\,{}^{1}\!/_{16}$ in.

Solution
The required area of the angle is

$$A_g = \frac{200}{0.9 \times 36} = 6.17 \text{ in.}^2$$

$$A_n = \frac{200}{0.75 \times 58} = 4.6 \text{ in.}^2$$

Try two angles, 5 × 3 × ½ in., with $A_g = 7.51$ in.^{2} for 1 in. diameter A325 bolts in the 5 in. leg with hole size 1 1/16 in.; the net area of the angle is

$$A_n = 7.51 - 2 \times \frac{1}{2} \times \frac{17}{16} = 6.45 \text{ in.}^2$$

Conservatively assume the reduction coefficient, U, to be applied to the net area of bolted connection = 0.9

The effective area is

$$A_e = UA_n = 0.90 \times 6.45 = 5.8 \text{ in.}^2 > 4.6 \text{ in.}^2$$

The selected 5 × 3 × 1/2 in. angles OK.

3.4.1.4 Bottom Chord of a Long-Span Truss

Given

A one-way, long span truss framing the roof of a sports facility. Truss top and bottom chords, diagonals and verticals consist of wide flange shapes of grade 50 steel with $F_y = 50$ ksi and $F_u = 65$ ksi.

The factored axial tension, P_u, in the bottom chord at which a splice is anticipated is equal to 2400 kip. The Engineer of Record has decided to use a bolted splice to transfer the tension load, between two wide flange sections of the bottom chord of the truss. Bolts are 1 1/8 in. diameter A325 slip critical, in 1 7/16 in. oversized holes to facilitate field-assembly of the truss. Bolts will be placed in two rows in each flange, and also in the web.

Required

Design a W14 wide flange section capable of transferring the tension load of $P_u = 2600$ kip, across the splice.

Solution

With a W14 wide-flange section of Grade 50 ksi steel, the required minimum gross area is

$$A_g = \frac{P_u}{\phi_t F_y} = \frac{2400}{0.9 \times 50} = 53.34 \text{ in.}^2$$

Note: ϕ_t for tensile yeilding = 0.9.

Try a W14 × 233 section which has an area = 68.5 in.^{2}, $t_f = 1.72$ in., $t_w = 1.07$ in.

$$\text{Net area } A_n = 68.5 - (2 \times 1.72 \times 1.437 \times 2 + 2 \times 1.07 \times 1.437)$$

$$= 68.5 - (9.887 + 3.075) = 55.53 \text{ in.}^2 > 53.34 \text{ in.}^2 \text{ OK}$$

Since all parts of the wide-flange section are connected at the splice connection, $U = 1$ for the determination of the effective area. Thus, $A_e = A_n = 55.53\,\text{in.}^2$. The design strength is

$$\phi_t P_n = 0.75 \times 65 \times 55.5 = 2707\ \text{kip} > 2400\ \text{kip OK}$$

Note: ϕ_t for tensile rupture = 0.75.

Use W14 × 233
See Figure 3.21 for splice detail schematics.

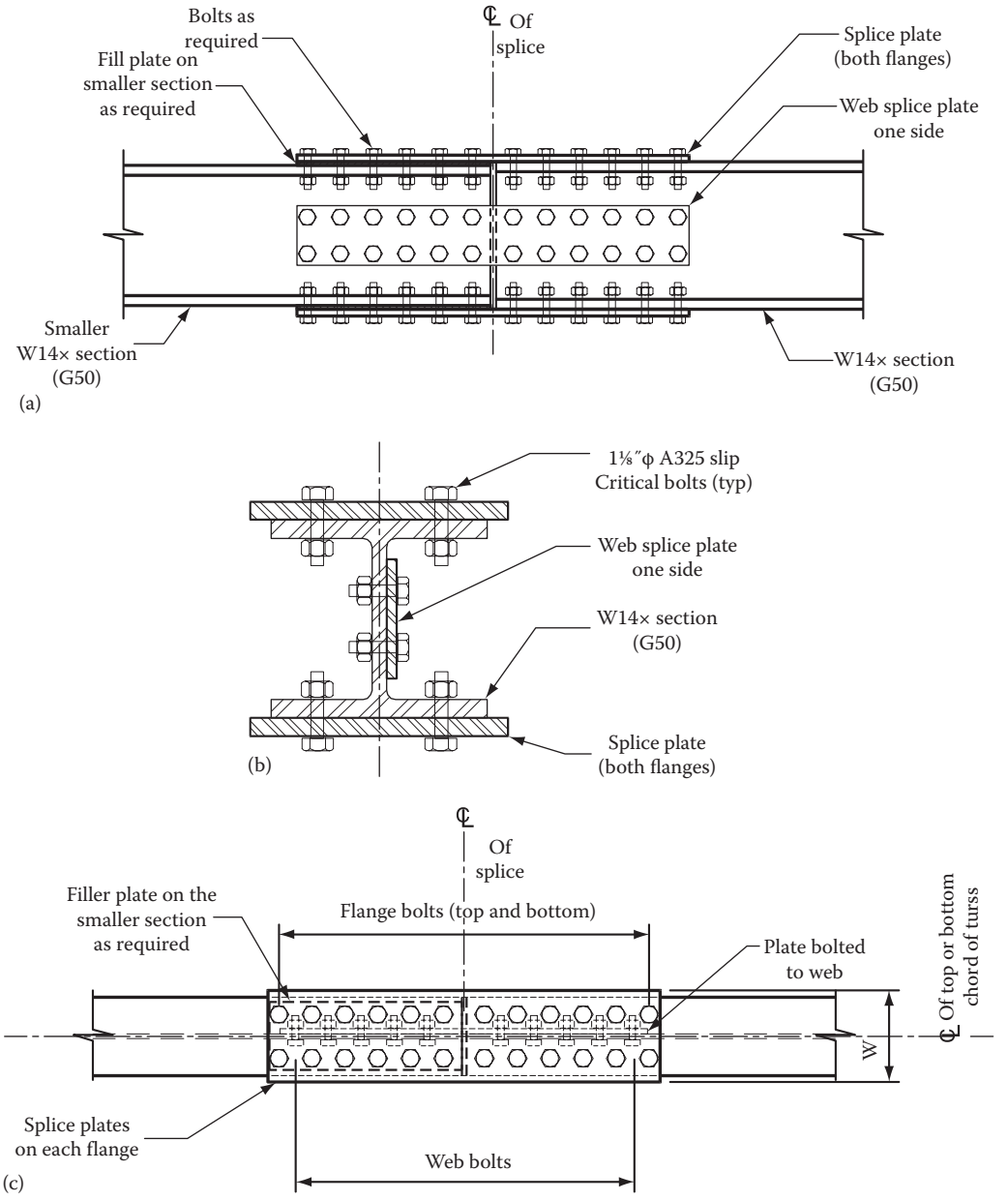


FIGURE 3.21 Design example: tension splice of truss bottom chord. (a) Elevation; (b) cross section; (c) plan.

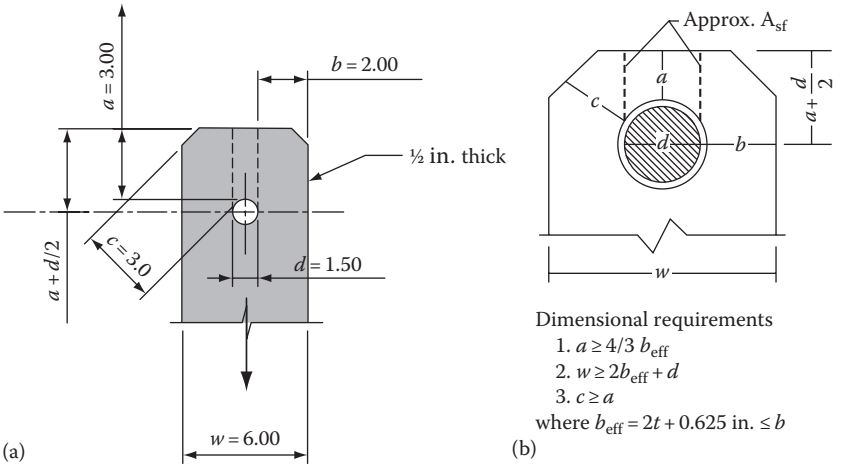


FIGURE 3.22 (a) Pin-connected tension member; (b) Dimensional requirements for pin-connected members.

3.4.1.5 Pin-Connected Tension Member

Given

Shown in Figure 3.22 is an ASTM A36 pin-connected tension member with the dimensions shown below, carries a dead load of 20 kip and a live load of 5 kip in tension. The diameter of the pin is 1½ in., in a 1/32–in. oversized hole. Assume that the pin itself is adequate. Verify the strength by LRFD.

Solution

Material Properties

Plate ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi

Geometric Properties

$w = 6.00$ in. $t = 0.500$ in. $d = 1.50$ in. $a = 3.00$ in. $c = 3.00$ in.

Check dimensional requirements

1. $b_{eff} = 2t + 0.63$ in. $= 2(0.500$ in.$) + 0.63$ in. $= 1.63$ in. < 2.00 in. OK.
2. $a \geq 1.33 b_{eff}$ 3.00 in. $\geq (1.33)(1.63$ in.$) = 2.17$ in. OK.
3. $w \geq 2b_{eff} + d$ 6.00 in. $\geq 2(1.63$ in.$) + 1.50$ in. $= 4.76$ in. OK.
4. $c > a$ 3.00 in. > 2.50 in. OK.

Calculate the required tensile strength

$$P_u = 1.2(20.0 \text{ kip}) + 1.6(5.00 \text{ kip}) = 32 \text{ kip}$$

Calculate the available tensile rupture strength on the net effective area

$$P_n = 2tb_{eff}F_u = (2)(0.500 \text{ in.})(1.63 \text{ in.})(58 \text{ ksi}) = 94.5 \text{ kip}$$

$$\phi_t = 0.75$$

$$\phi_t P_n = 0.75(94.5 \text{ kip}) = 70.9 \text{ kip}$$

Calculate the available shear rupture strength

$$A_{sf} = 2t(a + d/2) = 2(0.500 \text{ in.})[3.00 \text{ in.} + (1.50 \text{ in.}/2)] = 3.75 \text{ in.}^2$$

$$P_n = 0.6F_uA_{sf} = (0.6)(58 \text{ ksi})(3.75 \text{ in.}^2) = 130.5 \text{ kip}$$

$$\phi_{sf} = 0.75$$

$$\phi_{sf}P_n = 0.75(130.5 \text{ kip}) = 97.87 \text{ kip}$$

Calculate the available bearing strength

$$A_{pb} = 0.500 \text{ in.}(1.50 \text{ in.}) = 0.75 \text{ in.}^2$$

$$R_n = 1.8F_yA_{pb} = 1.8(36 \text{ ksi})(0.75 \text{ in.}^2) = 48.6 \text{ kip}$$

$$\phi_t = 0.75$$

$$\phi_tP_n = 0.75(48.6 \text{ kip}) = 36.45 \text{ kip}$$

Calculate the available tensile yielding strength

$$A_g = 6.00 \text{ in.}(0.500 \text{ in.}) = 3.0 \text{ in.}^2$$

$$P_n = F_yA_g = 36 \text{ ksi}(3.0 \text{ in.}^2) = 108.0 \text{ kip}$$

$$\phi_t = 0.90$$

$$\phi_tP_n = 0.90(108.0 \text{ kip}) = 97.2 \text{ kip}$$

The available tensile strength is governed by the bearing strength limit state

$$\phi_tP_n = 36.45 \text{ kip}$$

$$36.45 \text{ kip} > 32.0 \text{ kip OK.}$$

3.4.1.6 Eyebar Tension Member

Given

See Figure 3.23. A 5/8 in. thick eyebar member, ASTM A36, carries a dead load of 35 kip and a live load of 20 kip in tension. The pin diameter d_b is 3.25 in.

Required

Verify the strength by LRFD.

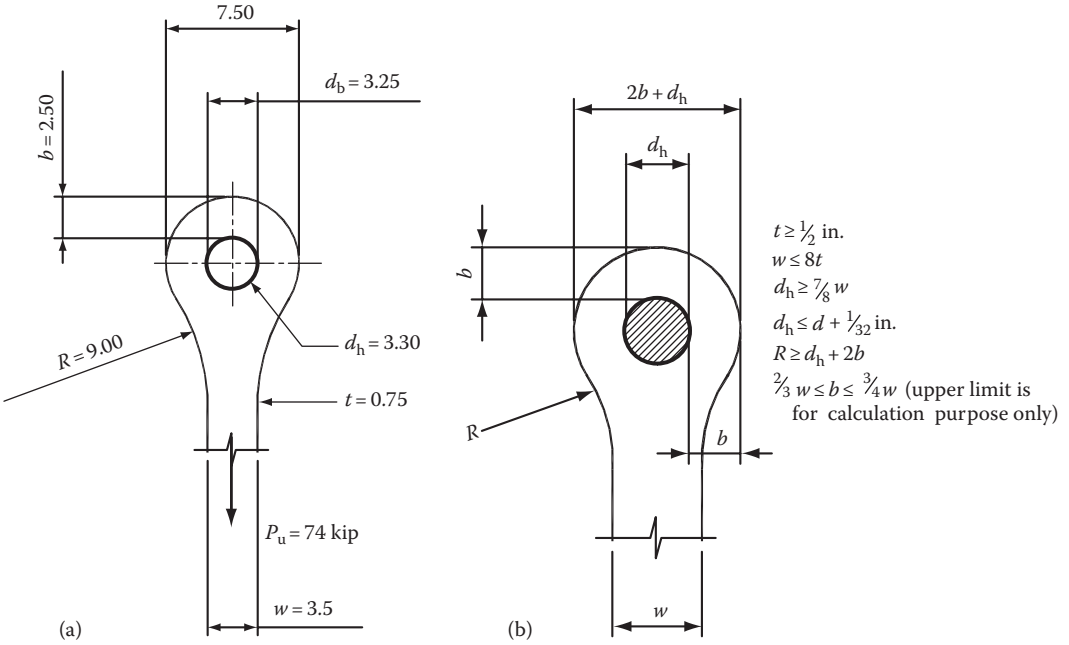


FIGURE 3.23 (a) Eye bar tension members; (b) dimensional limitation for eye bars.

Solution

Material Properties

Plate ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi

Geometric Properties

$w = 3.5$ in. $b = 2.50$ in. $t = 0.75$ in.
$d_b = 3.25$ in. $d_h = 3.30$ in. $R = 9.00$ in.

Check dimensional requirements

1.	$t \geq 1/2$ in.	0.75 in. $\geq$ 0.500 in.	OK.
2.	$w \leq 8t$	4.00 in. $\leq$ 8(0.75 in.) = 6 in.	OK.
3.	$d \geq 7/8w$	3.25 in. $\geq$ 7/8(3.5 in.) = 3.06 in.	OK.
4.	$d_h \leq d + 1/32$ in.	3.30 in. $\leq$ 3.25 in. + (1/32 in.) = 3.331 in.	OK.
5.	$R \geq d_h + 1/32$ in.	9.00 in. $\geq$ 3.30 in. + 2(2.50 in.) = 8.30 in.	OK.
6.	$2/3w \leq b \leq 3/4w$	2/3(3.350 in.) $\leq$ 2.50 in. $\leq$ 3/4(3.50 in.)	
		2/3(3.350 in.) $\leq$ 2.50 in. $\leq$ 3/4(3.50 in.)	OK.

Calculate the required tensile strength

$$P_u = 1.2(35.0 \text{ kip}) + 1.6(20.0 \text{ kip})$$

$$74 \text{ kip}$$

Calculate the available tensile yield strength at the eyebar body (at *w*)

$$A_g = 3.5 \text{ in.}(0.75) = 2.625 \text{ in.}$$

$$P_n = F_y A_g = (36 \text{ ksi})(2.625 \text{ in.}^2) = 94.5 \text{ kip}$$

$$\phi_t = 0.90$$

$$\phi_t P_n = 0.90(94.5 \text{ kip}) = 85 \text{ kip}$$

$$85.0 \text{ kip} > 74.0 \text{ kip OK.}$$

The eyebar tension member available strength is governed by the tension yield limit state.

Note: The eyebar detailing limitations ensure that the tensile yielding limit state at the eyebar body will control the strength of the eyebar itself. The pin should be checked for shear yielding, and if its material strength is lower than that of the eyebar, it should also be checked for bearing.

3.5 DESIGN FOR SHEAR, ADDITIONAL COMMENTS

There are two methods for determining the shear strength of singly or doubly symmetric I-shaped beams and built-up sections. One method does not utilize the post-buckling strength of the web, while the other does. Conservatively, the first method may be used for all designs. Consideration of the effect of bending on the shear strength is not required because the effect is deemed negligible.

Nominal shear strength
The nominal shear strength of a web is a product of the shear yield force $0.6F_yA_w$ and the shear-buckling reduction factor C_v.

The nominal shear strength, V_n, of unstiffened or stiffened webs, is given by

$$V_n = 0.6F_yA_wC_v$$

The available shear strength also referred to as design shear strength, V_v, shall not exceed ϕ_vV_n. Thus, $\phi_vV_n \geq V_v$.

Note that $\phi_v = 1.0$ for webs of rolled I-shaped members with $h/t_w \lesssim 2.24\sqrt{E/F_y}$. For all other cases $\phi_v = 0.9$.

(a) For webs of rolled I-shaped members with $h/t_w \text{ " } 2.24\sqrt{E/F_y}$:

$$C_v = 1.0, \quad \phi_v = 1.0$$

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

(i) For $h/t_w \text{ " } 1.10\sqrt{k_vE/F_y}$

$$C_v = 1.0, \quad \phi_v = 0.9$$

(ii) For $1.10\sqrt{k_vE/F_y} < h/t_w \leq 1.37\sqrt{k_vE/F_y}$

$$C_v = \frac{1.10\sqrt{k_vE/F_y}}{h/t_w}, \quad \phi_v = 0.9$$

(iii) For $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y}, \quad \phi_v = 0.9$$

where A_w is the overall depth times the web thickness, dt_w, in.^{2}

The web plate buckling coefficient, k_v, is determined as follows:

(i) For unstiffened webs with $h/t_w < 260$, $k_v = 5$
(ii) For stiffened webs,

$$k_v = 5 + \frac{5}{(a/h)^2}$$

$$= 5 \text{ when } a/h > 3.0 \quad \text{or} \quad a/h > \left[\frac{260}{(h/t_w)}\right]^2$$

where

a is the clear distance between transverse *stiffeners*, in. (mm)
h is for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)
= For built-up welded sections, the clear distance between flanges, in.
= For built-up bolted sections, the distance between *fastener* lines, in.
= For tees, the overall depth, in.

The shear buckling reduction factor, C_v, shown in Figure 3.24 has three segments.

For webs with $h/t_w \text{ " } 1.10\sqrt{Ek_v/F_y}$, the nominal shear strength V_n is based on buckling of the web.

When $h/t_w > 1.37\sqrt{Ek_v/F_y}$, the web strength is determine from the elastic buckling stress.

For practical purposes and without loss of accuracy, we may use

$$k_v = 5 + \frac{5}{(a/h)^2}$$

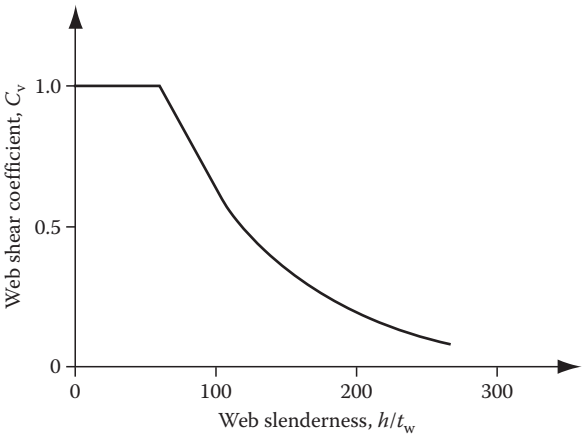


FIGURE 3.24 Shear buckling coefficient C_v for $F_y = 50$ ksi and $k_v = 5.0$.

When the panel ratio a/h becomes large, as in the case of webs without transverse stiffeners, then $k_v = 5$.

It should be noted that for all ASTM A6 W, S, M, and HP shapes except M12.5 × 12.4, M12.5 × 11.6, M12 × 11.8, M12 × 10.8, M12 × 10, M10 × 8, and M10 × 7.5, when $F_y \leq 50$ ksi, $C_v = 1.0$.

3.5.1 Transverse Stiffeners

Transverse *stiffeners* are not required where $h/t_w \leq 2.46\sqrt{E/F_y}$, or where the required shear strength is less than or equal to the available shear strength.

Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. When single stiffeners are used, they shall be attached to the compression flange, to resist any uplift tendency due to torsion in the flange. When *lateral bracing* is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit 1% of the total flange *force*.

When transverse stiffeners are needed, they must be rigid enough to cause a buckling node to form along the line of the stiffener. This requirement applies whether or not tension field action is counted upon.

The vertical component of the tension field force that is developed in the web panel must be resisted by the transverse stiffener. In addition to the rigidity required to keep the line of the stiffener as a nonmoving point for the buckled panel, the stiffener must also have a large enough area to resist the tension field reaction.

Transverse *stiffeners* subject to *tension field action* shall meet the following limitations:

1. $(b/t)_{st} \leq 0.56\sqrt{\dfrac{E}{F_{yst}}}$

2. $A_{st} > \dfrac{F_y}{F_{yst}}\left[0.15D_s ht_w\left(1-C_v\right)\dfrac{V_r}{V_c} - 18t_w^2\right] \geq 0$

where

$(b/t)_{st}$ is the width–thickness ratio of the stiffener

F_{yst} is the *specified minimum yield* stress of the stiffener material, ksi

C_v is the coefficient 1

D_s is 1.0 for stiffeners in pairs

= 1.8 for single angle stiffeners

= 2.4 for single place stiffeners

V_r is the *required shear strength* at the location of the stiffener, kip

V_c is the available shear strength; $\phi_v V_n$

When tension field action discussed presently, is permitted, the nominal shear strength, V_n, shall be

(a) For $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w$$

(b) For $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w\left(C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}\right)$$

3.5.2 Tension Field Action

Limits on the use of Tension Field Action

Consideration of *tension field action* is permitted for flanged members when the web plate is supported on all four sides by flanges or *stiffeners*. Consideration of tension field action is not permitted for

(a) *End panels* in all members with transverse stiffeners
(b) Members when a/h exceeds 3.0 or $[260/(h/t_w)]^2$
(c) $2A_w/(A_{fc} + A_{ft}) > 2.5$
(d) h/b_{fc} or $h/b_{ft} > 6.0$

where

A_{fc} is the area of compression flange, in.^{2}
A_{ft} is the area of tension flange, in.^{2}
b_{fc} is the width of compression flange, in.
b_{ft} is the width of tension flange, in.

In these cases, the nominal shear strength, V_n, shall be determined according to the provisions of Section G2 of AISC specifications.

The panels of the web of a built-up member, bounded on top and bottom by the flanges and on each side by the transverse stiffeners, are capable of carrying loads far in excess of their "web buckling" load.

When transverse stiffeners are properly spaced and are strong enough to act as compression struts, membrane stresses due to shear forces greater than those associated with the theoretical web buckling load form diagonal tension fields in the web panels. The resulting combination in effect results in a truss-type response that increases the strength to resist shear forces unaccounted for by the linear buckling theory.

The key point in the development of tension field action in the web of plate girders is the ability of the stiffeners to support the compression from the two panels on either side of the stiffener. In the case of end panels there is a panel only on one side. The support of the tension field forces is also reduced when the panel aspect ratio becomes too large. For this reason the inclusion of the tension field enhancement is not permitted for end panels and when a/h exceeds 3.0 or $\left[260/(h \cdot t_w)\right]^2$.

3.6 DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION (IN OTHER WORDS, MEMBERS SUBJECTED TO TORTURE)

Dealing with one of the principal actions to which structural members are subjected such as axial tension, compression, and flexure is no big sweat. In each case, we are able to clearly see the effects of the single action on the members and develop procedures for selecting member cross sections to resist the effect. As it happens, few structural members are single-effect members in their total function. As an example from nature, a tree trunk subject to wind is at once a compression member, a beam bending about any one of an infinite number of transverse axes, and a torsional shaft resisting the twisting effect of unbalanced wind drag on the unsymmetrically arranged leaf surfaces in the wind stream.

Similarly, in our man-made structural members, we need to recognize the many ways in which such members are exposed to other incidental effects often referred to as *secondary* effects. Observe that combined effects are not always secondary. These effects are often large enough to require attention in calculation and member selection.

A designer can often avoid combined effects by configuring systems in which the secondary effects do not exist. However, it is preferable and often economical to select members suitable for

combined effects. It becomes necessary therefore, to establish a systematic approach to do so without sacrificing safety. The level of safety should be at least not less than intended for single effect itself.

Using allowable stress design, ASD as an example, if there was one level of allowable stress common to all superposable effects, the design of a member subject to multiple effects would be easy. We would use a simple equation of the following form for the most highly stressed point on the cross section.

$$\sum_{i+1}^{I} f_i \le F$$

where each f_i is the stress caused by the individual ith effect.

But in structural steel design we have a multiplicity of different values of allowable stress F, each based on different considerations—whether yield, rupture, stability, or other. The most common approach is to seek some form of interaction equation that accounts for the difference in allowable stresses.

In 1923, the first AISC Specification required that the stresses due to flexure and compression be added and that the sum not exceed the allowable value. An interaction equation appeared first in the 1936 Specification, stating "Members subject to both axial and bending stresses shall be so proportioned that the quantity $\frac{f_a}{F_a} + \frac{f_b}{F_b}$ shall not exceed unity," in which F_a and F_b are, respectively, the axial and flexural allowable stresses permitted by this Specification, and f_a and f_b are the corresponding stresses due to the axial force and the bending moment, respectively. Thus,

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$$

This linear interaction equation was in force until the 1961 Specification, when it was modified to account for frame stability and for the $P\delta$ effect (i.e., the secondary bending between the ends of the members). The $P\Delta$ effect, that is, the second-order bending moment due to story sway, was not accommodated.

The modified interaction equation given in later AISC Specifications is of the following form:

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F_e^1}\right) F_b} \le 1.0$$

The allowable axial stress F_a is determined for an effective length of columns in moment frames. The term $\frac{1}{1-(f_a/F_e^1)}$ is the amplification of the interspan moment due to member deflection multiplied by the axial force (the $P\delta$ effect). C_m accounts for the effect of the moment gradient. This interaction equation has been part of all the subsequent editions of the AISC, ASD Specifications since 1961.

Amidst the myriad of equations given in later editions of AISC Specification, it is heartening to know that a linear equation similar to equation given above, is still permitted. The corresponding equation in AISC 360-05 is

$$\frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \le 1.0 \quad \text{(AISC Eq. H2-1)}$$

This equation provides a conservative and yet a simple way to deal with such members subjected to combined forces. The upper case stresses *f* are the required axial and flexural stresses computed by elastic analysis for the applicable loads, including second-order effects where appropriate, and the lower case stresses *F* are the available stresses corresponding to the limit state of yielding or buckling. The subscripts w and z refer to the principal axes of the unsymmetric cross section.

In the above equation,

f_a = the required axial stress at the point of consideration, ksi
F_a = the *available axial stress* at the point of consideration, ksi
f_{bw}, f_{bz} = the required flexural stress at the point of consideration, ksi
F_{bw}, F_{bz} = the *available flexural stress* at the point of consideration, ksi
w = the subscript relating symbol to major principal axis bending
z = the subscript relating symbol to minor principal axis bending

For design according to Section B33 (LRFD)

f_a = the required axial stress using LRFD *load combinations*, ksi
$F_a = \phi_c F_{cr}$ = the *design axial stress*, determine in accordance with Chapter E for compression or Section D2 for tension, ksi
f_{bw}, f_{bz} = the required flexural *stress* at the specific location in the cross section using LRFD load combinations, ksi
$F_{bw}, F_{bz} = \dfrac{\phi_b M_n}{S}$ = the *design flexural stress* determine in accordance with Chapter F, ksi

Use the section modulus for the specific location in the cross section and consider the sign of the stress.

ϕ_c = the *resistance factor* for compression = 0.90
ϕ_t = the resistance factor for tension
ϕ_b = the resistance factor for flexure = 0.90

The above equation shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as appropriate. When the axial *force* is compression, second order effects shall be included.

It should be noted that a more detailed analysis of the interaction of flexure and tension is permitted in lieu of the linear interaction equation.

3.7 DESIGN FOR STABILITY

The seemingly never-ending series of changes and additional complexity, built into the latest AISC 360 provisions for stability design, makes proper application of design standards one of the most significant demands of our profession. The challenge may even be overwhelming, but by focusing on the fundamental basis for, and the reasons behind the provisions, we may perhaps, best meet the challenges.

In the following sections, we attempt to explain the theory behind the AISC provisions by occassionally making reference to AISC manual, 9th edition.

3.7.1 Behavior of Beam Columns

Building columns, particularly those that resist lateral loads, often called frame columns, are in effect "beam–columns," that is, they are subject to simultaneous bending caused by lateral loads, and axial compression due to gravity loads. For example, consider the column shown in Figure 3.25,

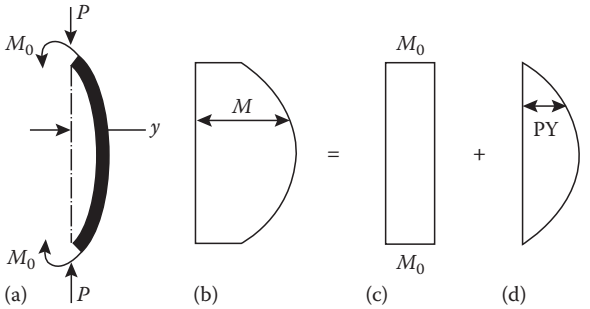


FIGURE 3.25 Moments in beam–columns: (a) column subjected to simultaneous axial load and bending moments; (b) combined moment diagram; (c) moment diagram due to equal end moments M_0; (d) moment due to $P\Delta$ effect.

subjected to simultaneous action of axial load and moments at the ends. At any location along the length of the column, the total moment M can be considered as a combination of the moment M_0 due to end moments plus the addition of the moment caused by the axial load, P, acting at an eccentricity y (Figure 3.25). Thus, $M = M_0 + P_y$. Since the deflection is maximum at midheight, the secondary moment also reaches its maximum value at that height. A similar effect is caused when bending is produced by a lateral load as shown in Figure 3.26. Since the deflection y, and, hence, the magnitude of the secondary moment are functions of the end moments, a differential equation formulation is required for determining the stresses in beam–columns. Simple cases of beam–columns subjected to end moments and concentrated loads, uniformly distributed loads, etc., have been solved some 200 years ago by differential equation techniques. In a practical structure, such a closed-form solution is extremely complicated if not impossible. Therefore, previous AISC Specifications such as the 9th Edition of *Steel Construction Manual*, discussed briefly in this section, gave provisions for approximate evaluation of the slenderness effect. The method in essence requires that the moments obtained by a so-called first-order analysis be magnified by a moment magnification factor.

The direct addition of the maximum $P\Delta$ moment to the maximum primary moment is valid only when the beam–column is subjected to equal moments at the ends subjecting the column to bend in a single curvature. For all other cases, it represents an upper bound, resulting in a moment magnification factor much larger than that in a real structure. If the two end moments are unequal but of the same sign, producing single curvature, the primary movement M_0 is certainly magnified but not to the same extent as when the moments are equal. If the end moments are of opposite sign, producing a reverse curvature in the column, the moment magnification effect will be very small. A moment

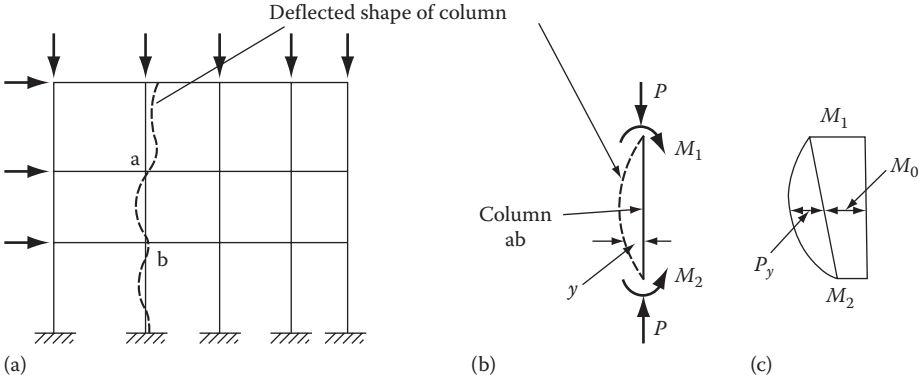


FIGURE 3.26 Behavior of building column: (a) building frame showing deflected shape of column; (b) column subjected to simultaneous action of axial loads and moments; (c) moment diagram due to end moment and $P\Delta$ effect.

magnification coefficient C_m is therefore used to take into account the relative magnitude and sense of the two end moments. It is given by the expression

$$C_m = 0.6 - 0.4M_1/M_2$$

In this equation, M_1 and M_2 represent the smaller and larger end moments, respectively. The ratio M_1/M_2 is positive when the column bends in a reverse curvature and negative when the moments produce a single curvature. As can be expected, when $M_1 = M_2$, as in a column subjected to equal end moments, the value of C_m become equal to 1.0. The foregoing expression applies only to members braced against side sway. For columns that are part of the lateral resisting system, the maximum moment magnification occurs, that is, $C_m = 1$, as illustrated in the following discussion.

Consider Figure 3.27 that shows the deflected shape of an unbraced portal frame subjected to the simultaneous action of gravity and lateral loads. Considering only the lateral loads the deflection of the portal frame may be represented by solid lines as shown in Figure 3.27a. The corresponding moments at the ends of a typical column are as shown in Figure 3.27b.

When axial load is imposed on the deflected frame, additional sway occurs in the frame, as shown by dashed lines in Figure 3.27a. This additional deflection imposes secondary moments in the column, as shown in Figure 3.27c. It is seen that both the primary and secondary moments are of the same sign and have maximum values at the same locations, namely, at the two ends of the columns. They are, therefore, fully additive, as shown in Figure 3.27d, meaning that the value of $C_m = 1$ for the unbraced frames.

One approach to address a stability problem is to modify individual member design in a manner that accounts for frame buckling effects. This is done by isolating a compression member together with its adjoining members at both ends and determining its critical load in terms of effective length factor K. The member is then analyzed a beam–column by a simplified interaction equation, which accounts for the moment magnification caused by the $P\Delta$ effect. Therefore, instead of the entire frame analysis for the PΔ effect, a relatively simple member analysis is substituted.

We have seen earlier that using a total moment obtained by the direct addition of secondary and primary moments results in an over design if both these moments do not occur at the same location. The coefficient C_m in the interaction equation prevents over design by reducing the design moment by taking into account the relative magnitude and sense of the moments occurring at the ends of columns.

Values of C_m less than 1.0 increase F_b, offsetting the effects of axial load when the shape of the elastic curve increases stability. When there is no joint translation and where the shape of the curve

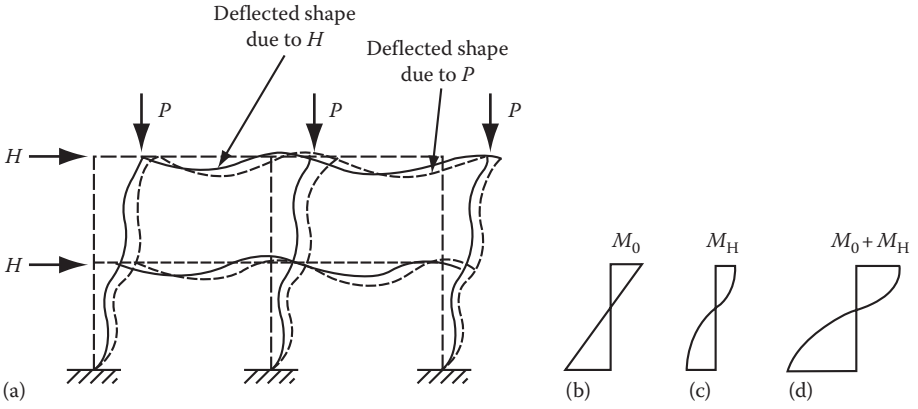


FIGURE 3.27 $P\Delta$ effects in laterally unbraced frames: (a) deflected shapes due to horizontal load H and vertical load P; (b) moment at column ends due to horizontal load H; (c) moment at column ends due to axial load P; (d) maximum moment due to H and P occurs at the ends of columns resulting in $C_m = 1.0$.

is not affected by transverse loading, reverse curvature bending may reduce C_m to as little as 0.4. To prevent a dramatic increase in F_b, which can result in unsafe designs, an interaction equation that does not contain the term C_m is also required to be satisfied. It should be evident by now that the calculation of stress ratios in frame columns is essentially an exercise in the evaluation of stresses due to simultaneous axial and bending action.

Prior to 1963, structural engineers could have made peace with the entire process of beam–column design by using the formula

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$$

Since then, engineers have had to deal with many seemingly formidable factors that have been added onto the above interaction equation. For example, the allowable bending stress F_b now has a factor $(1 - f_a/F_e')$ to account for the reduction in the bending capacity because of axial loads. The more the axial load in the column, the greater the reduction of F_b. Reducing the allowable stress is mathematically equivalent to increasing the design moment for the $P\Delta$ effects. F_e' is the familiar Euler's stress divided by the same factor of safety, 23/12, that governs the allowable stress long columns.

Consideration of only uniaxial bending reduces the AISC equations to the less intimidating format as follows:

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - f_a / F_e'\right) F_b} \le 1.0$$

$$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} \le 1.0$$

where

f_a is the axial stress in the column due to vertical loads
F_a is the allowable axial stress
f_b is the bending stress in the column
F_b is the allowable bending stress
C_m is the coefficient for modifying the actual bending moment to an equivalent moment diagram for purposes of evaluating secondary bending
F_e' is the Euler's stress divided by safety factor, 23/12
F_y is the yield stress of column steel

For the general case of axial load plus biaxial bending, the interaction equations for calculating the stress ratios areas follows: If f_a is compressive and $f_a/F_a > 0.15$, the compressive stress ratio CR is given by the larger of CR_{1a} and CR_{1b}, where

$$CR_{1a} = \frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \dfrac{f_a}{F_{ex}'}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \dfrac{f_a}{F_{ey}'}\right) F_{by}} \quad \text{(AISC 9th Edition, Equation H1-1)}$$

and

$$CR_{1b} = \frac{f_a}{0.60F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \quad \text{(AISC 9th Edition, Equation H1-2)}$$

If $f_a/F_a \leq 0.15$, $CR = CR_2$, where

$$CR_2 = \frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \quad \text{(AISC 9th Edition, Equation H1-3)}$$

C_{mx} and C_{my} are the coefficients that represent distribution of moment along member length. Although, as mentioned previously, their value could be as low as 0.4, in practice they are conservatively assumed equal to 1.0 in most cases.

If f_a is tensile or zero, the tensile stress ratio TR is given by the larger of TR_1 and TR_2, where

$$TR_1 = \frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \quad \text{(AISC 9th Edition, Equation H2-1)}$$

and

$$TR_2 = \frac{f_{bx}}{F_{bs}} + \frac{f_{by}}{F_{by}}$$

3.7.2 Buckling of Columns

The most fundamental theoretical formulation in stability design and buckling of columns is the Euler formula, which defines the elastic axial buckling strength of a member as

$$P = \frac{\pi^2 EI}{L^2}$$

The theory behind this strength equation assumes that the column is perfectly plumb and straight, behaves elastically, and has pinned ends that are restrained against lateral movement. These assumptions all are breached to various degrees in real structures. In practice, columns lean due to fabrication and erection tolerances and are not perfectly straight between floors due to mill and fabrication tolerances. Further, as a structure is loaded, deformations and drift also occur, adding second-order forces and moments. Not only must these effects be accounted for in design, but the fact that column buckling always involves both an axial force and bending effects also needs to be recognized.

3.7.3 Second-Order Effects

It is evident by now that flexure is introduced into an axially loaded member from the axial force acting through the sidesway of a frame and curvature of a member. This influence is referred to as a second-order effect. The analysis of the structure must be modified to capture the impact of these effects, as they will not be realized in a first-order design model of initially plumb frames and straight members. The primary effects to be considered are $P\Delta$ moments, associated with the sidesway of the structure, and $P\delta$ moments, associated with the curvature of each individual member as it deflects and deforms.

The moments generated by these effects can be captured in the analysis in several ways, and any method that a designer chooses to analyze a structure that accounts for each of the possible effects is acceptable. Therefore, it is equally permissible to analyze a structure by a direct, rigorous second-order analysis, or to use an approximate method of second-order analysis.

In the former case, the frame and member deformations are tracked directly within the analysis software as a part of the analysis. In the latter case, a first-order analysis is made and the resulting forces and moments are amplified using the variables B_1 and B_2. B_1 captures the amplification of forces and moments due to the curvature or out-of-straightness of the member ($P\delta$ effect), and B_2

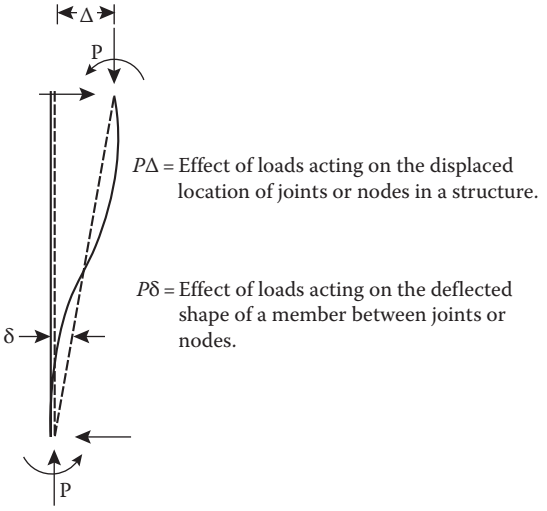


FIGURE 3.28 *P*Δ and *P*δ effects in beam columns.

captures the amplification of forces and moments from the drift of the frame (*P*Δ effect). These effects are illustrated for a single column in Figure 3.28.

3.7.4 Deformation of the Structure

While the structure can be analyzed conventionally for the deflections of individual members, it is important to be sure that these deflections are captured in a second-order analysis of the frame. It also is important to consider the effect of connection and panel-zone deformations in the analysis.

3.7.5 Residual Stresses

Residual stresses are introduced into structural shapes as a result of the production process. They include stresses due to temperature, as some elements of the hot rolled cross-section will cool faster than others, and also due to the effects of straightening that must be done to meet ASTM tolerances. Areas with residual stress will yield prior to the overall yielding of the section, causing the column to lose some of its stiffness before reaching its theoretical buckling strength. The effects of residual stresses on member strength are accounted for in determining the strength of the columns. However, the loss of stiffness due to residual stresses also will increase the frame and member deformations. This is accounted for in the Direct Analysis Method by using a reduced stiffness for all members in the analysis: multiplying the axial stiffness, *EA*, of all members by 0.8 and multiplying the flexural stiffness, *EI*, of all members by $0.8\tau_b$, where τ_b is the column stiffness reduction factor.

Stresses that remain in structural members after rolling or fabrication are known as *residual stresses*. The magnitude of the stresses is usually determined by removing longitudinal sections and measuring the resulting strain.

In a hot-rolled structural shape, the residual stresses result from unequal cooling rates after rolling. For example, in a wide-flange beam, the center of the flange cools more slowly and develops tensile residual stresses that are balanced by compressive stresses elsewhere on the cross section. See Figure 3.29C(a). In a welded member, tensile residual stresses develop near the weld and compressive stresses elsewhere provide equilibrium, as shown for the welded box section in Figure 3.29C(b).

In a welded I-shaped member, the stress condition in the edges of flanges before welding is reflected in the final residual stresses as shown in Figure 3.29C(c).

The residual-stress distributions are usually relatively constant along the length of the member. However, residual stresses also may occur at particular locations in a member, because of localized plastic flow from fabrication operations, such as cold straightening or heat straightening.

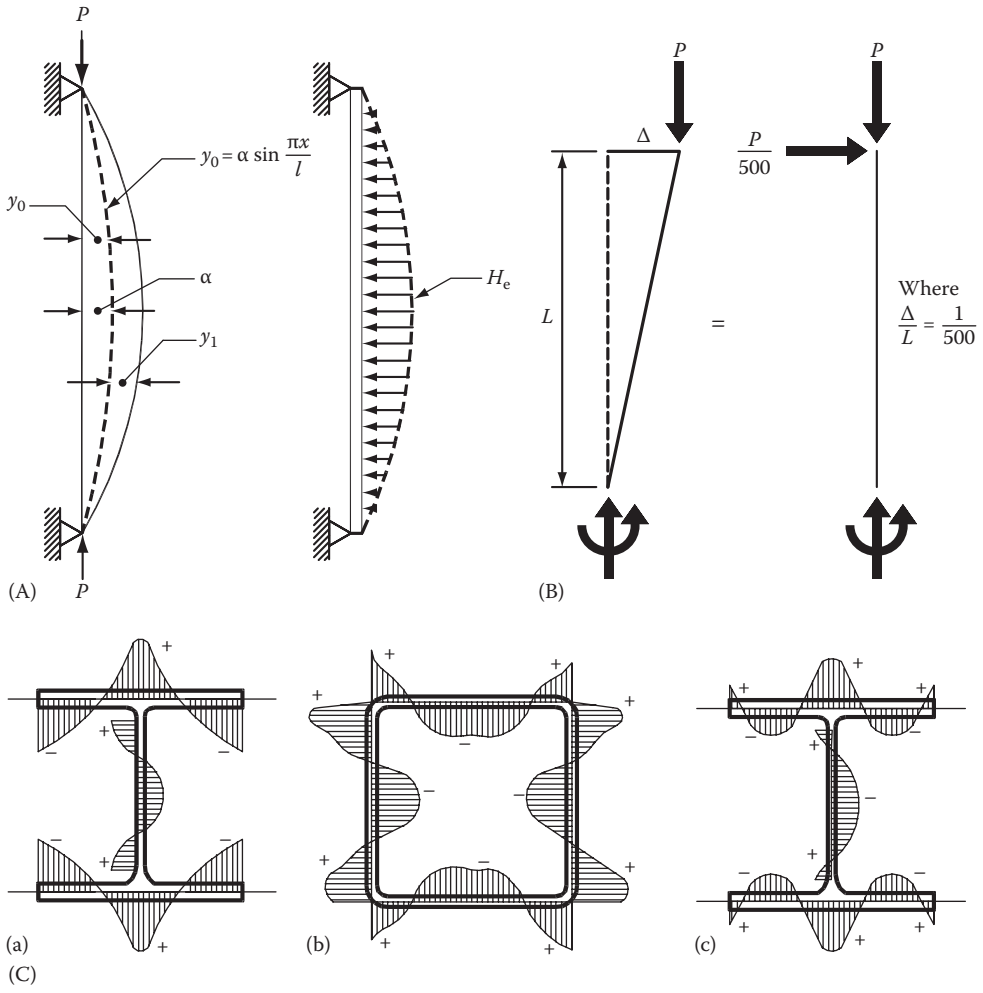


FIGURE 3.29 (A) Concept of notional load: (a) column with initial curvature; (b) equivalent lateral load H_e. (B) Equivalent loading using notional loads to represent the effect of geometric imperfections of a column. (C) Residual Stresses: (a) hot-rolled shapes; (b) welded box sections; (c) I-shape fabricated from flame-cut plates. *Note*: + indicates tension; − indicates compression.

When loads are applied to structural members, the presence of residual stresses usually causes some premature inelastic action; that is, yielding occurs in localized portions before the nominal stress reaches the yield point. Because of the ductility of steel, the effect on strength of tension members is not usually significant, but excessive tensile residual stresses, in combination with other conditions, can cause fracture. In compression members, residual stresses decrease the buckling load from that for an ideal member without residual stresses.

In bending members that have residual stresses, a small inelastic deflection of insignificant magnitude may occur with the first application of load. However, under subsequent loads of the same magnitude, the behavior is elastic. Furthermore, in "compact" bending members, the presence of residual stresses has no effect on the ultimate moment. Consequently, in the flexural design of statically loaded members, it is not usually necessary to consider residual stresses.

3.7.6 Notional Load

When a column is submitted to the action of horizontal load only, a small initial curvature of the column has no effect on the bending and final deflection curve is obtained by superposing the

deflections due to initial curvature on the deflections calculated as for a straight column. However, if there is an axial force acting on the column, the deflections produced by this force will be substantially influenced by the initial curvature in the column.

Let us consider, as an example, the case in which the initial shape of the axis of the column is given by the following equation (see Figure 3.29A):

$$y_0 = a \sin \frac{\pi x}{l}$$

Thus, the axis of the column has initially the form of a sine curve with a maximum ordinate at the middle equal to a. If this column is submitted to the action of a vertical compressive force P, additional deflections y_1 will be produced so that the final ordinates of the deflection curve are

$$y = y_0 + y_1$$

and the bending moment at any cross section is

$$M = P(y_0 + y_1)$$

Using the differential equative approach, it can be shown that the deflections y_1 due to the axial load is given by

$$y_1 = \frac{\alpha}{1-\alpha} \alpha \sin \frac{\pi x}{l}$$

in which the notation α is the ratio of the vertical load to its critical value, that is,

$$\frac{1}{\alpha} = \frac{\pi^2}{k^2 \ell^2}$$

It is seen that, owing to the action of the vertical compressive force, the initial deflection α at the middle of the column is magnified in the ratio $1/(1 - \alpha)$. The final ordinates increase with α and accrue indefinitely as α approaches unity, that is, when the compressive force approaches its critical value.

The problem of bending of an initially curved column can be approached in a different way by replacing the effect of an initial curvature on deflections with the effect of an *equivalent lateral load*. An equivalent lateral load, then, will be one that will produce the same bending-moment diagram as the vertical forces produce if initial deflections only are taken.

This concept of replacing the effect of initial curvature by an equivalent lateral load is used in the stability analysis as shown in Figure 3.29B. In here, the initial our-of-plumbness of column, equal to 1/500 of the floor-to-floor height of the column, is used for determining the equivalent notional load.

3.7.7 Geometric Imperfections

Geometric imperfections inherent in all structures, for the most part as frame out-of-plumbness is modeled directly in the Direct Analysis Method using notional loads acting laterally at each floor.

A notional load is an equivalent lateral load of appropriate magnitude such that it will generate a story shear in the structural model equivalent to the effect of the axial loads in a story acting in the deformed position, as illustrated in Figure 3.29A. Typically, the permissible tolerance on out-of-plumbness of any individual column of length L, is no larger than $L/500$, and the notional load is specified to generate a story shear corresponding to this value of out-of-plumbness. A horizontal notional load of 0.002 times the story gravity load in the horizontal direction is applied, with the 0.002 coefficient being equal to 1/500 (see Figure 3.29B).

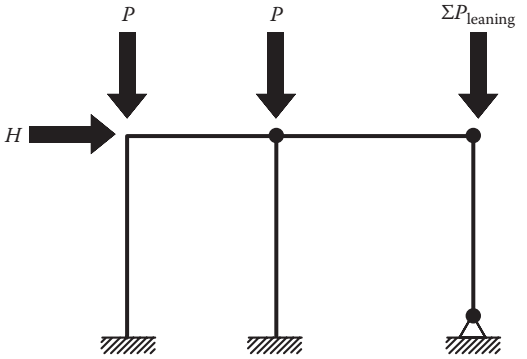


FIGURE 3.30 Frame model to illustrate effects of leaning column.

3.7.8 Leaning Columns

In any stability analysis, it is necessary to capture the destabilizing effects of columns that rely on the lateral frame for stability but are not a part of the lateral frame. These columns are commonly referred to as "leaning columns." When modeling the frame, leaning-column effects can be captured either by developing a complete 3D model of the frame or by assigning a single equivalent leaning column carrying the summation of all of the gravity loads on all of the leaning columns in the structure, as a pin-connected part of a 2D frame. An example of how this might be modeled in a 2D analysis is shown in Figure 3.30.

3.8 AISC 360-10 STABILITY PROVISIONS

The AISC 360-10 provisions regarding the stability analysis recognizes the wide range of analysis in common use. It gives designers the freedom to select their own methods of analysis and design. An advanced rigorous method called the Direct Analysis Method that was regarded as a research tool only a few years ago now appears in Chapter C of the Specifications. Additionally in Appendix 7, Alternative Methods of Design for Stability are presented immediately followed by Approximate Second-order Analysis in Appendix 8.

At first glance, the stability provisions in Chapter C, Appendices 7 and 8, and their corresponding commentaries, are often enough to make the novis designer run for cover and the veteran designer yearn for olden days when the design provisions were much simpler. The concern is perhaps, Quite Justified. It is the author's hope, however, that the brief summary presented here without the esoteric jargon would be of help in removing such fears.

The Direct Analysis Method is really simple, consisting of just four steps:

- A second-order analysis
- Use of reduced stiffness in the analysis
- Application of a "notional load" in the analysis under certain conditions
- Strength check of members using $K = 1$ for compressive strength

3.8.1 Second-Order Analysis

Second-order analysis must consider both $P\Delta$ effects (the effects of loads acting on the displaced locations of joints or nodes in the structure) and $P\delta$ effects (the effects of loads acting on the deformed shapes of individual members).

Nowadays, almost all computer programs that have the ability to do second-order analysis handle $P\Delta$ effects adequately, but some do not consider $P\delta$ effects. However, it should be noted that for many real-world buildings, it is acceptable to use a program that neglects the effect of $P\delta$ on the

overall response of the structure: If the ratio of second-order drift to first-order drift is less than 1.5, and no more than one-third of the total gravity load on the building is on columns that are moment-connected in the direction of translation being considered, the error in a $P\Delta$-only analysis will be negligible. It is necessary, however, in all cases to consider the effect of $P\delta$ on individual compression members.

Instead of performing a rigorous second-order analysis, the designer still has the option of using the familiar "B_1 and B_2" procedure, in which the results of a first-order analysis are amplified by factors B_1 and B_2 to account for second-order effects. Factor B_1, accounts for $P\delta$ effects; B_2, calculated for each story of the building and each direction of lateral translation of the story, accounts for $P - \Delta$ effects. A first-order analysis amplified by B_1 and B_2 is in essence a second-order analysis.

3.8.2 Reduced Stiffness in the Analysis

A factor of 0.8 is applied across the board to the stiffnesses of all the members in the analysis. An additional factor, τ_b, is applied to the flexural rigidity of framed columns in which the axial force is greater that half the yield force.

In lieu of applying the additional reduction factor τ_b, the designer may apply at each floor an additional lateral load of 0.001 times the vertical load applied on that floor.

3.8.3 Application of Notional Loads

A "notional load" (which is an additional lateral load intended to simulate the effects of initial our-of-plumbness of the building) must be applied under certain conditions. When required, the magnitude of the notional load on each floor is 0.002 times the vertical load applied on that floor.

If the ratio of second-order drift to first-order drift is greater than 1.5 notional loads must be included in all load combinations. If the ratio is less than 1.5, notional loads need to be applied only in gravity-only load combinations; they need not be applied in combination with other lateral loads.

3.8.4 Member Strength Checks

When the forces and moments in members have been determined from the analysis outlined above, member capacities can be checked using an effective length factor, K, of unity for members subject to compression.

The only significant difference between the Direct Analysis Method and the Effective Length Method is that the former uses reduced stiffness in the analysis and $K = 1$ in the member strength check, the latter uses nominal stiffness in the analysis and K is determined from a sideways buckling analysis in the strength check of columns in moment frames.

The First-Order Analysis Method uses mathematical manipulation to achieve approximately the same results as the Direct Analysis Method. In this method, and additional lateral load is applied in a first-order analysis to simulate the effects of a B_2 multiplier, a stiffness reduction, and a notional load; B_1 multipliers are then applied explicitly to all beam columns.

The Direct Analysis Method is the primary method of design for stability; it is presented in Chapter C of AISC 360-10 while the Effective Length Method and the First-Order Analysis Method are in Appendix. The "B_1 and B_2" procedure for approximate second-order analysis is in a separate appendix, to emphasize that the $B_1 - B_2$ procedure is simply an analysis technique, applicable wherever a second-order analysis is required.

It should be noted that permissible variations from theoretical dimensions of an erected structure are specified in the AISC "Code of Standard Practice for Steel Buildings and Bridges." It states that variations are within the limits of good practice or erected tolerance when they do not exceed the cumulative effect of permissible rolling and fabricating and erection tolerances. These tolerances are restricted in certain instances to total cumulative maximums.

The AISC "Code of Standard Practice" has a descriptive commentary that explains the application of the mill, fabrication and erection tolerances for a building or bridge. Also specified therein are the requirements for special or more restrictive tolerances for a particular type of structure.

An example of tolerances that govern the plumbness of a multistory building is the tolerance for columns. In multistory buildings, columns are considered to be plumb if the error does not exceed 1:500, except for columns adjacent to elevator shafts and exterior columns, for which additional limits are imposed.

Measurement of tolerances requires experience. Structural steel is not static but moves due to varying ambient conditions and changing loads imposed during the construction process. Ambient conditions can be so extreme as to require final plumbing and span closing during nighttime hours. Making all components and attachments fit takes skill and experience on the part of designers and craftsmen.

3.8.5 Step-by-Step Procedure for Direct Analysis Method

1. Create a model of the lateral frame being analyzed, including the leaning columns.
2. Reduce the stiffness of lateral framing members in your model as follows:
   (a) For a braced frame modeled with pinned connections, this can be done by modifying the modulus of elasticity in your model to 0.8 times E(23,200 ksi).
   (b) For a moment frame (or a braced frame modeled with rigidly connected members), this can be done by modifying the modulus of elasticity in your model to $0.8\tau_b E$.

   As an alternative to applying τ_b, add a notional load of 0.001 times the gravity load to the notional load in step 3. τ_b may b taken as 1 if $\alpha P_r < P_y$. This may, however, require iteration in the analysis if the member sizes change and the inequality reverses. Moment frames will often satisfy this inequality, allowing τ_b to be taken as 1.
3. Apply notional loads to all load combinations equal to 0.002 times the gravity load on each story at the story level. The notional loads only need to be added to load combinations in which the notional load is larger than the lateral load on the frame. Thus, notional loads usually can be ignored in all but the gravity only load combinations. However, if a designer wishes to simplify the design process, it is always conservative to include the notional loads.
4. Conduct a second-order analysis of the structure under applied loads, either by the Amplified First-Order Analysis approach or by completing a direct, rigorous second-order analysis of the structure.
5. Reset your modulus of elasticity to E = 29,000 ksi and design the members using the equations in the AISC Specification to resist the forces that you have just determined, with K = 1.
6. Check drift limits for wind and seismic design.

3.9 UNDERSTANDING HOW COMMERCIAL SOFTWARE WORKS

There are numerous items the design engineers must be aware of when selecting or using structural analysis software for stability analysis. Communication between design engineers and software providers is the only way the full capacity and limitations of the programs can be assessed.

A discussion of pros and cons of some common software available in the market is beyond the scope of this work. However, we will touch upon certain important items design engineers must be aware of when using structural analysis software to carry out stability analysis and design.

AISC *Specification* states that any second-order elastic analysis method that captures the effects of $P\Delta$ and $P\delta$ may be used. This statement is backed by providing in Appendix 7 of AISC 360-10 an approximate method that can be used based upon first-order analysis forces amplified by B_2 and B_1 to satisfy the requirement.

Most programs use either an iterative approach or a geometric-stiffness-based method. In both approaches, it is common that small deformations are assumed.

Iterative methods perform repeated linear-elastic evaluations as the structure deforms laterally to capture increased moments and forces imposed on the structure. The iterations continue until convergence is reached—until the deformations stop increasing. If convergence does not occur, the structure is said to be unstable and needs to be stiffened. Some commercially available programs make use of such an approach. Nodal deformations are first used to generate forces, which are then iterated until convergence is reached.

The *geometric stiffness* methods change the stiffness of the structure by altering the stiffness matrix to simulate the effects of $P\Delta$. When using this approach, iteration is not required and the resulting analysis can be less computationally demanding.

$P\delta$ effects are caused by deformations (curvature) of individual members, but they also can affect the overall force amplification when lateral loads are introduced. Therefore, AISC Specification requires that they be considered. In many cases, however, $P\delta$ effects are small and may be neglected.

It should be noted that software developers have proposed various methods to incorporate $P\delta$ effects in second-order analysis by adding nodes between support points of members.

To take into account all relevant deformations, most commonly used programs, at a minimum, account for flexural deformations because they contribute significantly to overall drift in moment frames. However, it is important that the contribution of other deformations be considered when they are significant, including axial and shear deformations. While most programs account for these deformations, other effects like panel zone deformations are typically not accounted for. One method of taking this into account is to model beams from column centerline-to-centerline—rather than face-to-face.

The Direct Analysis Method is a powerful approach that can be used to capture the effects of residual stresses and initial imperfections. The direct modeling approach for initial imperfections also can be used to account for temperature gradients and foundation settlements. In cases of second-order effects such that $\Delta_{2nd-order}/\Delta_{1st-order} > 1.5$ the Direct Analysis Method must be used.

Two deceptively simple tools in this method—notional lateral loads and stiffness reductions—account for a host of destabilizing influences on the structure. Notional loads must be applied to all gravity-only load combinations. Where $\Delta_{2nd-order}/\Delta_{1st-order} > 1.5$, the notional load must be applied in all load combinations. These notional loads account for the effects of structure out-of-plumbness and are derived from the tolerances provided in the AISC *Code of Standard Practice*. However, the *Specification* also permits the use of notional loads that are smaller if a lesser out of plumbness is known to exist or can be ensured. Direct modeling of the actual geometric imperfections is also permitted.

Some programs allow the user to input notional loads, while certain others automatically compute and add them to other lateral loads based on the gravity loads in the members. Automatic modeling of geometric imperfections is not provided by most structural analysis software developers at this time (2011).

When calculating second-order effects a 20% reduction in axial and flexural stiffness must be made to all members that contribute to stabilizing the structure. A larger reduction applies to columns with high axial load by using an additional multiplier τ_b. These stiffness reductions must not be confused with the reductions that are made to the stiffness matrix in the second-order analysis to modify the geometric stiffness methods. One is mandated by the AISC *Specification* whereas the other is at the behest of the software developer; and the purposes of the two are different albeit related.

Inputting stiffness reduction is not difficult because it can simply be applied to the modulus of elasticity (E). However, the reductions are only required for purposes of generating design forces and drifts—amplified by second-order effects and notional loads—and not for strength checks, or evaluation of the system for serviceability.

This may require that programs differentiate between members that are part of the lateral system and those that are not. The program may then use two different sets of stiffness for those that are part of the lateral system; one for analysis and another for strength checks. The entire process can however, be automated including the calculation of τ_b for each member.

4 Gravity Systems for Composite Buildings

PREVIEW

Gravity composite systems may be broadly classified into composite floors and composite columns. Composite floor systems typically consist of simply supported steel rolled sections, and occasionally nonprismatic steel sections such as tapered, dapped, or castellated beams including haunch girders, trusses, or stub girders. Invariably, in all cases the steel member is attached to the concrete topping via shear connectors welded to the member through a formed composite metal deck. The entire floor system consisting of steel sections, metal deck, and concrete topping responds as a series of flexural T-beams under gravity loads. Shown in Figure 4.1 are the typical components of composite floor used nearly exclusively in steel-framed buildings in North America.

Composite construction is not new. It is at least 30 years older than the oldest person reading this book. It is characterized by interactive behavior between structural steel and concrete components designed to take advantage of the best load-resisting characteristics and economy of each material.

We begin this chapter with a discussion of composite slabs that use profiled steel deck as permanent form. In this system, the deck itself acts as the positive reinforcement of the composite slab. Next, we discuss the most common composite floor system consisting of rolled or built-up steel beams connected to formed steel deck and concrete slab. The steel section and the concrete slab are most often mechanically connected to each other using headed steel studs to the almost complete exclusion of other types of shear connectors. Also discussed in this section are other composite floor systems such as tapered and dapped girders, joists, trusses, and stub girders that attempt to use structural steel more efficiently.

The next section centers on the design of two basic types of composite columns: those with steel sections encased in concrete and those with the steel sections filled with concrete, the latter most often consisting of hollow sections fabricated from plates. In the first type, often referred to as encased composite column, the steel section can be designed to provide support for the erection of several floors ahead of concreting the steel column for composite action. The procedure expedites construction schedule and has potential for reducing overall construction cost.

The design of composite beams particularly for partial composite action is not an easy task, nor is the design of encased or filled composite columns. Unless design aids or a computer with a suitable design program is used, it is extremely cost prohibitive to perform manual calculations in a design office. Therefore, instead of presenting manual design examples in excruciating details, in this chapter, as in other chapters of this work, we present abbreviated versions emphasizing basic concepts, fundamental structural actions, and practical applications.

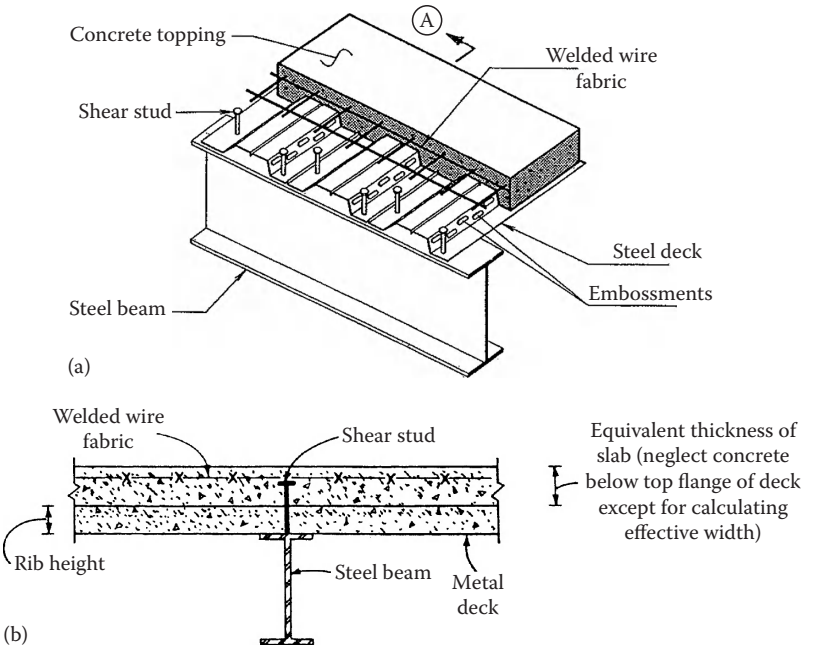


FIGURE 4.1 Components of composite floor system: (a) schematics showing metal deck perpendicular to beam; (b) section.

4.1 COMPOSITE METAL DECK

Metal deck is manufactured from steel sheets by a fully mechanized, high-speed cold rolling process. Although it is possible to produce shapes up to ½ in. and even in ¾ in. thick by cold forming, cold-formed steel construction is generally restricted to plates and sheets weighing from 0.5 psf to a maximum of 0.9 psf.

Composite metal deck is manufactured with embossments formed in the steel sheet specifically designed to increase composite action between the metal deck and concrete. The shear connection between the two is provided through lugs, corrugations, ridges, or embossments formed in the profile of the sheet to increase the chemical bond between the two materials. The steel deck profile is typically trapezoidal with relatively wide flutes suitable for through-deck welding of shear studs. Metal deck may also include closed cells to accommodate floor electrification system. Noncellular deck panels may be blended with cellular panels as part of the total floor system. Metal deck is commonly available in 1½, 2, and 3 in. depths with rib spacings of 6, 7½, 8, 9, and 12 in.; See Figure 4.2 for some typical deck profiles.

A composite slab is usually designed as a simply supported reinforced concrete slab with steel deck acting as positive reinforcement. Typical mesh used for control of temperature cracking does not provide enough negative reinforcement for typical beam spacing of 8–15 ft. Although the slab

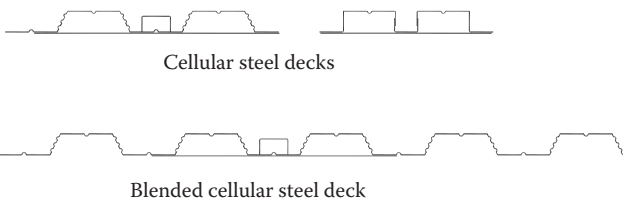


FIGURE 4.2 Typical composite metal deck profiles.

is designed as a simple span, it is a good practice to provide a nominal reinforcement of say #4 @ 24 in. c-c at the top of beams to control excessive cracking of slab. It is generally believed that cracking of the slab in the negative regions does not materially impair the composite beam strength.

The Steel Deck Institute (SDI), regarded as the industry standard by the metal deck manufacturers, has published a manual which encompasses the design of composite decks, form decks, and roof decks. A brief description of the SDI specifications is given in the following section.

4.1.1 SDI Specifications

The steel used for the fabrication of composite metal deck shall have a minimum yield point of 33 ksi. The specified yield point is the primary criterion for strength under static loading. The tensile strength is of secondary importance because fatigue strength and brittle fracture, which relate to tensile strength rather than yield point, are rarely of consequence; metal deck is rarely subjected to repetitive loads and the characteristic thinness invariably precludes the development of brittle fracture.

Considerable variations in thickness of metal deck may occur because of rolling tolerances. Therefore, SDI stipulates that the delivered thickness of bare steel without specified finishes such as phosphotizing and galvanizing shall not be less than 95% of the specified thickness. The increase in the stiffness of deck due to galvanizing is not relied upon in design of metal deck.

Opinions differ among engineers whether the metal deck used inside of a building which is not directly exposed to weather needs galvanizing or not. SDI does not mandate any particular type of finish. The appropriate finish is left to the discretion of the engineer with the recommendation that due consideration be given to the effects of environment to which the structure is subjected. However, SDI in its commentary recommends a galvanized coating conforming to ASTM A-525 G.60 requiring a minimum galvanizing of 0.75 ounce per square feet of metal deck. Other salient features of SDI specifications are as follows:

1. Minimum compressive strength of concrete f'_c shall be 3.0 ksi. The compressive stress in concrete is limited to 0.4 f'_c under the applied load for unshored construction and under the total dead and live loads for shored construction. The flexural or shear bond is to be based on ultimate strength analysis with a minimum safety factor of 2. The minimum temperature and shrinkage reinforcement in a composite slab is a function of the area of concrete, as in typical reinforced concrete slabs, but only the area above the metal deck need be considered in calculating the area of concrete.
2. The use of admixtures containing chloride salts is prohibited because salts can erode the steel deck.
3. When designing the metal deck section as a form, the section properties for flexural design are to be calculated as per AISC *Specification for the Design of Cold-formed Steel Structural Members.*
4. Bending stress is limited to 0.6 times the yield strength of steel. An upper limit of 36.0 ksi is imposed on the allowable stress. In addition to the weight of wet concrete and deck, allowance should be made for construction live loads of 20 psf of uniform load or a 150 lb concentrated load. This is to account for the weight of one person working on a 1 ft (305 mm) width of deck. It is a common practice to allow for a 200 lb point load as the equivalent load. This is because the loading is considered temporary with a 33% increase in the stress, which is equivalent to reducing the 200 lb load by 25%. Clear spans are to be used in the moment calculations.
5. For calculating deflections, it is not necessary to consider the construction loads since the deck, which is designed to remain elastic, will rebound after the removal of construction loads. The calculated deflection based on the weight of concrete is limited to the smaller value $L/180$ or ¾ in., in which L is the clear span of the deck. Deflections of composite slabs due to live loads of 50–80 psf are seldom a design concern because the deflections

are usually less than $L/360$, where L is the span of deck. Because the slab is assumed to have cracked at the supports, the deflections are best predicted by using the average of the cracked and uncracked moment of inertia using transformed sections. Note that when slabs are cast level to compensate for the deflection of metal deck, a 10%–15% of additional concrete is required for slab construction.

6. A minimum bearing of 1½ in. is specified for proper deck seating on supports.
7. A maximum average spacing of 12 in. for puddle welds is specified to obtain proper anchorage to supporting members. The maximum spacing between the adjacent welds is limited to 18 in. Welding of decks with thickness less than 0.028 in. is not practical because of the likelihood of burning off the sheet. Therefore, SDI stipulates use of welding washers for floor decks of less than 0.028 in. in thickness. Stud welding through the metal deck to the steel top flange can be used instead of puddle welds to satisfy the minimum spacing requirements. However, since it is possible to get uplift forces during wind storms, puddle welds should be used to prevent metal decks from blowing off buildings during construction.
8. Mechanical fasteners which satisfy the anchorage criteria can be used in lieu of puddle welds.
9. Side laps with proper fasteners are required between two longitudinal pieces of deck to (1) prevent differential deflection; (2) provide sufficient diaphragm strength; and (3) sustain local construction loads without distortion or separation. The edges of metal deck shall be connected with ¾ in. diameter fusion welds at a maximum spacing of 3 ft throughout for simply supported spans. Button punching at 2 ft on centers is an acceptable alternative to minimum fusion welding.

To function as form work, decks supporting cantilevers should be proportioned to satisfy the following criteria: (1) dead load deflection should be limited to $L/90$ of overhang or 3/8 in., whichever is smaller; (2) steel stress should be limited to 26.7 ksi for overhang, or steel stresses limited to 20.0 ksi for dead load plus 20 psf of additional load, whichever is more severe; (3) the deck should receive one fusion weld at the cantilever end, and the spacing of welds throughout the cantilever span should not exceed 12 in. Button punching can be used as an acceptable alternative to fusion welding.

4.2 COMPOSITE BEAMS

Three types of composite beam construction, shown in Figure 4.3, are recognized in the AISC specifications: (1) fully encased steel beams; (2) concrete filled HSS; (3) steel beams with mechanical anchorage to slab. In fully encased steel beams, the natural bond between concrete and steel interface is considered sufficient to provide the resistance to horizontal shear provided that: (1) the concrete thickness is 2 in. or more on the beam sides and soffit with the top of the beam at least 1½ in. below the top and 2 in. above the bottom of the slab; (2) the encasement is cast integrally with the slab and has adequate mesh or other reinforcing steel throughout the depth and across the soffit of the beam to prevent spalling of concrete.

The third type consisting of steel beams, metal deck, concrete topping, and shear connectors is by far the more popular in the construction of buildings in North America. Invariably composite action is achieved by providing shear connectors between top flange of the steel beam and concrete topping. Fully encased steel beam construction is not used because encasing of beams with concrete requires expensive form work.

Composite sections have greater stiffness than the summation of the individual stiffness of slab and beam and, therefore, can carry larger loads or similar loads with appreciably smaller deflection and are less prone to transient vibrations. Composite action results in an overall reduction of

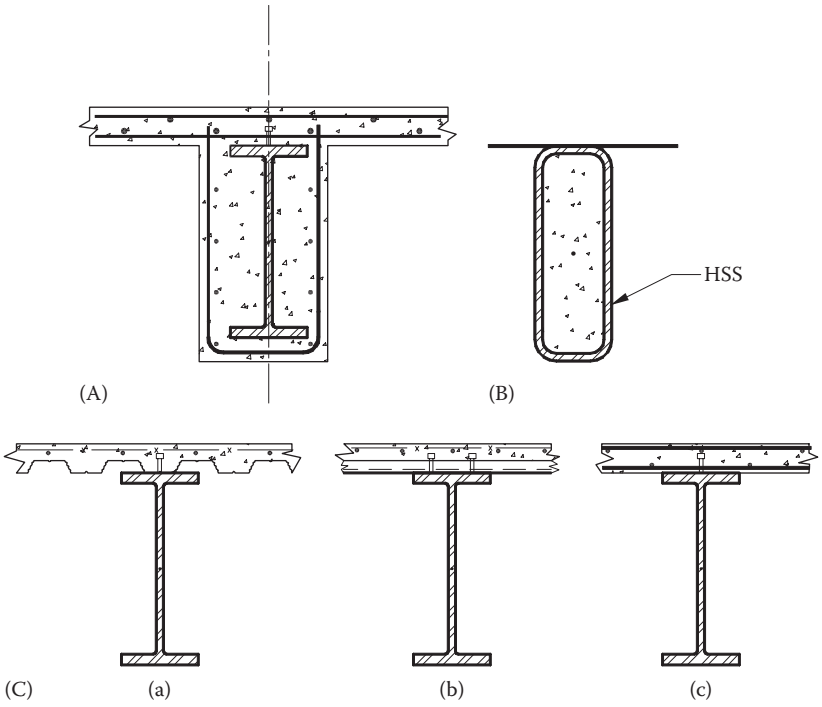


FIGURE 4.3 Three types of composite beams addressed in the AISC manual: (A) fully encased steel beam; (B) concrete filled HSS; (C) steel beams with mechanical anchorage to slab. (a) Metal deck parallel to beam; (b) metal deck perpendicular to beam; (c) cast-in-place slab without metal deck.

floor depth, and for high-rise buildings, the cumulative savings in curtain walls, electrical wiring, mechanical ductwork, interior walls, plumbing, etc., can be considerable.

Composite beams can be designed either for shored or unshored construction. For shored construction, the cost of shoring should be evaluated in relation to the savings achieved by the use of lighter beams. For unshored construction, steel is designed to support by itself, the wet weight of concrete and construction loads. The steel section, therefore, is heavier than in shored construction. In composite floor construction, because top flange of steel beam is attached to concrete by the use of shear connectors, the slab becomes part of the compression flange. As a result, the neutral axis of the section shifts upward, making the bottom flange of the beam more effective in tension. Since concrete is required to serve as floor system, the only additional cost is that of the shear connectors. In addition to transmitting horizontal shear forces from the slab into beam, the shear connector prevents any tendency for the slab to rotate independently of the beam.

The stud shear connector is a short length of round steel bar welded to the steel beam at one end and having an anchorage provided in the form of a round head at the other end. The most common diameters are 1/2, 5/8, 3/4 in. The length is dependent on the depth of metal deck and should extend at least 1½ in. above the top of the deck. The welding process typically reduces their length by about 3/16 in. The upset head thickness of the studs is usually 3/8 or 1/2 in., and the diameter 1/2 in. larger than the stud diameter. The studs are normally welded to the beam with an automatic welding gun, and when properly executed, the welds are stronger than the steel studs. Studs located on the side of the trough toward the beam support are more effective than studs located toward the beam centerline. The larger volume of concrete between the stud and the pushing-side of the trough helps in the development of a larger failure cone in concrete, thus increasing its horizontal shear resistance.

The length of stud has a definite effect on the shear resisted by it. As the length increases, so does the size of the shear cone, with a consequent increase in the shear value. The shear capacity of the

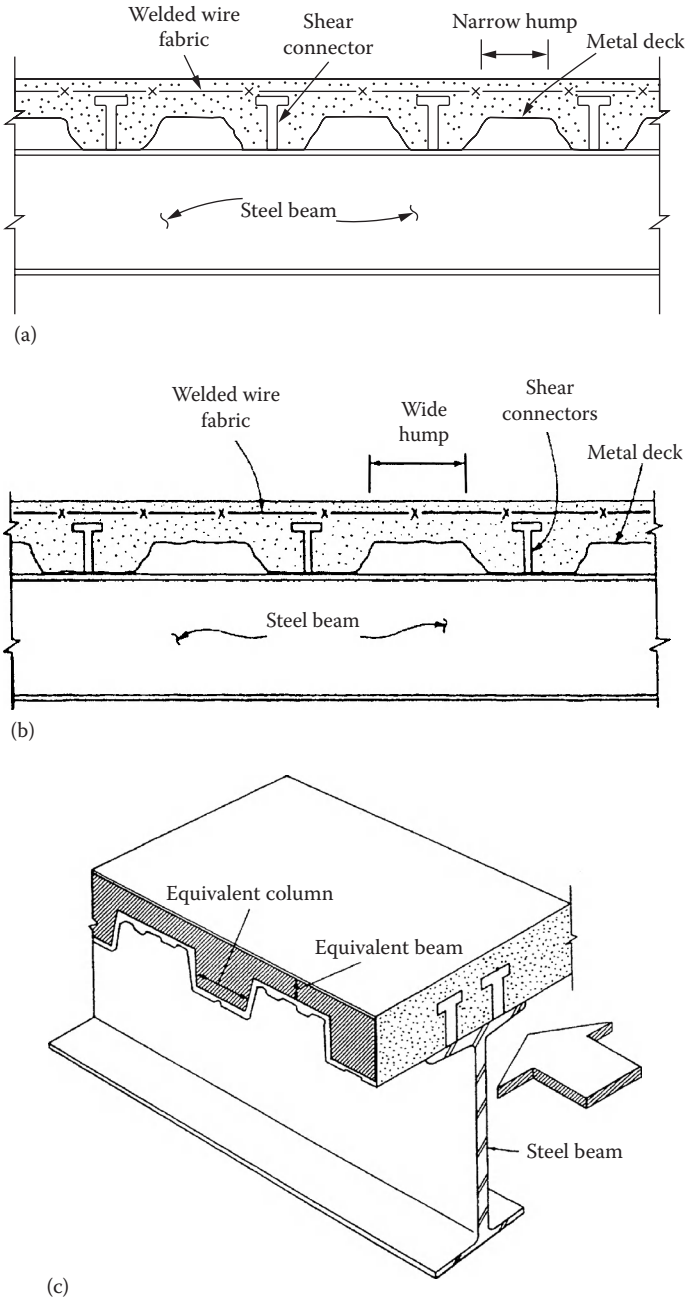


FIGURE 4.4 Relative shear strength of shear connectors: (a) narrow rib deck; (b) wide rib deck; (c) equivalent portal frame.

stud also depends on the profile of the metal deck. To get a qualitative idea, consider the two types of metal decks shown in Figure 4.4a and b. The deck in Figure 4.4a has a wide hump compared to the one in Figure 4.4b. When subjected to a horizontal load *V*, the concrete over the metal deck tends to behave as a portal frame. The concrete in the troughs can be thought of as columns with concrete in the humps acting as beams (see Figure 4.4c). A narrow hump of the portal frame, results in an equivalent beam of smaller span when compared to the one with a wider hump, meaning that a deck profile with the widest trough and narrowest hump will yield the highest connector strengths.

However, other considerations such as volume of concrete, section modulus, and the stiffness of deck also influence the shear strength of the connector.

Metal decks for composite construction are available in the United States in three depths, 1½, 2, and 3 in. The earlier types of metal deck did not have embossments, and the interlocking between concrete and metal deck was achieved by welding reinforcement transverse to the beam. Later developments of metal deck introduced embossments to engage concrete and metal deck and dispensed with the transverse-welded reinforcement,

The spans utilizing composite metal deck are generally in the range of 8–15 ft.

In floor systems using 1½ in. decks, the electrical and telephone services are generally provided by the so-called poke-through system, which is simply punching through the slab at given locations for passage of under-floor ducts. A deeper deck is required if the power distribution system is integrated as part of the structural slab; as a result, 2 and 3 in. metal decks were developed. Experiments have shown that there is very little loss of composite beam stiffness due to the ribbed configuration of the metal deck in the depth range of 1½–3 in. As long as the ratio of width to depth of the metal deck is at least 1.75, the entire capacity of the shear stud can be developed similar to beams with solid slabs. However, with deeper deck a substantial decrease in shear strength of the stud occurs, which is attributed to a different type of failure mechanism. Instead of the failure of shear stud, the mode of failure is initiated by cracking of the concrete in the rib corners. Eventual failure takes place by separation of concrete from the metal deck. When more than one stud is used in a metal deck flute, a failure cone can develop over the shear stud group, resulting in lesser shear capacity per each stud. The shear stud strength is therefore closely related to the metal deck configuration and factors related to the surface area of the shear cone.

Often special details are required in composite design to achieve optimum results. Openings interrupt slab continuity, affecting capacity of the composite beam. For example, beams adjacent to elevator and stair openings may have full effective width for part of their length and perhaps half that value adjacent to the openings. Elevator sill details normally require a recess in the slab for door installations, rendering the slab ineffective for part of the beam length. A similar problem occurs in the case of trench header ducts, which require elimination of concrete, as opposed to the standard header duct, which is completely encased in concrete. When the trench is parallel to the composite beam, its effect can be incorporated in the design by suitably modifying the effective width of compression flange. The effect of the trench oriented perpendicular to the composite beam could range from negligible to severe depending upon its location. If the trench can be located in the region of minimum bending moment, such as near the supports in a simply supported beam, and if the required number of connectors could be placed between the trench and the point of maximum bending moment, its effect on the composite beam design is minimum. If, on the other hand, the trench must be placed in an area of high bending moment, its effect may be so severe as to require that the beam be designed as a non-composite beam.

The slab thickness normally employed in high-rise construction with composite metal deck is usually governed by fire-rating requirements rather than the thickness required by the bending capacity of the slab. In certain parts of the United States, it may be economical to use the minimum thickness required for strength and to use sprayed-on or some other methods of fireproofing to obtain the required ratings. Some major projects have used 2½ in. thick concrete on 3 in. deep metal deck spanning as much as 15 ft. A comparative study which takes into consideration the vibration characteristics of the floor is necessary to zero in on the most economical floor system.

In continuous composite beams the negative moment regions can be designed such that (1) the steel beam alone resists the negative moment or (2) it acts compositely with mild steel reinforcement placed in the slab parallel to the beam. In the latter case, shear connectors must be provided through the negative moment region.

Careful attention should be paid to the deflection characteristics of composite construction because the slender not-yet-composite shape deflects as wet concrete is placed on it. There are three ways to handle the deflection problem.

1. Use relatively heavy steel beams to limit the dead-load deflection and pour lens-shaped tapering slabs to obtain a nearly flat top. Although a reasonably flat surface results from this construction, the economic restraints of speculative office buildings do not usually permit the luxury of the added cost of additional concrete and heavier beams.
2. Camber the steel beam to compensate for the weight of steel beam and concrete. Place a constant thickness of slab by finishing the concrete to screeds set from the cambered steel. Continuous lateral bracing as provided by the metal deck is required to prevent the lateral torsion buckling of beam. If steel deck is not used, this system requires a substantial temporary bracing system to stabilize the beam during construction.
3. Camber and shore the steel beam. The beam is fabricated with a camber calculated to compensate for the deflection of the final cured composite section. Shores are placed to hold the steel at its curved position while the concrete is being poured. As in method 2, slab is finished to screeds set from cambered steel. Although methods 1 and 3 are occasionally used, the trend is to use method 2 because it is the least expensive.

4.2.1 AISC Design Criteria: Composite Beams with Metal Deck and Concrete Topping

For purposes of design, it is useful to consider two types of composite beams.

1. Deck perpendicular to beam (Figure 4.5)
   a. Concrete below the top of steel decking shall be neglected in computations of section properties and in calculating the number of shear studs, but the concrete below the top flange of deck may be included for calculating the effective width.
   b. The maximum spacing of shear connectors shall not exceed 32 in. along the beam length.

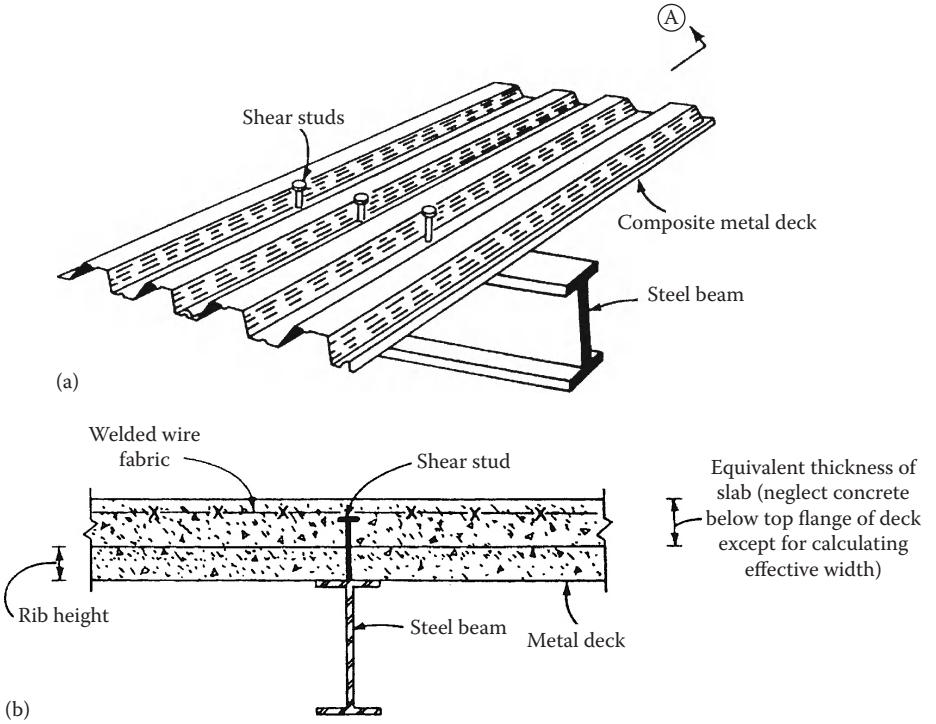


FIGURE 4.5 Composite beam with deck ribs perpendicular to beam: (a) schematic view; (b) section showing equivalent thickness of slab.

c. The steel deck shall be anchored to the beam either by welding or by other means at a spacing not exceeding 16 in.
d. Shear stud reduction factors, R_g and R_p as described presently, should be used for reducing the shear capacity of stud connectors.
e. The recommended sequence for installing studs when deck is perpendicular to beams is as follows:
   i. Deck ribs at 6 in. on center. Start at beam ends and place a single stud at every fourth flute, working toward the center of beam. If studs remain, fill in empty ribs, again starting at beam ends and working toward the center without exceeding 30 in. for stud spacing.
   ii. Deck ribs at 12 in. on center. Start at beam ends and place a single stud in every other flute working toward the center of beam. If studs remain, fill in empty ribs, again starting at beam ends and working toward the center of beam without exceeding 24 in. (610 mm) for study spacing.
   iii. If the number of studs is more than the number of ribs, place a double or triple row as needed, always starting from beam ends and working toward the beam center. In general, if studs cannot be uniformly spaced, the greatest number of studs should occur at the ends.

2. Deck ribs parallel to beam (Figure 4.6)
   a. The major difference between the perpendicular and parallel orientation of deck ribs is that when deck is parallel to beam, the concrete below the top of the decking can be included in the calculations of section properties and must be included when calculating the number of shear studs.
   b. If steel deck ribs occur on supporting beam flanges, it is permissible to cut high-hat to form a concrete haunch.
   c. When nominal rib height is 1½ in. or greater, minimum average width of deck flute should not be less than 2 in. for the first stud in the transverse row plus four stud diameters for each additional stud. This gives minimum average widths of 2 in. for one stud, 2 in. plus 4*d* for two studs, 2 in. plus 8*d* for three studs, etc., where *d* is the diameter of stud. Note that if a metal deck cannot accommodate this width requirement, the deck can be split over the girder to form a haunch.

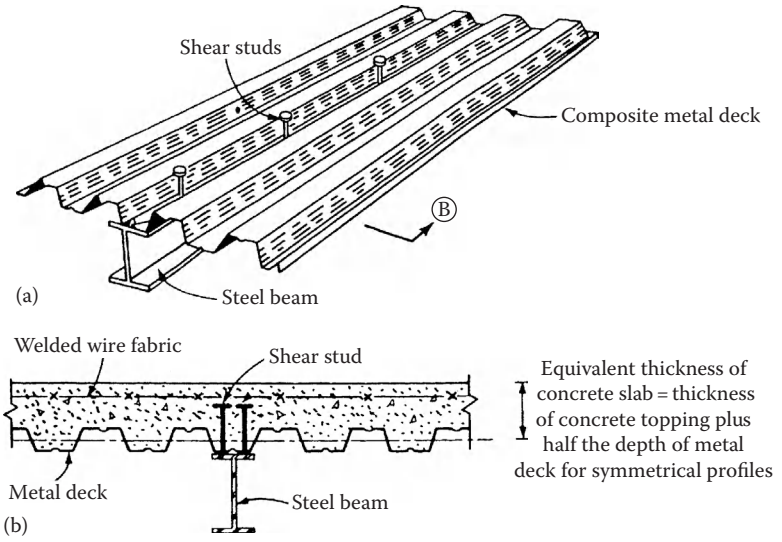


FIGURE 4.6 Composite beam with deck ribs parallel to beam: (a) schematic view; (b) section.

d. Shear stud reduction factors, R_g and R_p, as explained presently, shall be used for reducing the shear capacity of stud connectors.
e. The recommended sequence for installing studs when deck is parallel to the girder is as follows. Start at the girder ends by placing the first stud at approximately 12 in. from the centerline of support and work toward the center of girder with uniform spaces between the studs. If a double row of studs is required, it is a good practice to place them in a staggered pattern rather than side by side.

4.2.1.1 AISC Requirements, General Comments

1. The rib deck height shall not exceed 3 in.
2. Rib average width shall not be less than 2 in. If the deck profile is less than 2 in., this minimum clear width shall be used in the calculation.
3. The section properties do not change a great deal from deck running perpendicular or parallel to the beam, but the change in the required number of studs can be significant.
4. The reduction formula for stud length is based on rib geometry, number of studs per rib, and embedment length of the studs.
5. The provision for calculating the partial section modulus makes selection of heavier, stiffer beams with fewer studs economically more attractive.
6. High shear values can be used in longer shear studs. Concrete cover over the top of the stud is not limited by the AISC specifications, but practical reasons the author recommends a minimum of ½ in.
7. Studs can be placed as close to the web of deck as needed for installation and to maintain the necessary spacing.
8. Deck anchorages can be provided by the stud welds.
9. Maximum diameter of shear connectors is limited to ¾ in.
10. After installation, the studs should extend a minimum of 1½ in. above the steel deck.
11. Total slab thickness including the ribs is used in determining the effective width without regard to the orientation of the deck with respect to the beam axis.
12. The slab thickness above the steel deck shall not be less than 2 in.
13. Shear studs may be welded directly through the deck or through pre-punched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is great than 16 gage for single thickness, or 18 gage for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 1.25 ounces/sq. ft, special precautions and procedures as recommended by the study manufacturer should be followed.
14. The majority of composite steel floor decks used today have a stiffening rib in the middle of each deck flute. Because of the stiffener, studs must be welded off-center in the deck rib.
15. Although it is recommended that studs be detailed in the strong position, ensuring that studs are place in the strong position is not necessarily an easy task because it is not always easy for the installer to determine where along the beam the particular rib is located, relative to the end, midspan or point of zero shear. Therefore, the installer may not be clear on which is the strong position and which is the weak position. However, it is reassuring to note that even a large change in shear strength does not result in a proportional decrease in the flexural strength of composite beams (see Figure 4.7a and b for definition of strong and weak stud position).
16. Uniform spacing of shear connectors is permitted, except in the presence of heavy concentrated loads.
17. Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear-resisting strength. To guard against this contingency, the size of a stud not located over the beam web is limited to 2½ times the flange thickness. The practical

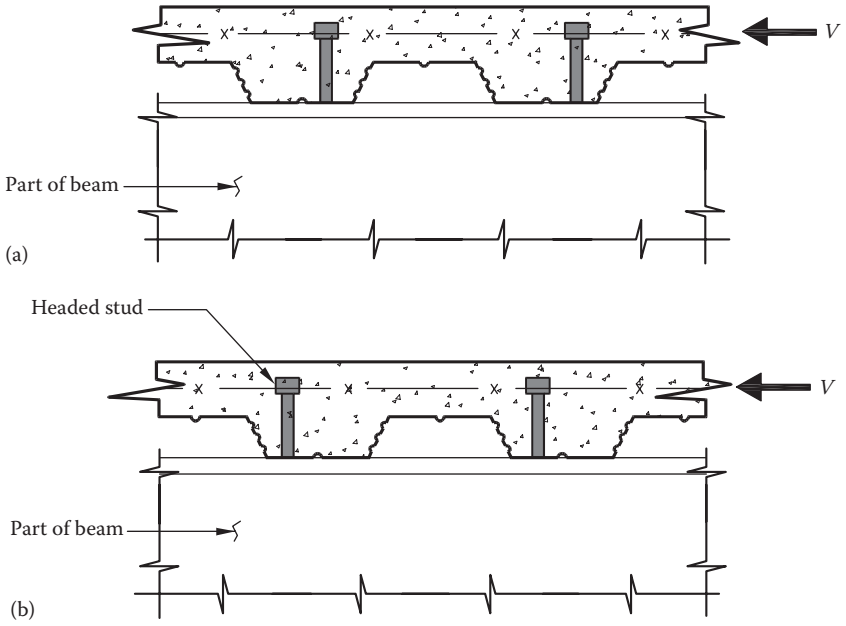


FIGURE 4.7 Possible stud positions: (a) weak position; (b) strong position. Note that AISC sets the default value for shear strength equal to that for the stud weak position.

application of this limitation is to select only beams with flanges thicker than the stud diameter divided by 2.5.

18. The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is six diameters. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered rows of studs. When deck ribs are parallel to the beam and the design requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure 4.8 shows possible connector arrangements.

See Figure 4.9 for a graphical display of some of the AISC requirements outlined above.

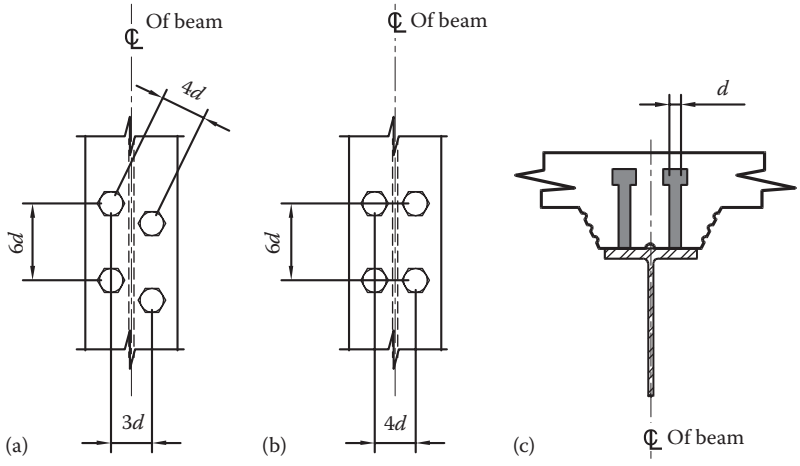


FIGURE 4.8 Shear connector arrangements: (a and b) plans; (c) section.

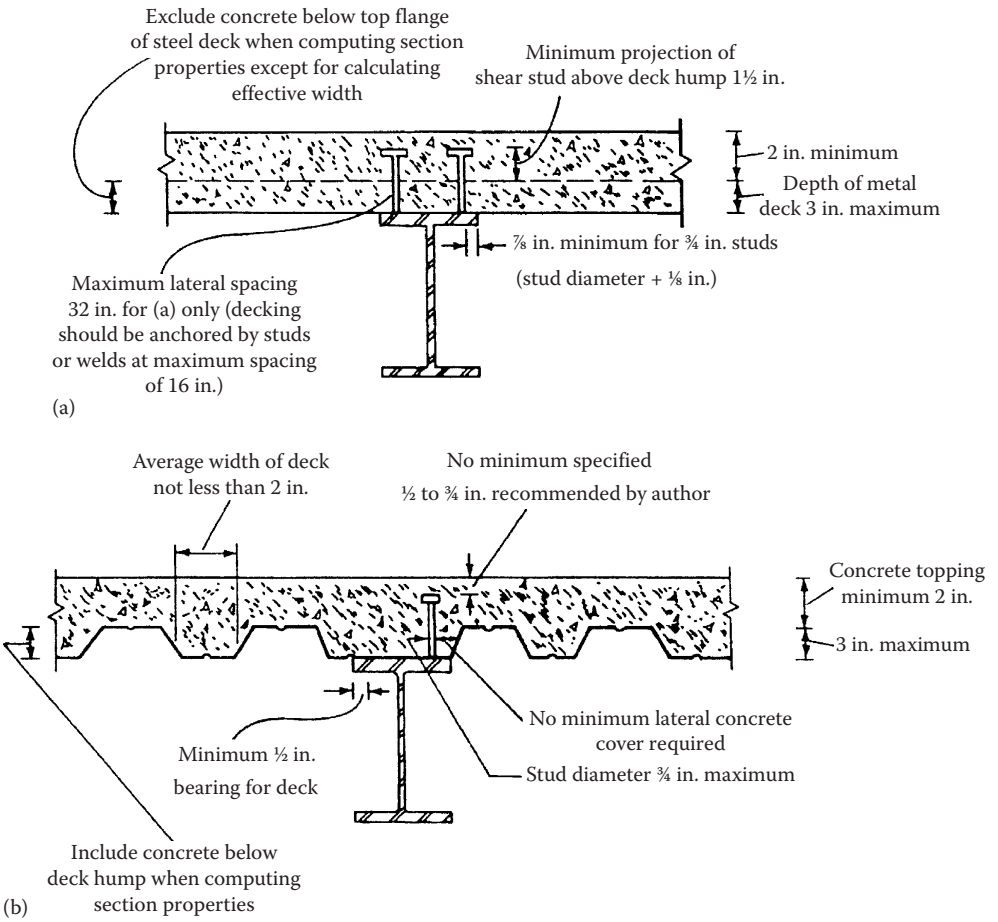


FIGURE 4.9 Composite beam, AISC requirements: (a) deck perpendicular to beam; (b) deck parallel to beam. *Note*: Dimension and clearance restrictions shown in either (a) or (b), apply to both unless noted.

4.2.1.2 Effective Width

The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. In cases where the effective stiffness of a beam with a one-sided slab is important, special care should be exercised since this model can substantially overestimate stiffness. To simplify design, effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

The *effective width* of the concrete slab shown in Figure 4.10 is the sum of the effective width for each side of the *beam* centerline, each of which shall not exceed:

1. One-eighth of the beam span, center-to-center of supports
2. One-half the distance to the centerline of the adjacent beam
3. The distance to the edge of the slab

For design purposes, a composite floor system is assumed to consist of a series of T beams, each made up of one steel beam and a portion of the concrete slab. The AISC limits on the width of slab that can be considered effective in the composite action are shown in Figure 4.10. When slab extends on one side of the beam only, as in beams adjacent to floor openings, the effective width naturally is less than when the slab extends on both sides of the beam.

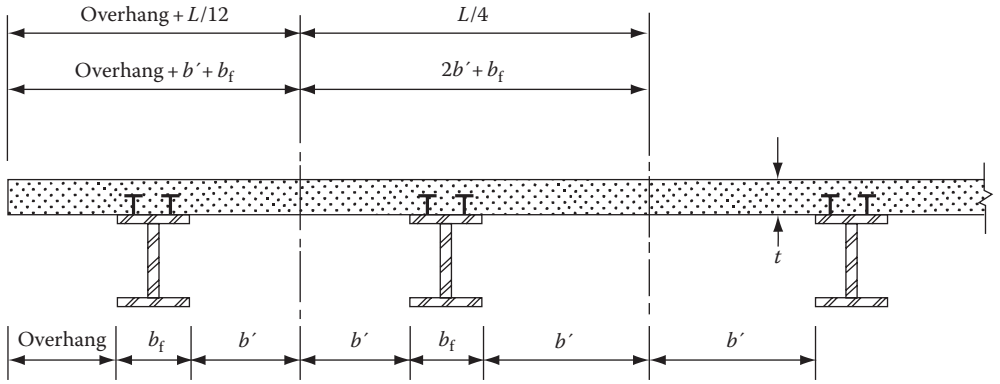


FIGURE 4.10 Effective width concept as defined in the AISC 360-05/10 specifications.

The design of composite beams is achieved by the transformed area method, in which the concrete effective area of the composite beam is transformed into an equivalent steel area.

The method assumes transverse compatibility at the concrete and steel interface. The unit stress in each material is equal to the strain times its modulus of elasticity. Because of strain compatibility, the stress in steel is n times the stress in concrete, where n is the modular ratio E_s/E_c. A unit area of steel is, therefore, mathematically equivalent to n times the concrete area. Therefore, the effective area of concrete $A_c = bt$ can be replaced by an equivalent steel area A_c/n.

For strength calculations, the AISC specification uses the value of n for normal-weight concrete of the specified strength. For deflection computations, n depends not only on the specified strength but also the weight of concrete. Therefore, in computing deflections, especially for beams subjected to heavy sustained loads, it is necessary to account for the effects of creep. This is even more important in shored construction when dead load of the concrete is also resisted by composite action. Creep effect is accounted for in computing deflections by using a higher modular ratio, n.

A multiplication factor of 2 for creep effects appears to be adequate in building designs. Live loads are always resisted by the composite section. If they are of short durations, the deflections are computed using the short-term modular ratio.

4.2.1.3 Positive Flexural Strength

The design positive flexural strength, $\phi_b M_n$, is determined using ϕ_b, the resistance factor for flexure, equal to 0.9 and M_n, the nominal flexural strength determined as follows:

1. For $h/t_w \text{ " } 3.76\sqrt{E/F_y}$,
   M_n is to be determined from the plastic *stress* distribution on the *composite* section for the limit state of *yielding* (plastic moment).
   It should be noted that all current ASTM A6, W, S and HP shapes satisfy the above requirement for $F_y \leq 50$ ksi.
2. For $h/t_w > 3.76\sqrt{E/F_y}$,
   M_n is determined from the superposition of elastic stresses, considering the effects of shoring, for the limit state of yielding.

4.2.1.4 Negative Flexural Strength

The design negative flexural strength, $\phi_b M_n$, is determined for the steel section alone using

$$\phi_b = 0.9$$

Alternatively, the available negative flexural strength may be determined from the plastic stress distribution of the *composite* section, provided that

1. The steel *beam* is *compact* and is adequately braced
2. *Shear connectors* connect the slab to the steel beam in the negative moment region
3. The slab reinforcement parallel to the steel beam, within the *effective width* of the slab, is *properly developed*

4.2.1.5 Shear Connectors

1. Load transfer for positive moment
   The entire *horizontal shear* at the interface between the steel *beam* and the concrete slab shall be assumed to be transferred by shear connectors, except for *concrete-encased beams*. For *composite* action with concrete subject to flexural compression, the total horizontal shear force, V', between the point of maximum positive moment and the point of zero moment shall be taken as the lowest value according to *the limit states of concrete crushing, tensile yielding* of the steel section, or strength of the shear connectors:
   a. Concrete crushing

$$V' = 0.85 f'_c A_c$$

   b. Tensile yielding of the steel section

$$V' = F_y A_s$$

   c. Strength of shear connectors

$$V' = \sum Q_n$$

   where
   A_c is the area of concrete slab with effective width, in.^{2}
   A_s is the area of steel cross section, in.^{2}
   ΣQ_n is the sum of nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment, kip

2. Load transfer for negative moment
   In continuous *composite beams* where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel *beam*, the total *horizontal shear force* between the point of maximum negative moment and the point of zero moment shall be taken as the lower value of tensile yielding of the steel reinforcement in the slab, or strength of the shear connectors:
   a. Tensile yielding of the slab reinforcement

$$V' = A_r F_{yr}$$

   where
   A_r is the area of adequately developed longitudinal reinforcing steel within the *effective width* of the concrete slab, in.^{2}
   F_{yr} is the *specified minimum yield stress* of the reinforcing steel, ksi

   b. Strength of shear connectors

$$V' = \sum Q_n$$

3. Strength of stud shear connectors
   The *nominal strength* of one stud shear connector embedded in solid concrete or in a *composite* slab is

$$Q_n = 0.54 A_{sc} \sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u$$

   where
   A_{sc} is the cross-sectional area of stud shear connector, in.^{2}
   E_c is the modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f'_c}$, ksi
   F_u is the *specified minimum tensile* strength of a stud shear connector, ksi
   R_g and R_p are the stud shear capacity reduction factors as described in the following.
   R_g = 1.0—(a) for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for any number of studs welded in a row directly to the steel shape; (c) for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the *average rib width* to rib depth ≥ 1.5
   = 0.85—(a) for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for one stud welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth ≤ 1.5
   = 0.7—for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape
   R_p = 1.0—for studs welded directly to the steel shape (in other words, not through steel deck or sheet) and having a haunch detail with not more than 50% of the top flange covered by deck or sheet steel closures
   = 0.75—(a) for studs welded in a *composite* slab with the deck oriented perpendicular to the beam and $e_{mid\text{-}ht} \geq 2$ in.; (b) for studs welded through steel deck, or steel sheet used as *girder filler* material, and embedded in a *composite* slab with the deck oriented parallel to the beam
   = 0.6—for studs welded in a composite slab with deck oriented perpendicular to the beam and $e_{mid\text{-}ht} \geq 2$ in.
   $e_{mid\text{-}ht} \geq 2$ in. = distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the *load* bearing direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), in.
   w_c = weight of concrete per unit volume ($90 \leq w_c \leq 155$ lb/ft^3)

   Values for R_g and R_p for typical cases are given in Table 4.1. Note the stud reduction factors R_g and R_p are equal to unity for composite steel beams with solid slab (Figure 4.3a).
4. Required number of shear connectors
   The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the *horizontal shear force* divided by the nominal strength of one shear connector.
5. Shear connector placement and spacing
   Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment. However, the number of shear connectors placed between any concentrated *load* and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the *concentrated load* point.

4.2.1.6 Deflection Considerations

When a composite beam is controlled by deflection, the design should limit the behavior of the beam to the elastic range under serviceability load combinations. In other words, calculate deflections using elastic properties of composite beam, and working stress design (WSD). It is often

TABLE 4.1
Values of Stud Reduction Factors R_g and R_p for Typical Cases

Condition	R_g	R_p
No decking	1.0	1.0
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \geq 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85	0.75
Decking oriented perpendicular to the steel shape		
Number of studs occupying the same decking rib		
1	1.0	0.6
2	0.85	0.6
3 or more	0.7	0.6

h_r, Nominal rib heights, in.; w_r, average width of concrete rib or haunch, in.

not practical to make accurate stiffness calculations of composite flexural members. Therefore, for realistic deflection calculations, one may use a lower bound moment of inertia, I_{lb}, as defined below:

$$I_{lb} = I_s + A_s(Y_{ENA} - d_3)^2 + (\sum Q_n/F_y)(2d_3 + d_1 - Y_{ENA})^2$$

where

A_s is the area of steel cross section, in.^{2}
d_1 is the distance from the compression force in the concrete to the top of the steel section, in.
d_3 is the distance from the resultant steel tension force for full section tension yield to the top of the steel, in.
I_{lb} is the lower bound moment of inertia, in.^{4}

where

S_s is the section modulus for the structural steel section, referred to the tension flange, in.^{3}
S_{tr} is the section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3}

The above equations should not be used for ratios, $\Sigma Q_n/C_f$, less than 0.25. This restriction is to prevent excessive slip, as well as substantial loss in beam stiffness.

I_s = the moment of inertia for the structural steel section, in.^{4}
ΣQ_n = the sum of the nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kip

$$Y_{ENA} = [(A_s d_3 + (\sum Q_n/F_y)(2d_3 + d_1)) / (A_s + (\sum Q_n/F_y))]$$

The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design. The stiffness calculated using a weighted average of moments of inertia in the positive moment region and negative moment regions may take the following form:

$$I_t = aI_{pos} + bI_{neg}$$

where

I_{pos} is the effective moment of inertia for positive moment, in.^{4}

I_{neg} is the effective moment of inertia for negative moment, in.^{4}

The effective moment of inertia is based on the cracked transformed section considering the degree of composite action. For continuous beams subjected to gravity loads only, the value of a may be taken as 0.6 and the value of b may be taken as 0.5 for calculations related to drift.

For cases where elastic properties of partially composite beams are needed, the elastic moment of inertia may be approximated by

$$I_{equiv} = I_s + \sqrt{(\sum Q_n / C_f)}\,(I_{tr} - I_s)$$

where

I_s is the moment of inertia for the structural steel section, in.^{4}

I_{tr} is the moment of inertia for the fully composite uncracked transformed section, in.^{4}

ΣQ_n is the strength of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kip

C_f is the compression force in concrete slab for fully composite beam; smaller of $A_s F_y$ and $0.85 f'_c A_c$, kip

A_c is the area of concrete slab within the effective width, in.^{2}

The effective section modulus, S_{eff}, referred to the tension flange of the steel section for a partially composite beam, may be approximated by

$$S_{eff} = S_s + \sqrt{(\sum Q_n / C_f)}\,(S_{tr} - S_s)$$

4.2.1.7 Design Outline for Composite Beam

In this day and age of computers ruling the world, it is beyond the bounds of possibility or reason to expect engineers to perform hand calculations for designing composite beam, and for that matter any other structural component. However, in keeping up with the spirit of this book, in lieu of intense calculations, we will outline only the required design steps. The purpose of this indulgence is to enhance our understanding of what goes on in the black box, the computer.

The design steps are

1. *Limit material properties as follows*:
   Normal weight concrete: 3 ksi ≤ f'_c ≤ 10 ksi
   Lightweight concrete: 3 ksi ≤ f'_c ≤ 6 ksi
   Structural steel and reinforcing: $F_y \leq 75$ ksi
2. *Loads*
   Dead loads typically consist of slab and beam weight, miscellaneous of 10 psf for ceiling etc., and an allowance of 15 psf for partitions (when live loads are less than 100 psf).
   Live loads shall be as specified in the applicable building codes.
   Use higher loads if specified by the building user.
   Reduce live loads if permitted by the applicable building code
3. *Determine the required flexural strength*
   In the most typical case of simply supported beams subject to uniformly distributed load, w_u, and span ℓ (see Figure 4.11),

$$M_u = \frac{w\ell^2}{8}$$

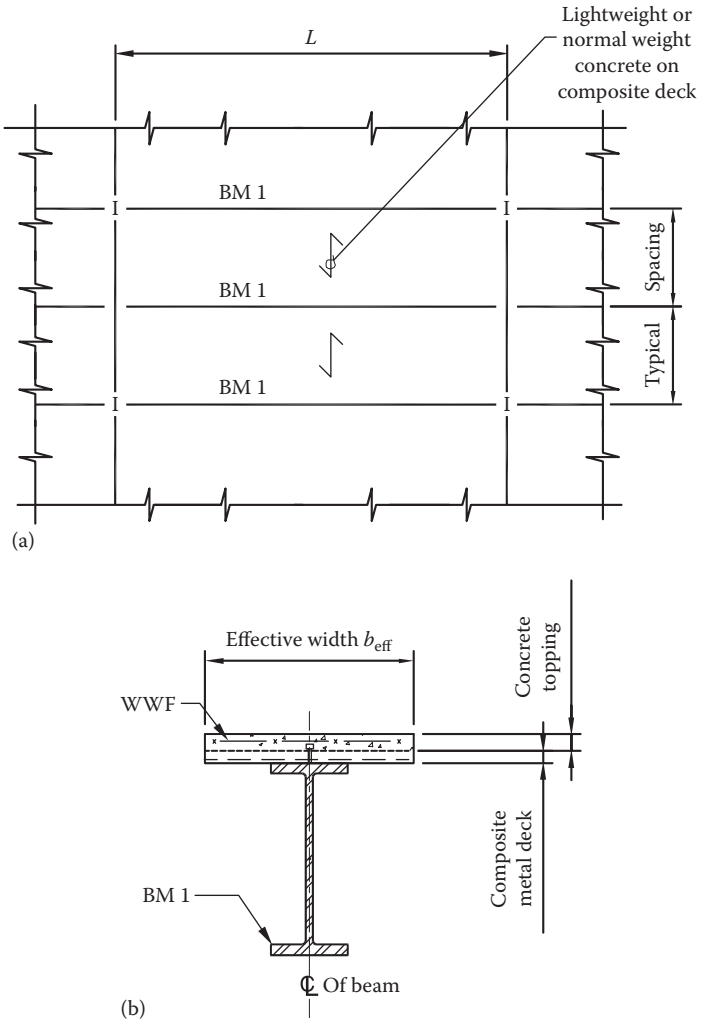


FIGURE 4.11 Composite beam: (a) partial framing plan; (b) section.

4. *Determine* b_{eff}
   The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, which shall not exceed:
   a. One-eighth of the beam span, center to center of supports
   b. One-half the distance to center-line of the adjacent beam
   c. The distance to the edge of the slab
5. *Calculate the moment arm for the concrete force measured from the top of the steel shape, Y2*
   Assume depth of compression block, $a = 1.0$ in. (Some assumption must be made to start the design process. An assumption of 1.0 in. has proven to be a reasonable starting point in many design problems.)

$$Y2 = t_{slab} - a/2$$

Enter AISC Steel Design Manual, Table 3-19 with the required strength and the calculated value of Y2. Select a beam and a plastic neutral axis location that indicates sufficient available strength. The trial beam is okay if $\phi_b M_n = 0.9 M_n \geq M_u$.

6. *Check the beam deflections and available strength*
   Check the deflection of the steel beam under construction loads considering only the weight of steel beam and concrete topping, and an allowance of 20 psf for construction live loads as contributing to the construction load.

   Limit deflection to a maximum of 2.5 in. to facilitate concrete placement. Camber the beam if the calculated deflection is greater than ¾ in. Provide camber equal to 75% of the calculated dead load deflection.

$$I_{\text{req}} = \frac{5}{384}\frac{w_{\text{DL}} l^4}{E\Delta}$$

   Revise trial section if I_{req} is greater than that of the trial section.
7. *Check selected member strength as an unshored beam under construction loads assuming adequate lateral bracing through the deck attachment to the beam flange*. Also determine ΣQ_{u}. Calculate the required strength of steel beam alone using the weight of the steel beam and concrete slab as dead loads, and an allowance of 20 psf construction load. Label this load as $w_{(\text{const})}$.

$$M_{\text{u(unshored)}} = \frac{w_{(\text{const})} \times l^2}{8}$$

$$\phi_{\text{b}} M_{n(\text{steel})} \geq M_{\text{u(unshored)}}$$

   Using the assumed values of Y_2 and the plastic neutral axis, PNA, determine the required horizontal shear ΣQ_n.
8. *Check the value a, using the relation*

$$a = \frac{\sum Q_{\text{u}}}{0.55 f'_{\text{c}}\, b}$$

   If the calculated value of *a* is less than the assumed value, the design is o.k.
9. *Check live load deflection*

$$\Delta_{\text{LL}} = \frac{5}{384}\frac{w_{\text{LL}} l^4}{EI_{\text{LB}}}$$

10. *Determine if the beam has sufficient available shear strength* $\phi V_n \geq V_{\text{u}}$. This is not an issue for typical uniformly loaded beams.
11. *Determine the required number of shear connectors = 2N*
    With the known direction and profile of metal deck, determine shear capacity of one ¾ in., diameter stud per rib for the given strength and weight of the concrete slab. Here, we identify ¾ in. diameter studs, because they are the most common type used in building construction, in North America.

$$N = \frac{\sum Q_n}{Q_n} \quad \text{(on each side of the beam)}$$

12. *Check the spacing of shear connectors*
    Use one stud every flute, starting at each support. If studs remain, double up studs on each end of the span. Check spacing requirement.

$$6d_{\text{stud}} < 12. < 8t_{\text{slab}}$$

    Note that stud length shall extend a minimum of 1½ in. into slab after welding.

4.3 COMPOSITE JOISTS AND TRUSSES

4.3.1 Composite Joists

Composite floors using joists and trusses typically involve simply supported members. The joists and trusses are linked by shear connectors to the concrete slab to form an effective T beam to resist gravity loads. The versatility of the system results from the inherent strength of the concrete floor component in compression and the ability of the joists and trusses to span long distances. Composite floor systems are advantageous in reducing material cost, on-site labor and construction time. They also result in reduced structural depth. The composite action of the framing element is by direct welding of shear studs through the steel deck while the composite action of the steel deck as flexural reinforcement for the concrete slab itself results from side embossments incorporated into the steel-sheet profile. The slab-and-beam arrangement typical in composite floor systems produces a horizontal diaphragm that provides stability to the overall building system while distributing wind and seismic shears to the lateral load-resisting elements.

Pre-engineered, proprietary open-web floor joists, joist girders, and fabricated floor trusses when combined with a concrete floor slab are viable composite members. The advantages of the system is in the increased span length and stiffness due to the greater structural depth and ease in accommodating electrical conduits, plumbing and heating, and air conditioning ductwork. The now nonexistent WTC Towers, New York, used composite open web joists.

4.3.2 Composite Trusses

Built-up fabricated composite floor trusses combine material efficiency in relatively long-span applications with maximum flexibility for incorporating building services ductwork and piping into the ceiling cavity. The triangular openings formed by web members of the truss allow the passage of large mechanical air ducts as well as other piping and electrical lines. The increased depth of the composite truss system over a standard rolled shape composite-beam system, with building services ductwork and piping passing below the beam, results in maximum material efficiency and high flexural stiffness. Generally, composite floor trusses are considered economical for floor spans in excess of 45 ft. A further requirement for floor truss systems is that the framing layout must be uniform and repetitive, resulting in relatively few types of trusses, which can be readily built in the fabrication shop. Otherwise the high level of fabrication inherent in the floor truss assemblage tends to offset the relative material efficiency. For this reason, composite floor truss systems are particularly attractive in high-rise office building applications where large column-free areas are required and floor configurations are generally repetitive over the height of the building.

Shown in Figure 4.12 are schematics of floor framing with composite trusses. Observe that the top and bottom chords consist of T-sections to allow for direct welding of double-angle web members without using gusset plates. When space between the diagonals is not sufficient to allow passage of mechanical and air-conditioning ducts, vertical webs may be welded between the truss chords to form a Vierendeel panel. See Figure 4.12(c).

A variety of chord and web member cross sections may be used in developing a truss system. The previous example showed T-sections for top and bottom chords to allow direct connection of web members without gusset plates. Web members are most often tees, single-angle or double-angle

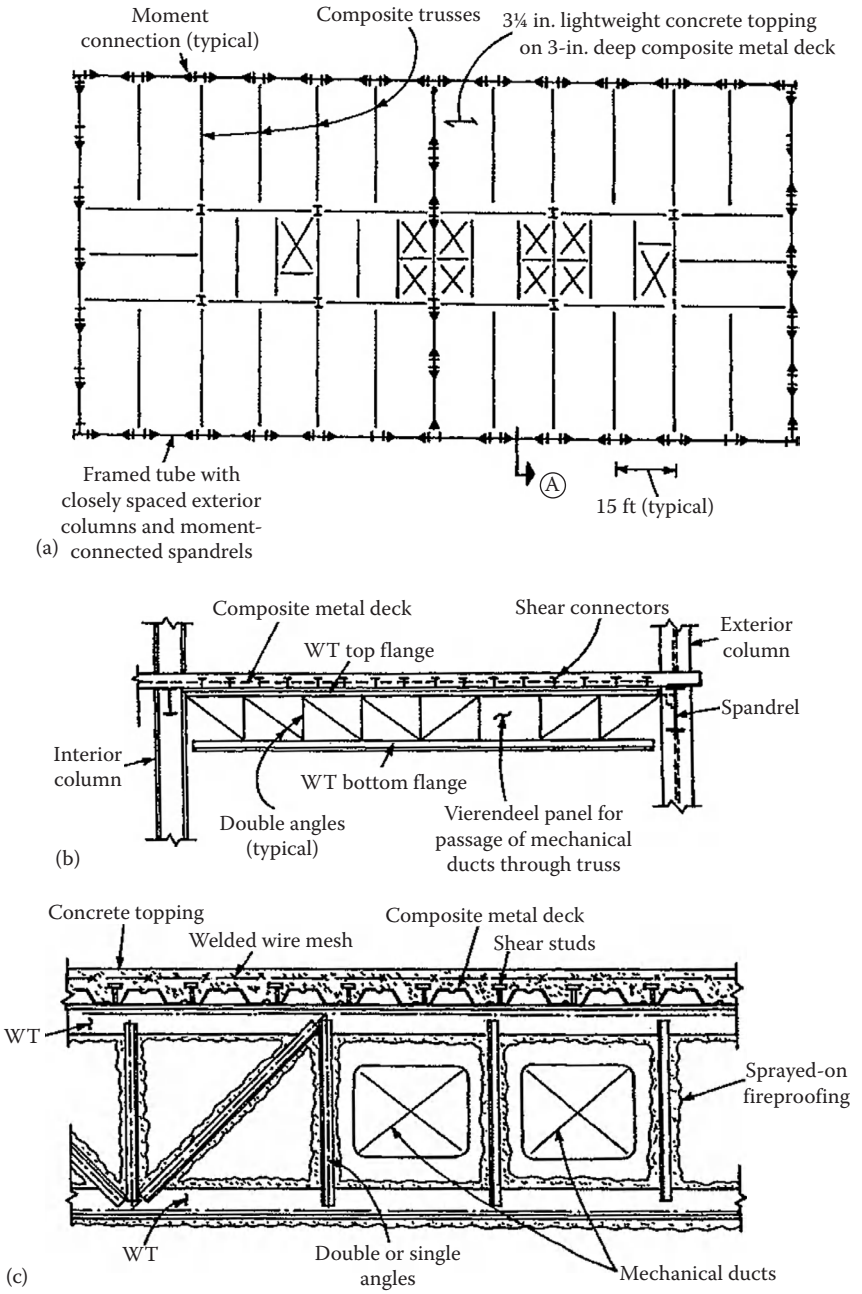


FIGURE 4.12 Composite truss: (a) floor framing plan; (b) section A; (c) Vierendeel panels.

sections welded directly to the chord tee. Some examples are shown for Figure 4.13. The composite floor trusses are connected to the concrete floor slab by shear connectors welded to the top chord of the truss. The floor trusses are normally spaced such that the metal deck can span between the trusses without shoring. The space between the diagonals is used for the passage of mechanical and air-conditioning ducts. When the space between the diagonals is not sufficient, vertical members may be welded between the chords to form a Vierendeel panel.

Figure 4.13 shows some examples of composite truss elevations while in Figure 4.14 schematic cross sections illustrating truss components are shown.

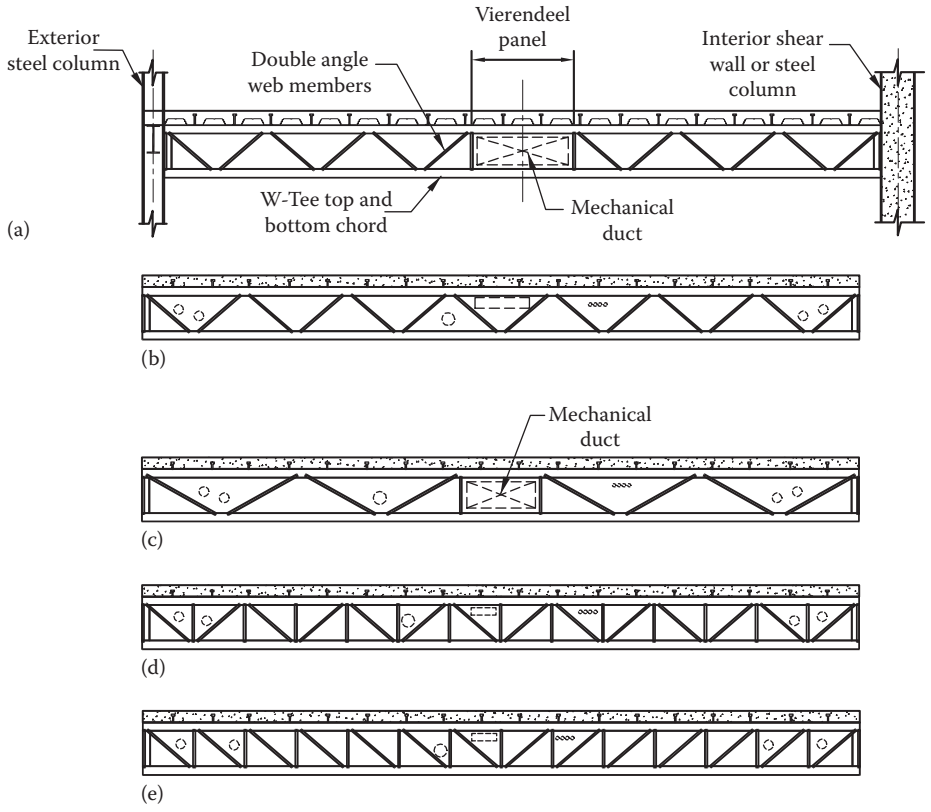


FIGURE 4.13 Examples of composite floor trusses: (a–e) elevations.

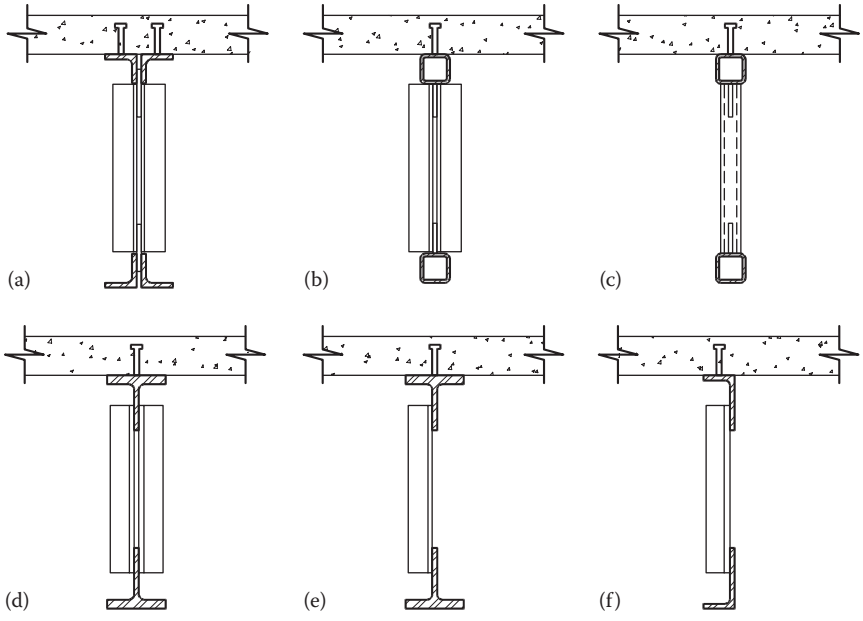


FIGURE 4.14 Composite truss schematic sections: (a through c) top and bottom chords with gusset plates; (d through f) top and bottom chords without gusset plates.

4.4 OTHER TYPES OF COMPOSITE FLOOR CONSTRUCTION

The slab elements of a composite floor system may take the form of a flat soffit reinforced concrete slab, precast-concrete planks or floor panels with a cast-in-place topping (see Figures 4.15 and 4.16).

Early composite floor systems used concrete-encased steel beams to support a formed reinforced concrete slab between the beams. The concrete encasement of the steel beam was eliminated with the development of economical lightweight sprayed-on fireproofing. The reinforcement for the slab in the direction perpendicular to the beam span is determined through conventional continuous reinforced concrete design. Light slab reinforcement is placed parallel to the beam span to control shrinkage and thermal cracking.

To eliminate the cost and additional temporary formwork for the slab, precast-concrete planks may be used as permanent formwork which also provides an effective working platform for the construction trades. Precast, prestressed hollow-core concrete planks spanning between the steel floor beams are well suited for jobs where a repetitive organization of beam spacings allow for effective prefabrication of the planks. Two to three inches of concrete is cast atop the planks to provide an effective diaphragm as well as a level, continuous floor surface. The concrete topping is reinforced, usually with welded-wire fabric, to control concrete cracking. In this type of construction, the topping extends down to the surface of the steel beams. Note that the shear studs extend up into the topping above the hollow concrete planks; see Figure 4.16 for schematics.

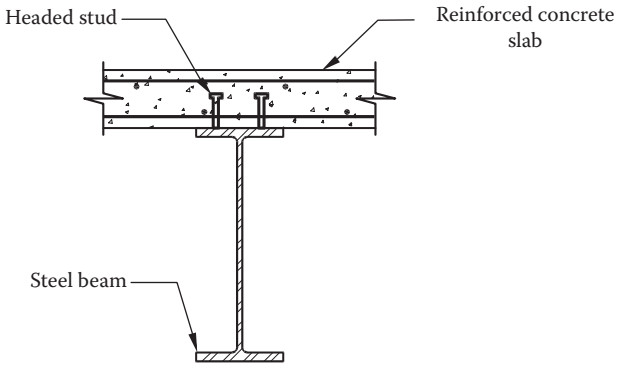


FIGURE 4.15 Composite beam with flat soffit reinforced concrete slab.

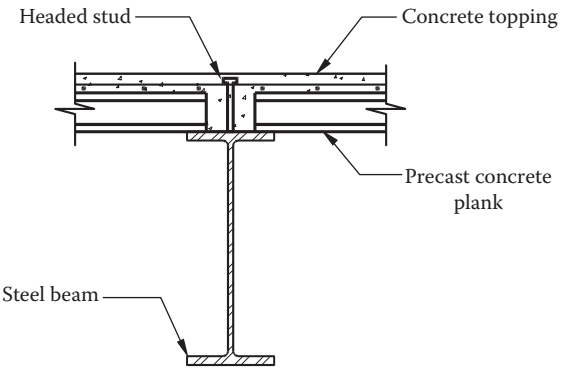


FIGURE 4.16 Composite beam with precast concrete plank and topping slab.

4.5 CONTINUOUS COMPOSITE BEAMS

Successful composite-beam design requires consideration of serviceability issues such as floor vibrations. Of particular concern is the issue of perceptibility of occupant-induced floor vibrations. The high flexural stiffness of composite floor framing systems results in relatively low vibration amplitudes from transitory heel-drop excitations and therefore is effective in reducing perceptibility. Typically, composite floor framing systems perform quite well and rarely transmit annoying vibrations to the occupants. However, particular care for spans in the 30–40 ft. range is recommended. Anticipated damping provided by partitions, ceiling construction, and the structure itself should be considered in conjunction with state-of-the-art prediction models to evaluate the potential for perceptible floor vibrations.

The serviceability characteristics of composite beams may be significantly improved through the use of semi-rigid connections. Such a typical semi-rigid joint is shown in Figure 4.17 in which partial end-restraint is mobilized through the addition of a bottom angle, and heavier reinforcement within the concrete slab in a band centered on the steel column. For unshored construction, the initial dead-load deflection is hardly altered, but live-load deflections, and quite possibly floor vibrations, are significantly reduced through the partial end restraint. Reduced positive moment results in a saving in steel tonnage. The partial end restraint may be used to shorten the effective unbraced length of the column as well. Full benefit of the system is obtained through shored construction where all loads are resisted by the semi-rigid composite connection. However, application of semi-rigid connections is limited because of the difficulty in determining the moment–rotation characteristics of the joint.

A composite beam subject to negative bending moment experiences tensile stresses in the concrete zone and therefore, loses much of its advantage. However, when mild steel reinforcement is placed parallel to the beam within the effective width of slab, and is anchored adequately to develop the tensile forces, the advantage of continuous construction is restored. The reinforcing steel used in the tensile zone is included in computing the property of the composite section. Similarly, when the compressive stress in concrete subject to positive moment exceeds the allowable stress, it is permissible to use compressive steel in the effective width zone to reduce stresses.

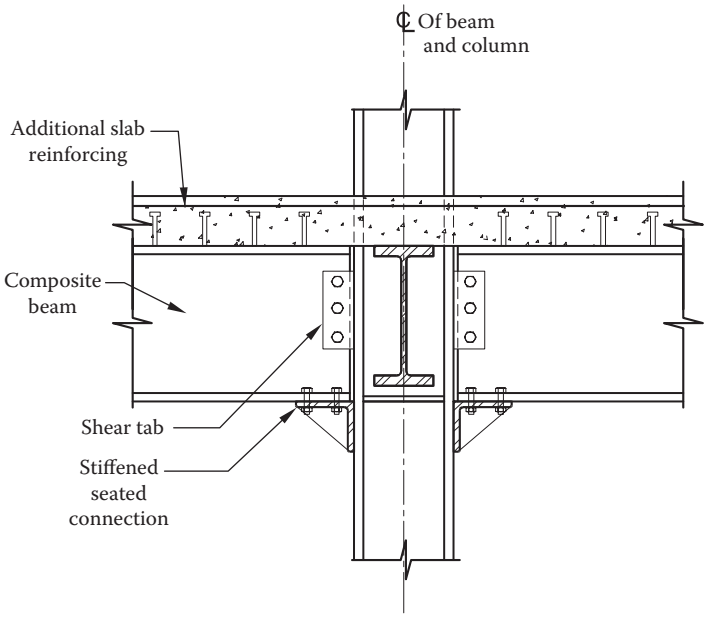


FIGURE 4.17 Semi-rigid composite beam-to-column connection.

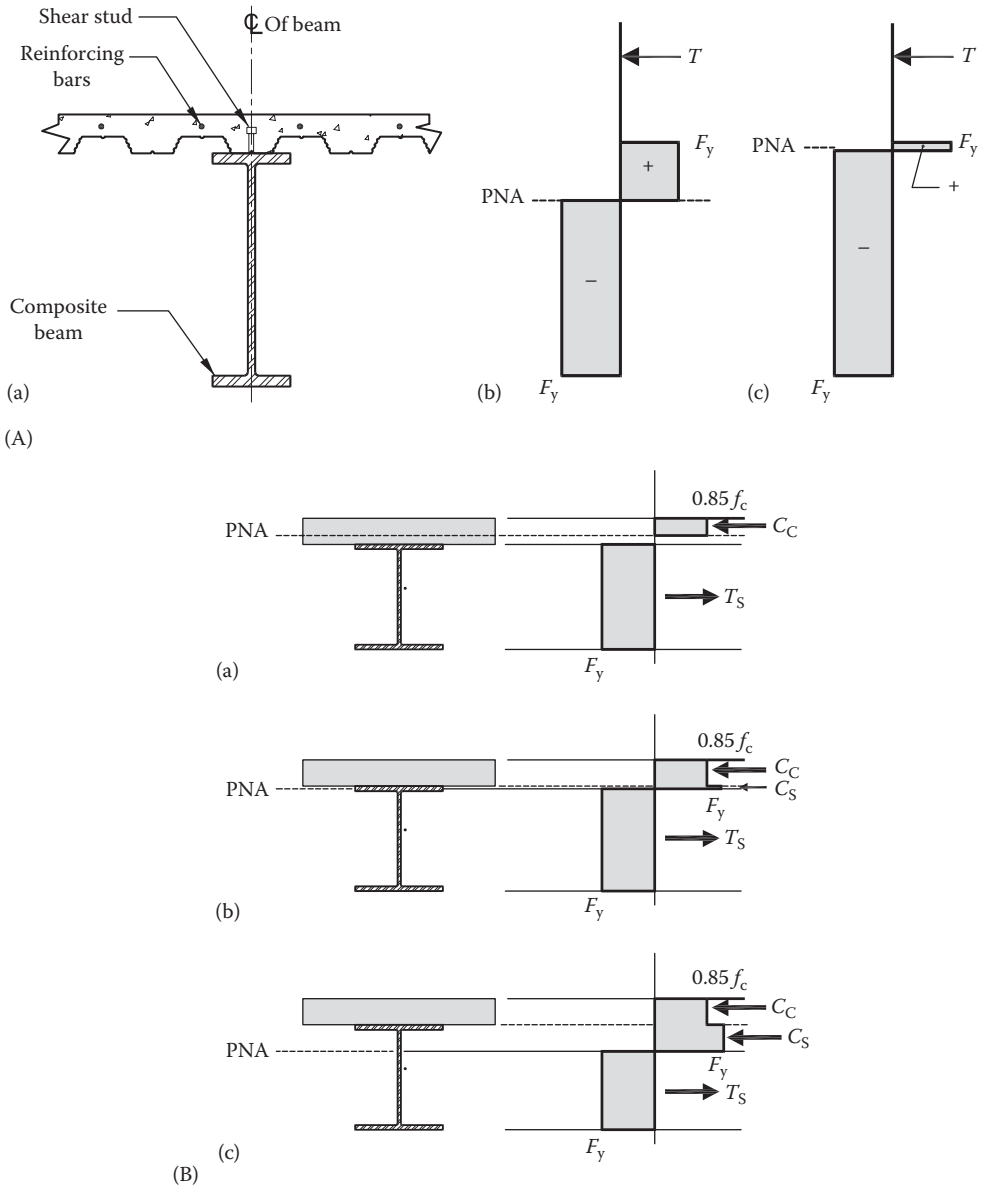


FIGURE 4.18 (A) Plastic stress distribution for negative moment: (a) composite beam section; (b) plastic neutral axis, PNA, in steel beam web; (c) PNA in beam flange. (B) Plastic stress distribution for positive moment: (a) plastic neutral axis, PNA, in concrete slab; (b) PNA in steel beam flange; (c) PNA in steel beam web.

Although the design negative flexural strength, $\phi_b M_n$, may be determined for the steel section alone, it is prudent to include the contribution of longitudinal reinforcement by considering plastic stress distribution in the composite section. Schematics of plastic stress distribution for negative and positive moments are shown in Figure 4.18.

4.6 NONPRISMATIC COMPOSITE BEAMS AND GIRDERS

A prismatic composite steel beam has two apparent disadvantages over other types of composite floor framing. One, the member must be designed for the maximum bending moment occurring near midspan and thus is under-stressed at all other sections along the span and, two, building

services ductwork and piping must pass beneath the beam or the beam must be provided with penetrations (usually reinforced with plates leading to high fabrication costs). For this reason, a number of composite girder articulations that allow passage of mechanical ducts and related services through the depth of the girder have been developed. These include tapered and dapped girders, castellated beams, and stub-girder systems.

Use of tapered composite girders with a maximum depth at center span is limited, as the main mechanical duct loop normally runs through the center of the span rather than at each end.

A castellated beam on the other hand, has a constant depth. It is formed from a single rolled wide-flange steel beam cut and reassembled by welding. The resulting beam has increased depth and provides for hexagonal openings over the entire span. However, its use in the United States is somewhat limited because of increased fabrication costs and standard castellated openings may not be large enough to accommodate the required mechanical ductwork.

The stub-girder system explained shortly in this chapter, involves the use of short sections of beam welded to the top flange of a continuous, heavier bottom girder. Continuous transverse secondary beams are positioned over the girder while the mechanical ducts pass through the openings formed between the beam stubs. This system, quite popular in the 1970s and 1980s, has been used in many building projects totaling well over 30 million square feet. The system, however, requires shoring that may offset some of the cost savings.

4.7 MOMENT-CONNECTED COMPOSITE HAUNCH GIRDERS

Composite haunch girders, although not often used as a floor framing system, merit mention because they minimize the floor-to-floor height without requiring complicated fabrication. As an example, shown in Figure 4.19 is a schematic floor plan in which composite haunch girders frame between exterior columns and interior core framing. The haunch girder typically consists of a shallow steel beam, 10–12 in. deep for span in the 35–45 ft. range. At each end of the girder, a triangular haunch is formed by welding a diagonally cut wide-flange beam usually 24–27 in. deep (see Figure 4.20). The haunch is welded to the shallow beam and to the columns at each end of the girder. In this manner the last 8 or 9 ft. of the haunch girder at either end flares out toward the column with a depth varying from about 10–12 in. at the center to about 27 in. at the ends. The system uses less steel and provides greater flexibility for mechanical ducts, which can be placed anywhere under the shallow central span. The reduction in floor-to-floor height further cuts costs of exterior cladding and heating and cooling loads. The system, however, is not common because of higher fabrication costs.

A variation of the same concept shown in Figure 4.21 uses non-tapered haunches at each end. The square haunch girder can be fabricated using a shallow-rolled section in the center and two relatively deep wide flange sections, one at each end. Another method of fabricating the girder is

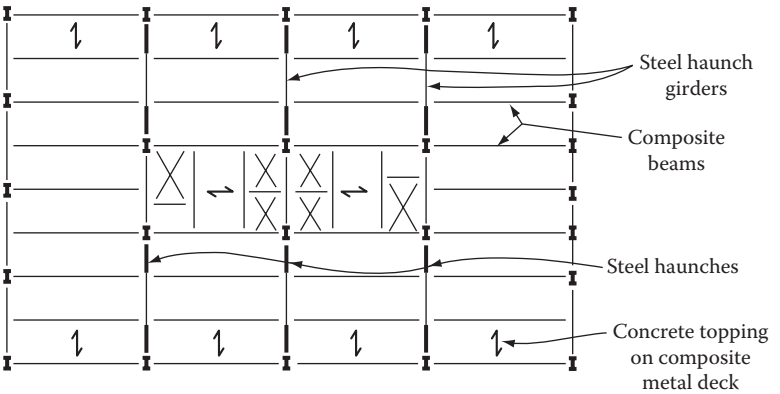


FIGURE 4.19 Schematic floor plan showing haunch girders.

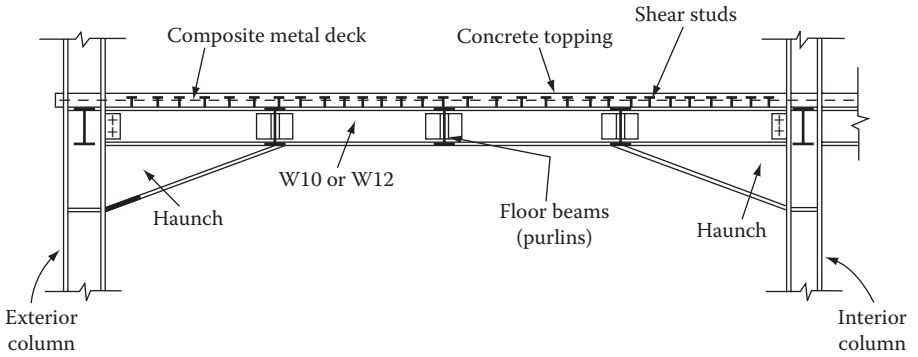


FIGURE 4.20 Composite girder with tapered haunch.

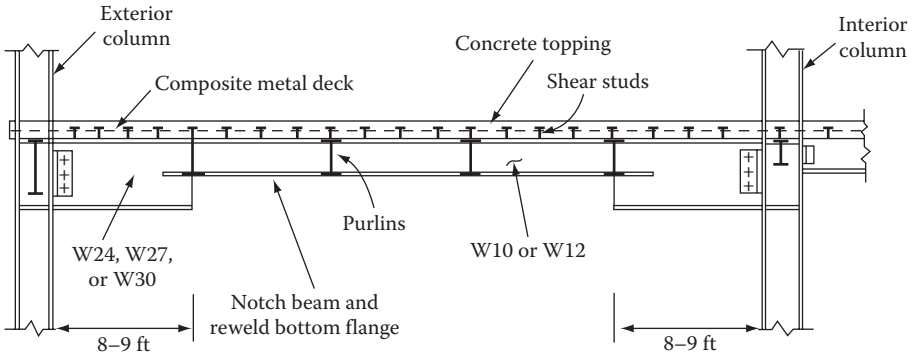


FIGURE 4.21 Composite girder with square haunch.

to notch the bottom portion of the girder at midspan and reweld the flange to the web. The method requires more steel but comparatively less fabrication work.

In comparison to a shallow girder of constant depth, a haunch girder is significantly stiffer. Hence, its use as a frame-beam for resisting lateral loads merits due considerations.

4.8 COMPOSITE STUB GIRDERS

In high-rise design, maximum flexibility is achieved if structural, mechanical, electrical, and plumbing trades have their own designated space in the ceiling. This is achieved in a conventional system by placing HVAC ducts, lights, and other fixtures under the beams. Where deep girders are used, penetrations are made in the girder webs to accommodate the ducts. In an office building, the typical span between the core and the exterior is about 40 ft requiring 18–21 in. deep beams. Usual requirements of HVAC ducts, lights, sprinklers, and ceiling construction result in overall depths of 4–4.25 ft between the ceiling and top of floor slab. The depth can, however, be decreased at a substantial penalty either by providing penetrations in relatively deep beams or by using shallower, less economical beam depths.

The stub girder system, Figure 4.22a through c, invented by engineer Dr. Joseph Caloco, attempts to eliminate some of these shortcomings while at the same time reduces the floor steel weight. The key components of the system are short stubs welded intermittently to the top flange of a shallow steel beam. Sufficient space is left between stubs to accommodate mechanical ducts. Floor beams are supported on top of, rather than framed into, the shallow steel beam. Thus, the floor beams are designed as continuous members which results in steel savings and reduced deflections. The stubs consist of short wide flange beams placed perpendicular to and between the floor beams. The floor system consists of concrete topping on steel decking connected to the top of stubs. The stub girders are typically spaced at 25–35 ft on center, spanning 40–45 ft between the core and the exterior of the building.

The behavior of a stub girder is akin to a Vierendeel truss; the concrete slab serves as the compression chord, the full-length steel beam as the bottom tension chord, and the steel stubs as vertical web members. From an overall consideration, the structure allows installation of mechanical system within the structural envelope, thus reducing floor-to-floor height; the mechanical ducts run through, and not under the floor (see Figure 4.22d and e).

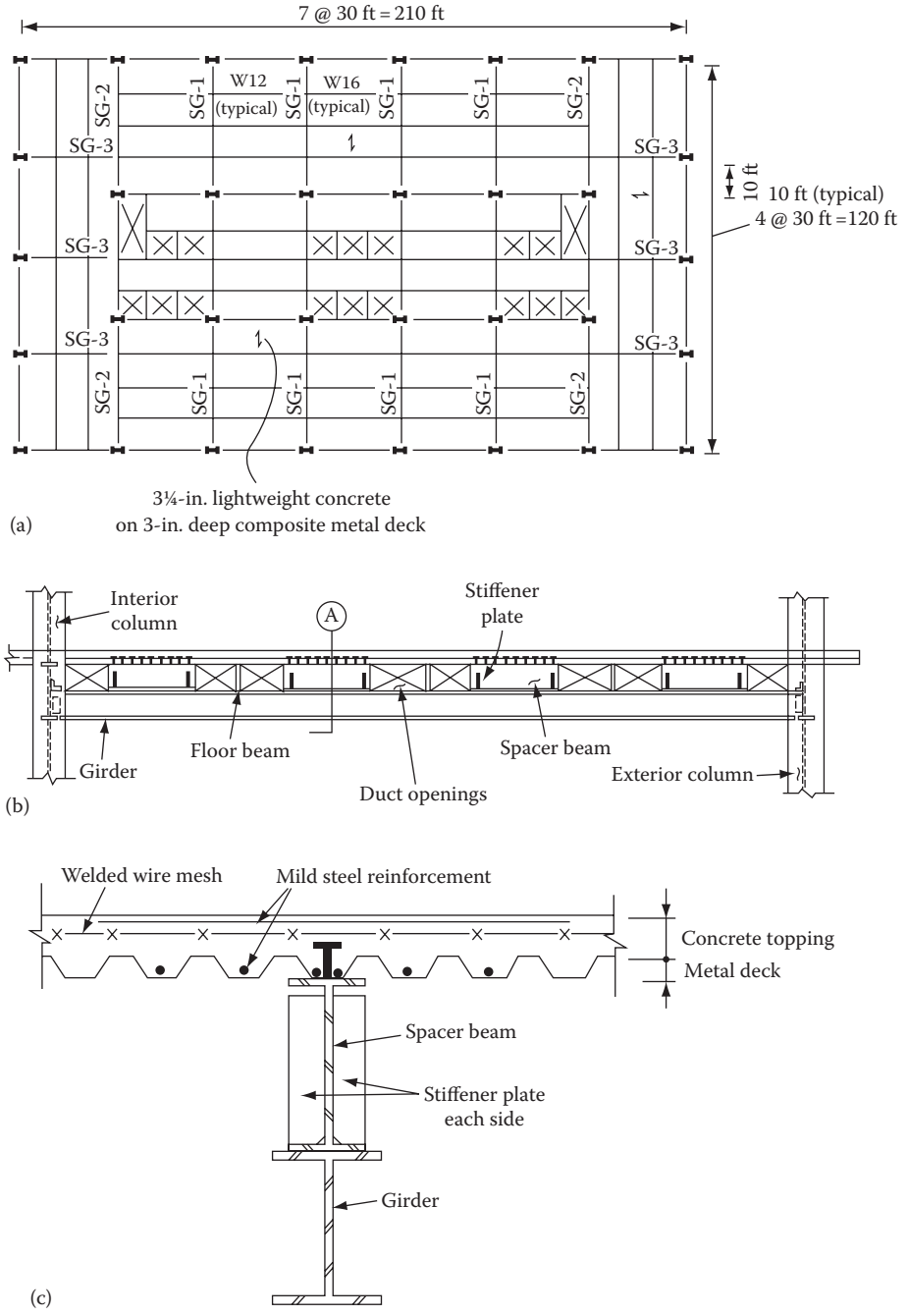


FIGURE 4.22 Stub girder framing: (a) framing plan; (b) elevation of stub girder SG-1; (c) section A through stub girder;



(d)



(e)

FIGURE 4.22 (continued) (d and e) photographs.

4.8.1 Behavior and Analysis

The primary action of a stub girder is similar to that of a Vierendeel truss; the bending moments are resisted by tension and compression forces in the bottom and top chords of the truss and the shearing stresses by the stub pieces welded to the top of wide-flange beam. The bottom chord is the steel wide-flange which resists the tensile forces. The compression forces are carried by the concrete slab with an effective width varying from about 6–7 ft requiring additional reinforcement to supplement the compression capacity of the concrete. The shear forces are resisted by the stub pieces, which are connected to the metal deck and concrete topping through shear connectors.

Figure 4.22a shows a typical floor plan with stub girders denoted as SG1, SG2, etc. Consider stub girder SG1, spanning 40 ft between the exterior and interior core of the building (Figure 4.22b). The deck consists of 3¼ in. (82.5 mm) lightweight structural concrete topping. A welded wire fabric is used as crack control reinforcement in the concrete slab.

The first step in the analysis is to model the stub girder as an equivalent Vierendeel truss. This is shown in Figure 4.23a. A 14 in. wide flange beam is used as the continuous bottom chord of the truss. The slab and the steel beam are modeled as equivalent top and bottom chords of the truss.

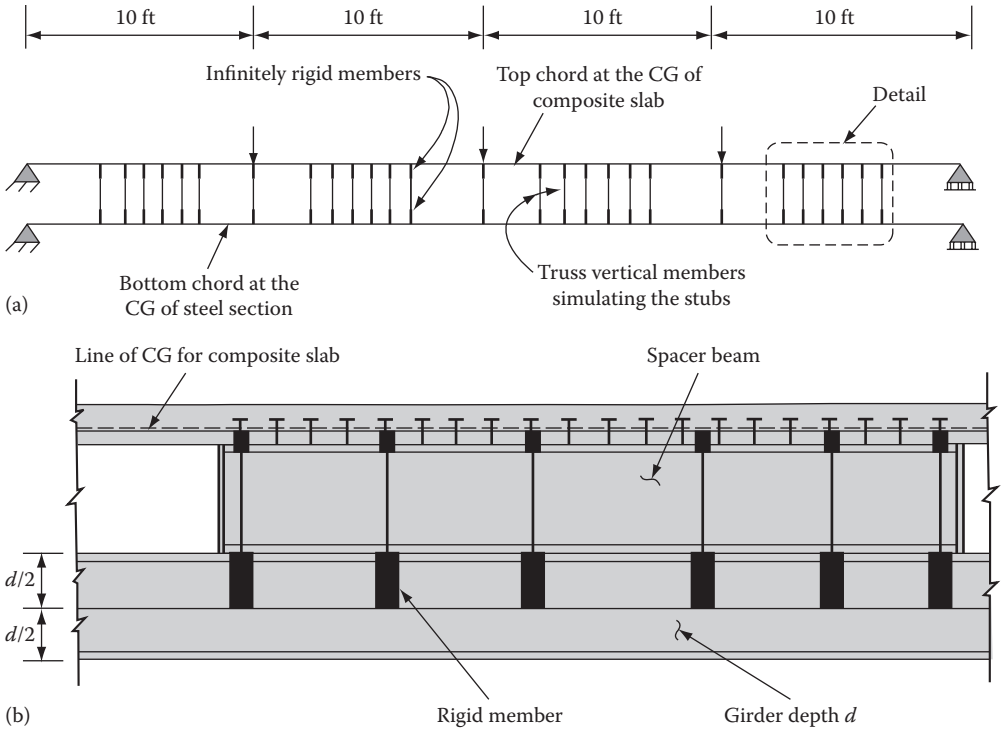


FIGURE 4.23 (a) Elevation of Vierendeel truss analytical model; (b) partial detail of model.

Note the beam elements representing these members are at the neutral axes of the slab and beam, as shown in Figure 4.23b.

The stub pieces are modeled as a series of vertical beam elements between the top and the bottom chords of the truss with rigid panel zones at the top and bottom.

The various steps of modeling of stub girder are summarized as follows:

1. *Top chord of Vierendeel truss*. As shown in Figure 4.23, the top chord consists of an equivalent transformed area of the concrete topping obtained by dividing the effective width of concrete slab by the modular ration $n = E_s/E_c$. In calculating the transformed properties, advantage can be taken by including the mild steel reinforcement used in the slab parallel to the stub girder. The moment of inertia I_t of the top chord is obtained by multiplying the unit value of I of the composite slab, given in deck manufacturer's catalog, by the effective width of slab.
2. *Bottom chord*. The properties of the steel section are directly used for the bottom chord properties.
3. *Stub pieces*. The web area and moment of inertia of the stub in the plane of bending of the stub girder are calculated and apportioned to a finite number of vertical beam elements representing the stubs. The more elements employed to represent the stub pieces, the better will be the accuracy of the solution. As a minimum, the author recommends one vertical element for 1 ft of stub width. The vertical segments between the top and bottom flanges of the stub and the neutral axes of the top and bottom chords are treated as an infinitely rigid member. Stiffener plates used at the ends of stubs can be incorporated in calculating the moment of inertia of the stubs.

Stub girder construction was quite popular in the 1970s and 1980s. The prevalent method for steel design at that time was Working Stress Design, WSD. One such example using WSD for steel members is as follows.

4.8.2 Stub Girder Design Example

(SG1, Span = 40 ft., Figure 4.22)

A 3¼ in. lightweight, 3 ksi concrete topping is used on 3 in. deep 18-gauge composite metal deck for the floor system. The steel purlins are spaced at 10 ft centers. A W14 × 53, 50 ksi structural steel beam is used for the bottom continuous chord of stub girder. The exterior and interior stub pieces are W16 × 26, 36 ksi steel beams with 6½ and 3½ ft lengths, respectively. Verify design of stub girder using a Vierendeel truss model.

As a first step, we compute the equivalent properties of (1) top chord; (2) bottom chord; and (3) stub pieces for setting up the stub girder model.

1. *Top chord*. The concrete slab extends on both sides of the stub girder SG1. The effective width, *b*, of concrete flange is determined by considering the following three values:

$$b = \text{distance between stub girders} = 30\,\text{ft}$$

$$b = 16t + b = 16(1.5 + 3.25) + 5 = 81\,\text{in.} = 6.75\,\text{ft}$$

$$b = L/4 = 40/4 = 10\,\text{ft}$$

For the example, the least value of effective width of the top chord = 6.75 ft.
Area *A* of transformed section of top chord using $n = 14$

$$A = (3.25 + 1.5) \times \frac{81}{14} = 27.48\,\text{in.}^2$$

The equivalent moment of inertia I_e of the top chord is equal to the moment of inertia *I* for the particular metal deck and slab thickness (given in the Manufacturer's Product Catalog), multiplied by the effective width of compression chord. Assuming that the moment of inertia of 3¼ in. slab on 3 in. composite deck as given in the catalog is 5.82 in.^{4}/ft, we get

$$I_e = 5.82 \times 6.75 = 39.3\,\text{in.}^4$$

2. *Bottom chord*.
W14 × 53 steel wide-flange beam has the following section properties:

$$A = 15.3\,\text{in.}^2 \quad I = 541\,\text{in.}^4$$

$$\text{Shear area } A_v = \text{depth of web} \times \text{web thickness}$$

$$= 13.92 \times 0.37 = 5.15\,\text{in.}^2$$

3(a). *Exterior stub piece*
W16 × 26, 6½ ft long

$$\text{cross section area of stub} = \text{length of stub} \times \text{web thickness of W16} \times 26$$

$$= 78 \times 0.25 = 19.5\,\text{in.}^2$$

Equivalent moment of inertia of stub girder

$$I = \frac{bh^3}{12} = \frac{0.25 \times 78^3}{12} = 9886\,\text{in.}^4$$

We divide the area and moment of inertia of the stub piece into the six elements. These are used in the computer model to represent the stub piece see Figure 4.23b. The resulting area of each vertical element is 19.5/6 = 3.25 in.^{2} and the moment of inertia I is 9886/6 = 1648 in.^{4}

3(b). *Interior stub piece*
W16 × 26, 3½ft long

$$A = 42 \times 0.25 = 10.5\,\text{in.}^2$$

$$I = \frac{0.25 \times 42^3}{12} = 1543.5\,\text{in.}^4$$

As before, we use six vertical members to represent the interior stub piece. See Figure 4.23a we divide the area and moment of inertia by 6 to get the equivalent values for the computer model.

$$A = \frac{10.5}{6} = 1.75\,\text{in.}^2$$

$$I = \frac{1548.5}{6} = 258.08\,\text{in.}^4$$

The next step is to set up a model of the equivalent Vierendeel frame to obtain computer results for the axial load, bending moment, and shear forces in the members. The adequacy of each member under the action of combined forces is checked using the ACI ultimate strength method and AISC WSD procedure. A brief description follows.

Bottom chord. Assume that the maximum axial tension and bending moment obtained from the computer analysis are T = 265 kip and M = 90 kip/ft, respectively. Check W14 × 53 for combined tension and moment; thus,

$$f_a = \frac{265}{15.3} = 17.2\,\text{ksi}$$

$$f_b = \frac{90 \times 12}{77.8} = 13.75\,\text{ksi}$$

$$F_a = 0.6F_y = 0.6 \times 50 = 30\,\text{ksi}$$

$$\frac{d}{t} = \frac{13.92}{0.37} = 37.62 > \frac{257}{\sqrt{F_y}} = 36.3$$

$$\therefore F_b = 0.60F_y = 30\,\text{ksi}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{17.2}{30} + \frac{13.75}{30} = 1.03$$

This is very nearly equal to 1.0 and therefore is OK.

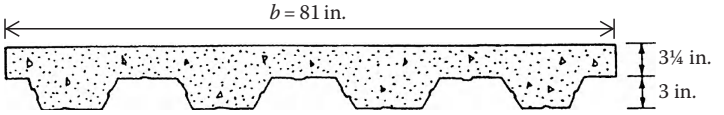


FIGURE 4.24 Cross section of equivalent compression chord.

Top chord. The top chord of the Vierendeel truss is subjected to compression and bending moment and therefore is designed as a reinforced concrete column subjected to compressive forces and bending. In the opinion of the author, any rational method that does not violate the spirit of the ACI 318 can be used in the design. One conservative procedure is to neglect the contribution of metal deck and design the slab section as an equivalent column. For purposes of calculation of moment magnification factor, the column can be conservatively assumed to have an effective length of 10 ft which is equal to the distance between the purlins.

For the example problem, assume that the computer results for axial compression and bending moment at critical sections are 250 kip and 10 kip-ft, respectively.

We now proceed to design the equivalent section of the compression chord shown in Figure 4.24 as a reinforced concrete column subjected to axial loads and bending moment. First, we calculate the slenderness ratio Kl_u/r by conservatively assuming a value of 1.0 for effective length factor K. The unsupported length l_u of the equivalent column as stated previously can be considered equal to 10 ft, the distance between the purlins. The radius of gyration for the equivalent rectangular column is 0.3 times the slab dimension in the direction of bending, that is, $0.3 \times 6.25 = 1.875$ in.

The slenderness ratio

$$\frac{Kl_u}{r} = \frac{1 \times 10 \times 12}{1.875} = 64.0$$

Since this ratio is greater than 22, it is necessary to consider slenderness effects in the design of column. The moment magnification procedure will be used to take into account the slenderness effects. A conservative approximation will be made by assuming that the value for the coefficient $C_m = 1.0$.

Since the axial load P and moment M obtained from the computer analysis are for working loads, these are converted to ultimate values by multiplying them with an average load factor of 1.5. Therefore,

$$P_u = 1.5 \times 260 = 390 \text{ kip}$$

$$M_u = 1.5 \times 10 = 15 \text{ kip-ft}$$

The critical axial load P_c is given by the relation

$$P_c = \frac{\pi^2 E_c I}{\left(Kl_u\right)^2}$$

For the example problem, we have

$$E_c = w_c^{1.5} \times 33\sqrt{f'_c} = 110^{1.5} \times 33\sqrt{3000} = 2085 \text{ ksi}$$

$$I = \frac{81 \times 6.25^3}{12} = 1648 \text{ in.}^4$$

Substituting, we get

$$P_c = \frac{3.14^2 \times 2085 \times 1648}{(10 \times 12)^2} = 2249\,\text{kip}$$

The moment magnification factor is given by

$$\delta_1 = \frac{C_m}{1 - (P_u/\phi P_C)}$$

$$= \left(\frac{1}{1 - \frac{390}{0.7 \times 2249}} \right) = 1.33$$

Therefore, design $M_u = 15 \times 1.33 = 19.95$ kip-ft.

The equivalent column is designed for $P_u = 390$ kip and $M_u = 19.95$ kip-ft. For the present example, ten #5 longitudinal reinforcement is found to be adequate to carry the design axial load and bending moment.

Computation of number of shear studs. The shear studs between the stub pieces and the concrete slab form the backbone of composite action of stub girders. They may be designed using the working stress method, WSD given in AISC manual.

4.8.3 Moment-Connected Stub Girder

The stub girder system, due to its relatively large overall depth of approximately 3 ft (0.92 m), has a fairly large moment of inertia and can therefore be used as part of a wind-resisting system. The analytical model used for analysis as before is a Vierendeel truss, where the concrete slab and the bottom steel beam are simulated as linear elements and each stub piece is divided into a number of equivalent vertical elements. The gravity and wind load shear forces and moments are introduced as additional load cases in the computer analysis, and the combined gravity and wind axial forces and moments in each section of the stub girder are obtained. All parts of the stub girder are checked for the combined axial forces, shear and moments using the procedure outlined for gravity loads. The controlling section for the slab is generally at the end of the first stub piece furthermost from the column. Particular care is required to transfer the moment at the column girder interface. If wind moments are small, moment transfer can take place between the slab and the steel beam. The slab needs to be attached to the column either by long deformed wire anchors or by welding reinforcing bars to the column. For relatively large moments, the solution for moment transfer is to extend the first stub piece to the column face. The top flange of the stub piece and the bottom flange of the beam are welded to the column as in a typical moment connection using full penetration welds. The design of the connection is, therefore, identical to welded beam–column moment connection. Depending upon the magnitude of stress due to reversal of wind load, bracing of the bottom chord may be necessary.

4.8.4 Strengthening of Stub Girder

Strengthening of existing stub girders for tenant-imposed loads higher than intended in the original design, is more expensive than in conventional composite construction. A speculative type of investment building is usually designed for imposed floor live loads of 50 psf plus 20 psf as a partition allowance. For heavier loads, strengthening of local framing is required. The bottom girder, which is in tension and bending, is relatively easy to reinforce by welding additional plates or angles to

the existing steel member. Reinforcing the top chord of the stub girder, which is in compression and bending, is somewhat tricky. Addition of structural steel angles by using expansion anchors to the underside of metal deck and welding of additional stub pieces to reduce the effective length of compression chord, which acts like a column, have been used in practice with good results. From the point of view of ultimate load behavior, it is acceptable to strengthen the bottom chord to resist to entire load without the truss action. However, it is important to check the lateral bracing requirements for the top flange of bottom chord.

4.9 COMPOSITE COLUMNS

The term "composite column" in building industry represents a unique form of construction in which structural steel interacts compositely with concrete. The steel section can be a tubular section filled with structural concrete or a central core element encased in reinforced concrete.

Historically, composite columns evolved from concrete encasement of structural steel shapes primarily intended as fire protection. Although the increase in strength and stiffness of steel members due to concrete encasement was intuitively known, it was not until the 1940s that composite design methods were developed. In fact, in the earlier days the design of the steel column was penalized by considering the weight of concrete as an additional dead load on the steel column. Later developments took into consideration the increased radius of gyration of columns because of concrete encasement, resulting in nominal reduction in structural steel weight. In some earlier high-rise designs, concrete encasement was ignored from strength considerations but the additional stiffness due to concrete was included in the lateral deflections calculations.

After the development of sprayed-on fireproofing in the 1950s and 1960s, use of concrete for fireproofing of structural steel was no longer an economical proposition. The high form-work cost of concrete could not be supported just for fireproofing. However, this plight was overcome if one could include the stiffness and strength properties of concrete in the design of columns.

Therefore, over the last 30 years, the use of encased steel columns has found applications in buildings varying from as low as 10 stories to as high as 70 stories or even taller.

Composite systems have successfully captured the traditional advantages steel and concrete construction: the speed of steel construction and the increased stiffness and moldability of concrete. Concrete columns with relatively small steel-cores used as erection columns were perhaps the earliest applications. Later, much heavier steel sections were used to limit the size of composite vertical elements.

4.9.1 Behavior

In a broad sense, the behavior of a composite column is a combination of the response of reinforced concrete and structural steel section. Generally, composite interaction is synergistic; that is, it enhances the performance of the whole to something superior to the sum of the component parts.

The basic function of *columns* is the delivery of gravity loads to the base of the structure. However, columns can be more than compression members. For example, frame columns that are connected to beams with moment resisting connections restrain the bending deflections of floor members as well as the lateral drift of the overall structure. Their design must be chosen to satisfy both axial and flexural demands.

The stability of slender columns is a measure of material stiffness rather than material strength. Steel, which has 5–8 times the stiffness and strength of concrete, is the more efficient structural material for slender columns. In building applications, structural columns are neither slender nor stocky. Instead, most are in the intermediate range between these two extremes.

Two basic types of composite columns are used in buildings: those with the steel section encased in concrete and those with the steel section filled with concrete.

Encased composite columns consist of structural shapes surrounded by structural concrete. The concrete encasement requires vertical and horizontal reinforcement to sustain the encasement of the steel core. Shear connectors may be needed as well to ensure interaction and force-transfer between the steel shape and the concrete encasement. The shear connectors transfer forces between the steel and concrete through attachment by welds to the steel shape, and by bearing against the surrounding concrete.

Filled composite columns are perhaps the most efficient application of the two materials for column cross sections. The steel shell can be a pipe, tubing or a hollow section fabricated from plates. The steel shell provides transverse confinement to the contained concrete, making the filled composite column ductile and tough to survive overloads. Since the concrete core is confined by the steel shell, interaction between the steel and concrete is assured. However, it may be desirable to provide additional bearing surfaces for shear transfer using welded studs or rebars particularly near the connections of the columns to the floor beams.

Design of composite columns for combined axial force and flexure is most often accomplished by the plastic distribution method. To assist in developing the load–moment interaction curve, a series of equations is provided in the AISC specifications for specific cases of reinforcement arrangement. When design cases deviate from these cases, the appropriate interaction equation can be derived from first principles. Slenderness effects are not directly considered in the derivation of interaction curves.

4.9.2 AISC Design Criteria, Encased Composite Columns

See Figure 4.25 for structural schematics.

4.9.2.1 Limitations

To qualify as an encased *composite column*, the following limitations shall be met:

1. The cross-sectional area of the steel core shall comprise at least 1% of the total composite cross section.
2. Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum *transverse reinforcement* shall be at least 0.009 in.^{2} per in. of tie spacing.

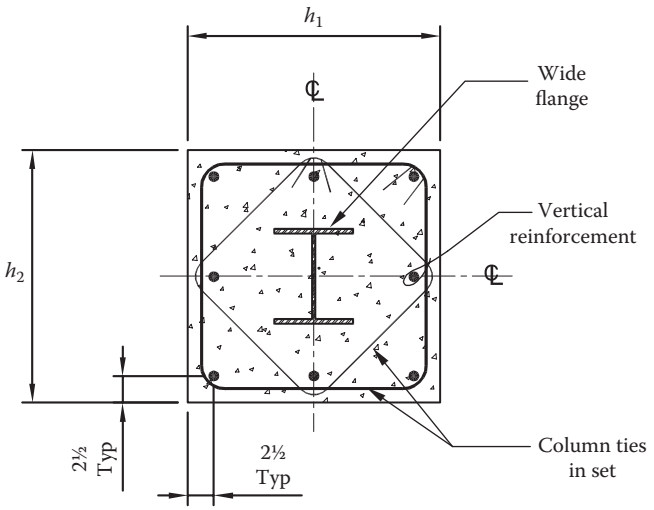


FIGURE 4.25 Encased composite column.

3. The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004, where ρ_{sr} is given by

$$\rho_{sr} = \frac{A_{sr}}{A_g}$$

where
A_{sr} is the area of continuous reinforcing bars, in.^{2}
A_g is the gross area of composite member, in.^{2}

4.9.2.2 Compressive Strength

The *design compressive strength*, $\phi_c P_n$, for axially loaded *encased composite columns* shall be determined for the limit state of *flexural buckling* based on column slenderness as follows:

$$\phi_c = 0.75$$

(a) When $P_e \geq 0.44P_o$

$$P_n = P_o \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right]$$

(b) When $P_e < 0.44P_o$

$$P_n = 0.877P_e$$

where

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c$$

$$P_e = \pi^2 (EI_{eff})/(KL)^2$$

and where
A_s = area of steel section, in.^{2}
A_c = area of concrete, in.^{2}
A_{sr} = area of continuous reinforcing bars, in.^{2}
E_c = modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f'_c}$, ksi
E_s = modulus of elasticity of steel = 29,000 ksi
f'_c = specified compressive strength of concrete, ksi
F_y = specified minimum yield stress of steel section, ksi
F_{yr} = specified minimum yield stress of reinforcing bars, ksi
I_c = moment of inertia of the concrete section, in.^{4}
I_s = moment of inertia of steel shape, in.^{4}
I_{sr} = moment of inertia of reinforcing bars, in.^{4}
K = effective length factor
L = laterally unbraced length of the member, in.
w_c = weight of concrete per unit volume ($90 \leq w_c \leq 155$ lb/ft^3)

where
EI_{eff} is the effective stiffness of composite section, kip-in.^{2}
$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c$

where

$$C_1 = 0.1 + \left(\frac{A_s}{A_c + A_s}\right) \leq 0.3$$

4.9.2.3 Tensile Strength

The design tensile strength, P_n is given by

$$P_n = \phi_t P_n = 0.90(A_s F_y + A_{sr} F_{yr})$$

4.9.2.4 Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone, plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone. The nominal shear strength of tie reinforcement may be determined as $A_{st}F_{yr}(d/s)$ where A_{st} is the area of tie reinforcement, d is the effective depth of the concrete section, and s is the spacing of the tie reinforcement. The shear capacity of reinforced concrete may be determined according to ACI 318-08.

4.9.2.5 Load Transfer

Loads applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

(a) When the external *force* is applied directly to the steel section, *shear connectors* shall be provided to transfer the required shear force, V', as follows:

$$V' = V(1 - A_s F_y / P_o)$$

where
V is the required shear force introduced to column, kip
A_s is the area of steel cross section, in.^{2}
P_o is the nominal axial compressive strength without consideration of *length effects,* kip

(b) When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the required shear force, V', as follows:

$$V' = V(A_s F_y / P_o)$$

(c) When load is applied to the concrete of an encased composite column by direct bearing the *design bearing strength,* $\phi_B P_p$, *of the concrete shall be*

$$P_p = 1.7 f'_c A_B$$

$$\phi_B = 0.65$$

where A_B is the loaded area of concrete, in.^{2}

4.9.2.6 Detailing Requirements

At least four continuous longitudinal reinforcing bars shall be used in encased composite columns. *Transverse reinforcement* shall be spaced at the smallest of 16 longitudinal bar diameters, 48 tie bar diameters or 0.5 times the least dimension of the composite section. The encasement shall provide at least 1.5 in. of clear cover to the reinforcing steel.

Shear connectors shall be provided to transfer the required shear force and shall be distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region. The maximum connector spacing shall be 16 in. Connectors that transfer axial load shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

If the composite cross section is built up from two or more encased steel shapes, the shapes shall be interconnected with *lacing, tie plates, batten plates*, or similar components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

4.9.2.7 Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete is

$$Q_n = 0.5A_{sc}\sqrt{f'_cE_c} \le A_{sc}F_u$$

where

A_{sc} is the cross-sectional area of stud shear connector, in.^{2}

F_u is the *specified minimum tensile strength* of a stud shear connector, ksi

4.9.3 AISC Design Criteria for Filled Composite Columns

See Figure 4.26 for structural schematics.

4.9.3.1 Limitations

To qualify as a filled *composite column* the following limitations shall be met:

1. The cross-sectional area of the steel *HSS* shall comprise at least 1% of the total composite cross section.
2. The maximum *b*/*t* ratio for a rectangular HSS used as a composite column shall be equal to $2.26\sqrt{E/F_y}$. Higher ratios are permitted when their use is justified by testing or analysis.
3. The maximum *D*/*t* ratio for a round HSS filled with concrete shall be $0.15E/F_y$.

4.9.3.2 Compressive Strength

The design compressive strength, $P_c = \phi_c P_n$, for axially loaded filled composite columns shall be determined for the *limit state* of *flexural buckling* based on the following formula:

$$P_o = A_sF_y + A_{sr}F_{yr} + C_2A_cf'_c$$

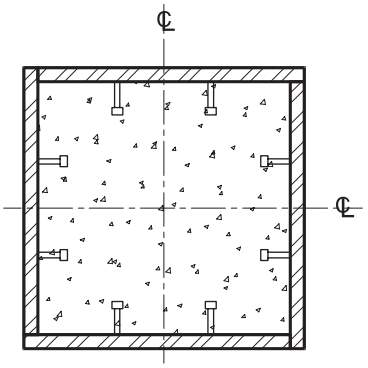


FIGURE 4.26 Filled composite column.

$C_2 = 0.85$ for rectangular sections and 0.95 for circular sections

$$EI_{\text{eff}} = E_s I_s + E_s I_{sr} + C_3 E_c I_c$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9$$

4.9.3.3 Tensile Strength

The *design tensile strength* $P_n = \phi_t P_n$, for filled composite columns shall be determined as:

$$P_t = \phi_t P_n = 0.90(A_s F_y + A_{sr} F_{yr})$$

$$\text{Note } \phi_t = 0.9$$

4.9.3.4 Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone or the shear strength of the reinforced concrete portion alone. The shear strength of reinforced concrete may be determined using ACI 318-08, Chapter 11.

4.9.3.5 Load Transfer

Loads applied to filled composite columns shall be transferred between the steel and concrete. When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core is required through *bond interaction,* shear *connection* or direct bearing. The force transfer *mechanism* providing the largest *nominal strength* may be used. These force transfer mechanisms shall not be superimposed.

When load is applied to the concrete of an encased or filled composite column by direct bearing the *design bearing strength,* $\phi_B P_p$, of the concrete shall be

$$\phi_B P_p = 0.65 \times 1.7 f'_c A_B$$

$$\text{Note } \phi_B = 0.65$$

where A_B is the loaded area, in.^{2}

4.9.4 Summary of Composite Design Column

Design of composite sections requires consideration of both steel and concrete behavior. The AISC specification uses a strength approach for design consistent with that used in reinforced concrete design, and assumes that the user is familiar with reinforced concrete design specifications needed for the concrete portion of the design, such as material specifications, anchorage and splice lengths, and shear and torsion provisions.

4.9.4.1 Nominal Strength of Composite Sections

The strength of composite sections may be computed based on the plastic distribution approach. The plastic stress distribution method is based on the assumption of linear strain across the cross section and elasto-plastic behavior. It assumes that the concrete has reached its crushing strength in compression at a strain of 0.003 and a corresponding stress (typically $0.85 f'_c$) on a rectangular stress block, and that the steel has exceeded its yield strain, typically taken as F_y/E_s.

Based on these simple assumptions, the cross-sectional strength for different combinations of axial force and bending moment may be approximated, for typical composite column cross sections.

The actual interaction diagram for moment and axial force for a composite section is thus similar to that of a reinforced concrete section. As a simplification, a conservative linear interaction between selected anchor points, depending on axis of bending, can be used.

The plastic stress approach for columns assumes that no slip has occurred between the steel and concrete portions and that the required width-to-thickness ratios prevent local buckling from occurring until extensive yielding has taken place.

As for reinforced concrete design, a limit of 10 ksi is imposed for strength calculations. A lower limit of 3 ksi is specified for both normal and lightweight concrete and an upper limit of 6 ksi specified for lightweight concrete. The use of higher strengths in computing the modulus of elasticity is permitted.

The design of concrete-encased and concrete-filled composite columns is treated separately, although they have much in common. The design for length effects is consistent with that for steel columns. The equations used are the same as in steel design, albeit in a slightly different format, and as the percent of concrete in the section decreases, the design defaults to that of a steel section.

4.9.4.2 Encased Composite Columns

4.9.4.2.1 Limitations

1. Composite columns may be designed with as little as 1% steel ratio (area of steel shape divided by the gross area of column).
2. The specified minimum quantity for transverse reinforcement is intended to provide good confinement to the concrete.
3. A minimum amount of longitudinal reinforcing steel is prescribed so that at least four continuous corner bars are used. Other longitudinal bars may be needed to provide the required restraint to the cross-ties, but that longitudinal steel cannot be counted toward the cross-sectional strength unless it is continuous and properly anchored.

4.9.4.2.2 Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The strength is not capped as in reinforced concrete column design for a combination of the following reasons:

1. The resistance factor has been lowered from 0.85 in previous editions to 0.75 in this specification.
2. The required transverse steel provides better performance than a typical reinforced column.
3. The presence of a steel section near the center of the section reduces the possibility of a sudden failure due to bulking of the longitudinal steel.

4.9.4.2.3 Shear Strength

The provisions require either the use of the steel section alone plus the contribution from any transverse shear reinforcement present in the form of ties or the shear strength calculated based on the reinforced concrete portion of the cross-section alone (in other words, longitudinal and transverse reinforcing bars plus concrete). This implies the following shear strengths:

$$V_n = 0.6F_yA_w + A_{st}F_{yr}\frac{d}{s}$$

$$\phi = 0.9$$

or

$$V_n = 2\sqrt{f_c'}bd + A_{st}F_{yr}\frac{d}{s}$$

$$\phi = 0.75$$

4.9.4.2.4 Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in encased composite columns, a transfer of load by direct bearing, shear connection, or a combination of both is required. Although it is recognized that force transfer also occurs by direct bond interaction between the steel and concrete, this is typically ignored for encased composite columns.

When the shear connectors are used in encased composite columns, a uniform spacing is appropriate in most situations, but when large forces are applied, other connector arrangements may be needed to avoid overloading the component to which the load is applied directly.

When supporting concrete area is wider on all sides than the loaded area, the nominal bearing strength for concrete may be taken as

$$N_b = 0.85f_c'\sqrt{A_2/A_1}$$

where

A_1 is the loaded area

A_2 is the maximum area of the supporting surface that is geometrically similar and concentric with the loaded area

The value of $\sqrt{A_2/A_1}$ must be less than or equal to 2. This specification uses the maximum nominal bearing strength of $1.7f_c'A_B$. The resistance factor for bearing, ϕ_B, is 0.65 in accordance with ACI. 318-08.

4.9.4.3 Filled Composite Columns

4.9.4.3.1 Limitations

1. As discussed for encased columns, it is now permissible to design composite columns with a steel ratio as low as 1%.
2. The specified minimum wall slenderness now takes into account the restraining effect of the concrete on the local buckling of the section wall.

4.9.4.3.2 Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The beneficial confining effect of the steel shell can be taken into account by increasing the crushing strength of the concrete to $0.95f_c'$.

4.9.4.3.3 Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in filled composite columns, a transfer of load by direct bearing, shear connection, or direct bond interaction is permitted, with the mechanism providing the largest resistance being permissible for use. However, superposition of these force transfer mechanisms is not permitted for filled composite columns.

Force transfer by direct bond is commonly used in filled composite columns as long as the connections are detailed to limit local deformations.

4.9.5 Combined Axial Force and Flexure

One of the three permitted methods in the AISC Specifications is the plastic distribution method for determining the combined effect of axial force, P_u, and moment M_u. This method is not unlike the method we use in everyday practice to generate the $P_u - M_u$ interaction diagrams for reinforced concrete columns.

For an encased composite column, perhaps the most general and fundamental approach is to treat the steel section as an equivalent reinforcement placed at discrete locations within concrete section. Thus, the generation of the $P_u - M_u$ interaction diagram for a composite column, reverts back to that for a reinforced concrete column.

Shown in Figure 4.27 is an interaction diagram obtained by using such an approach. The interaction diagram is for a 36 × 36 in. column with twelve #18 reinforcing bars and a W14 × 150 structural steel shape. For comparison purposes, the interaction diagram for the same concrete column without the embedded structural steel shape is given.

However, to assist in developing the interaction curves without an elaborate discreditization of the steel section, the AISC Specification provides a series of simplified equations for calculating the $P_u - M_u$ values on the interaction curve. The defining points on the interaction curve are shown as A, B, C, D, and E in Figure 4.28. The reader is referred to the AISC Commentary Sections for further details.

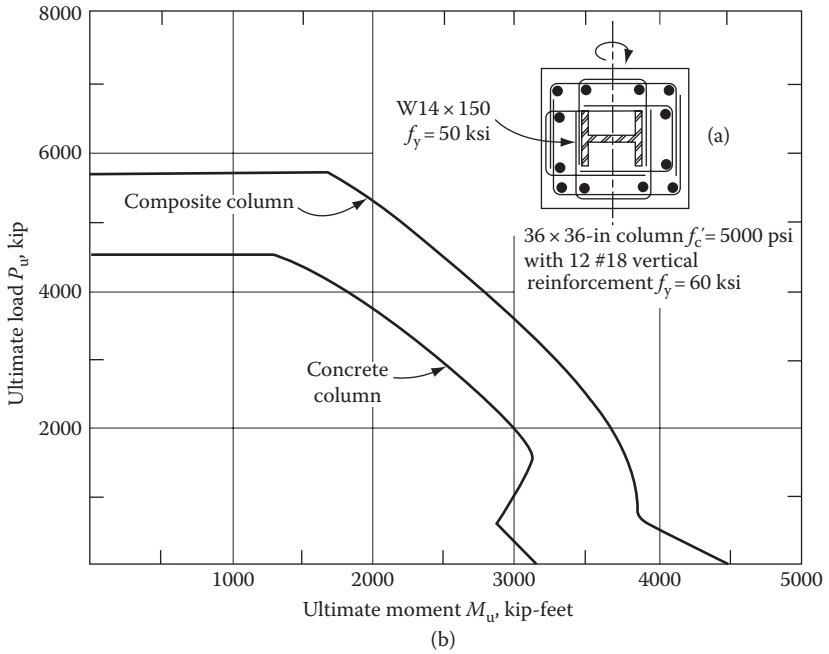


FIGURE 4.27 Composite column interaction diagram: (a) column detail; (b) interaction diagram.

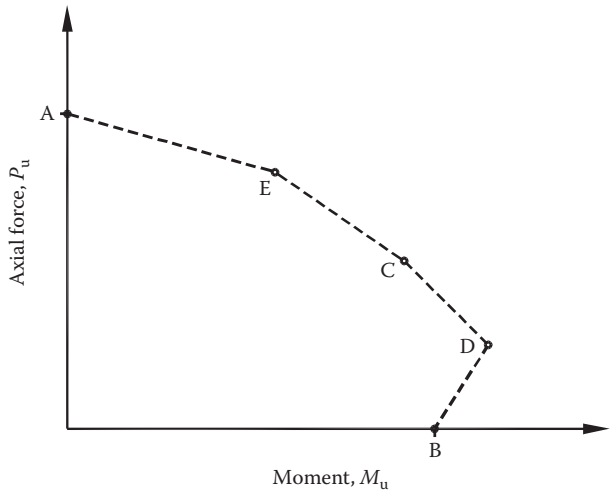


FIGURE 4.28 Interaction diagram for beam-column design.

5 Wind Loads

PREVIEW

Wind is air in motion. Obstacles in the path of wind—such as buildings and other topographic features—deflect or stop wind, converting the wind's kinetic energy into potential energy of pressure, thereby creating *wind load*.

In this chapter, we study how lateral pressures on a building are affected by the velocity of wind, its gradient profile, topographic effects such as hills and escarpments, and the shape and surface features of the building itself. Next, we study the probabilistic approach used to quantify the random characteristics of wind-induced turbulence and vortex shedding. Subsequent to this, wind load provisions of ASCE 7-05 are studied including dynamic effects due to gust buffeting and vortex shedding. Presented also are methods for determining along-wind displacements and accelerations. Next, wind-tunnel test procedures are discussed highlighting criteria used for limiting wind-induced accelerations to minimize human response to building motions.

5.1 DESIGN CONSIDERATIONS

In designing for wind, a building cannot be considered independent of its surroundings because configuration of nearby buildings and natural terrain has substantial influence on design loads. The sway response of a building, due to wind, defined as the horizontal displacement at top, may not be seen by a passerby, but is of concern in the design of tall buildings. There is scant evidence that winds, except those due to a tornado or hurricane, have caused major structural damage to buildings. The majority of buildings have sufficient strength and stiffness to control their response with respect to the ultimate strength limit state. The challenge then is to control the serviceability limit state of modern skyscrapers that are built using lightweight curtain walls, dry partitions, and high-strength materials. Thus, they are more prone to wind-related problems than their earlier brethren, which had the weight and higher damping advantage of heavy masonry partitions, stone facades, and massive structural members.

To be sure, all buildings sway during windstorms, but the motion in earlier tall buildings with heavy interior partitions and exterior cladding that resulted in higher damping, has usually been imperceptible and, therefore, has not been a cause for concern. Structural innovations coupled with lightweight construction have reduced the stiffness, mass, and damping characteristics of modern buildings. In these buildings, objects may vibrate, doors and chandeliers may swing, pictures may lean, and books may fall off shelves. Additionally, if the building has a twisting action, its occupants may get an illusory sense that the world outside is moving, creating symptoms of vertigo and disorientation. In more violent storms, windows may break, creating safety problems for pedestrians below. Sometimes, strange and frightening noises may be heard by occupants as the wind shakes elevators, strains floors and walls, and whistles around the building sides.

Although one cannot see wind, we know by experience that its flow is quite random and turbulent. Imagine taking a walk on a windy day. You will no doubt experience a constant flow of wind, but intermittently you may also experience sudden gusts of rushing wind. This sudden variation in wind speed, called gustiness or *turbulence*, is an important factor in determining dynamic response of tall buildings.

Unlike steady flow of wind, which for design purposes is considered static, turbulent wind loads associated with gustiness cannot be treated in the same manner. This is because gusty wind

velocities change rapidly and even abruptly, creating effects much larger than if the same loads were static. Wind loads, therefore, need to be studied as if they were dynamic, somewhat similar to seismic loads. The intensity of dynamic component of wind load depends on how fast the velocity varies and also on the response of the structure itself. Therefore, whether pressure on a building due to a wind gust is dynamic or static entirely depends on the gustiness of wind and the dynamic properties of the building to which it is applied.

Consider, for example, the lateral movement of an 800 ft tall building designed for a drift index of *H*/400, subjected to a wind gust. Under wind loads, the building bends slightly as its top moves. It first moves in the direction of wind, with a magnitude of, say, 2 ft (0.61 m), and then starts oscillating back and forth. After moving in the direction of wind, the top goes through its neutral position, then moves approximately 2 ft (0.61 m) in the opposite direction, and continues oscillating back and forth progressively with smaller drifts, until it eventually stops. The time it takes a building to cycle through a complete oscillation is known as *period* of the building. The period of oscillation for a tall steel building in the height range of 700–1400 ft (214–427 m) normally is in the range of 10–15 s, whereas for a 10-story concrete or masonry building it may be in the range of 0.5–1 s. Let us consider the dynamic effects of wind on these two buildings. As stated earlier, the action of a wind gust depends not only on how long it takes for the gust to reach its maximum intensity and decrease again, but on the period of the building also. If the wind gust reaches its maximum value and vanishes in a time much shorter than the period of the building, its effects are dynamic. On the other hand, the effects can be considered as static if the wind load increases and vanishes in a time much longer than the period of the building. For example, a wind gust that develops to its strongest intensity and decreases to zero in 2 s is a dynamic load for a tall building with a period of considerably larger than 2 s, but the same 2 s gust is a static load for a low-rise building with a period of less than 2 s.

Every structure has a natural frequency of vibration. Should dynamic loading occur at or near its natural frequency, structural damage that is way out of proportion to size of load may result. It is well known, for example, that bridges capable of carrying far greater loads than the weight of a company of soldiers may oscillate dangerously and may even break down under dynamic loading of soldiers marching over them in step. Similarly, certain periodic gust within the wide spectrum of gustiness in wind may find resonance with natural vibration frequency of a building, and although the total force caused by that particular gust frequency would be much less than the static design load for the building, dangerous oscillations may be set up. This applies not only to the structure as a whole, but also to components such as curtain wall panels and sheets of glass.

A second dynamic effect is caused by instability of flow around certain type of structures. Long narrow structures such as smoke stacks, light standards, and suspension bridges are particularly susceptible to this sort of loading, causing an alternating pattern of eddies that form in its wake. A side thrust is thus exerted on the structure similar to the lift on an aerofoil, and since this thrust alternates in direction, a vibration perpendicular to the direction of wind flow, commonly referred to as, vortex shedding, may result. Side-to-side wobbling effect of a straight stick pulled through water is an example of this phenomenon.

5.2 VARIATION OF WIND VELOCITY WITH HEIGHT (VELOCITY PROFILE)

The roughness of the earth's surface due to friction causes drag on wind flow. Therefore, wind speed at the surface is much less than speeds at higher levels. The effect of frictional drag gradually decreases with height, and at gradient level (around 1000–2000 ft) frictional drag effect becomes negligible. For strong winds, the shape of the vertical profile of wind speed depends mainly on the degree of surface roughness, including the overall drag effects of buildings, trees, and any other projections that impede flow of wind at the surface.

The height at which the drag effects cease to exist is called *gradient height*, and the corresponding velocity, *gradient velocity*. This characteristic increase of wind velocity with height is a

well-understood phenomenon, as evidenced by higher design pressures specified at higher elevations in most building codes.

At heights of approximately 1200 ft (366 m) above ground, wind speed is virtually unaffected by surface friction. Its movement at and above this level is solely a function of seasonal and local wind effects. The height through which the wind speed is affected by topography is called the *atmospheric boundary layer.*

The wind speed profile within the atmospheric boundary layer is given by

$$V_z = V_g(z/z_g)^{1/\alpha} \tag{5.1}$$

where

V_z is the mean wind speed at height z above ground
V_g is the gradient wind speed assumed constant above the boundary layer
z is the height aboveground
z_g is the nominal height of boundary layer also referred to as gradient height
α is the power law coefficient

See Figure 5.1 for schematics of mean wind profiles, values of z_g and α and variation of gust speed.

With known values of mean wind speed at gradient height and exponent α, wind speeds at height z are calculated by using Equation 5.1. The exponent $1/\alpha$ and the depth of boundary layer z_g vary with terrain roughness and the averaging time used in calculating wind speed. The value of α ranges from a low of 0.087 for open country to 0.20 for built-up urban areas, signifying that wind speed reaches its maximum value over a greater height in an urban terrain than in the open country.

5.3 PROBABILISTIC APPROACH

In many engineering sciences, the intensity of certain events is considered to be a function of the duration recurrence interval (return period). For example, in hydrology the intensity of rainfall expected in a region is considered in terms of a return period. This is to take into consideration the well understood fact that the rainfall expected for a shorter return period, say once in 10 years is less than the one expected for a longer return period, say once every 50 years. Similarly, in wind engineering, the speed of wind is considered to vary with return periods. For example, the nominal design 3 s gust wind speed, for Exposure C, at 33 ft (10 m) above ground in Dallas, TX, corresponding to a 50 year return period, is 90 mph (40 m/s), compared to the value of 96 mph (43 m/s) for a 100 year recurrence interval.

A 50 year return-period wind of 90 mph (40 m/s) means that on the average, Dallas will experience a wind faster than 90 mph within a period of 50 years. A return period of 50 years corresponds to a probability of occurrence of 1/50 = 0.02 = 2%. Thus, the chance that a wind exceeding 90 mph (40 m/s) will occur in Dallas within a given year is 2%. Suppose a building is designed for a 100 year lifetime using a design wind speed of 90 mph. What is the probability that this wind will exceed the design speed within the lifetime of the structure? The probability that this wind speed will not be exceeded in any year is 49/50. The probability that this speed will not be exceeded 100 years in a row is $(49/50)^{100}$. Therefore, the probability that this wind speed will be exceeded at least once in 100 years is

$$1 - (49/50)^{100} = 0.87 = 87\%$$

This signifies that although a wind with low annual probability of occurrence (such as a 50 year wind) is used to design structures, there still exists a high probability of the wind being exceeded within the lifetime of the structure. However, in structural engineering practice, it is believed that the actual probability of overstressing a structure is much less because of the factors of safety and the generally conservative values used in design.

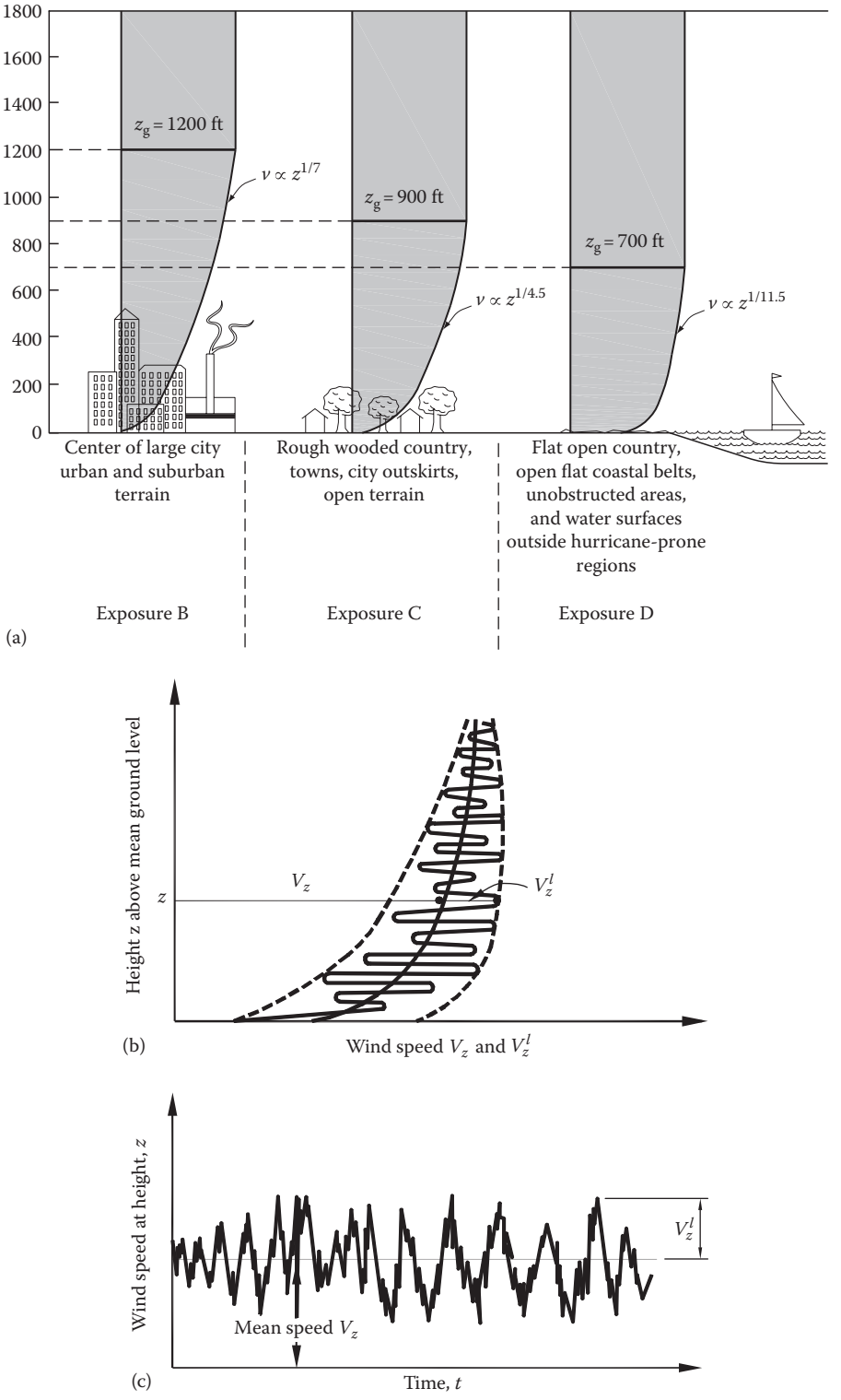


FIGURE 5.1 Wind velocity profiles: (a) ASCE 7-05 wind velocity profiles; (b) variation of mean and gust speed versus height; (c) variation of mean and gust speed over time *t*. *Note*: V_z = mean wind (also denoted as *v*), V_z^l = gust speed.

TABLE 5.1
Probability of Exceeding Design Wind Speed during Design Life of Building

Annual Probability P_a	Mean Recurrence Interval ($1/P_a$) Years	Exposure Period (Design Life), n (Years)					
		1	5	10	25	50	100
0.1	10	0.1	0.41	0.15	0.93	0.994	0.999
0.04	25	0.04	0.18	0.34	0.64	0.87	0.98
0.034	30	0.034	0.15	0.29	0.58	0.82	0.97
0.02	50	0.02	0.10	0.18	0.40	0.64	0.87
0.013	75	0.013	0.06	0.12	0.28	0.49	0.73
0.01	100	0.01	0.05	0.10	0.22	0.40	0.64
0.0067	150	0.0067	0.03	0.06	0.15	0.28	0.49
0.005	200	0.005	0.02	0.05	0.10	0.22	0.39

It is important to understand the notion of probability of occurrence of design wind speeds during the service life of buildings. The general expression for probability P that a design wind speed will be exceeded at least once during the exposed period of n years is given by

$$P = 1 - (1 - P_a)^n$$

where

P_a is the annual probability of being exceeded (reciprocal of the mean recurrence interval)
n is the exposure period in years

Consider a building in Dallas designed for a 50 year service life instead of 100 years. The probability of exceeding the design wind speed at least once during the 50 year lifetime of the building is

$$P = 1 - (1 - 0.02)^{50} = 1 - 0.36 = 0.64 = 64\%$$

The probability that wind speeds of a given magnitude will be exceeded increases with a longer exposure period of the building and the mean recurrence interval used in the design. Values of P for a given mean recurrence interval and a given exposure period are shown in Table 5.1.

5.4 VORTEX SHEDDING

In general, wind buffeting against a bluff body is diverted in three mutually perpendicular directions, giving rise to these sets of forces and moments. In aeronautical engineering, all six components are significant. However, in civil and structural engineering the force and moment corresponding to the vertical axis (lift and yawing moment) are of little significance. Therefore, aside from the effects of uplift forces on large roof areas, flow of wind is considered two-dimensional (2D), as shown in Figure 5.2, consisting of *along wind* and *transverse wind*.

The term along wind—or simply wind—is used to refer to drag forces while transverse wind is the term used to describe crosswind. Generally, in tall building design, the crosswind motion perpendicular to the direction of wind is often more critical than along-wind motion.

Consider a prismatic building subjected to a smooth wind flow. The originally parallel upwind streamlines are displaced on either side of the building, as illustrated in Figure 5.3. This results in spiral vortices being shed periodically from the sides into the downstream flow of wind. At relatively low wind speeds of, say, 50–60 mph (22.3–26.8 m/s), the vortices are shed symmetrically in pairs, one from each side. When the vortices are shed, that is, break away from the surface of the building, an impulse is applied in the transverse direction.

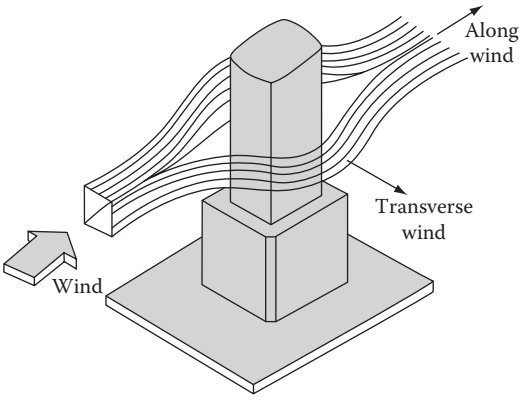


FIGURE 5.2 Simplified 2D wind flow consisting of along-wind and across-wind.

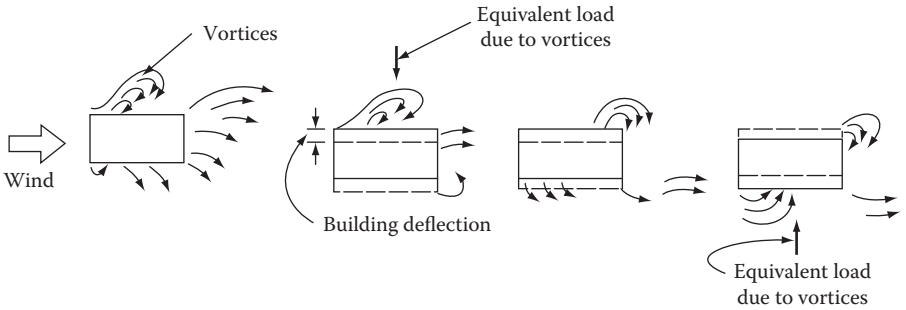


FIGURE 5.3 Vortex shedding: Periodic shedding of vertices generates building vibrations transverse to the direction of wind.

At low wind speeds, since the shedding occurs at the same instant on either side of the building, there is no tendency for the building to vibrate in the transverse direction. Therefore, the building experiences only along-wind oscillations parallel to wind direction. However, at higher speeds, vortices are shed alternately, first from one and then from the other side. When this occurs, there is an impulse in the along-wind direction as before, but in addition, there is an impulse in the transverse direction. However, the transverse impulse occurs alternately on opposite sides of the building with a frequency that is precisely half that of the along-wind impulse. This impulse due to transverse shedding gives rise to vibrations in the transverse direction. The phenomenon is called *vortex shedding* or *Karman vortex* street, terms well known in the field of fluid mechanics.

There is a simple formula to calculate the frequency of the transverse pulsating forces caused by vortex shedding:

$$f = \frac{V \times S}{D} \tag{5.2}$$

where

f is the frequency of vortex shedding in hertz
V is the mean wind speed at the top of the building
S is a dimensionless parameter called the Strouhal number for the shape
D is the diameter of the building

In Equation 5.2, the parameters V and D are expressed in consistent units such as ft/s and ft, respectively.

The Strouhal number is not a constant but varies irregularly with wind velocity. At low air velocities, S is low and increases with the velocity up to a limit of 0.21 for a smooth cylinder. This limit is reached for a velocity of about 50 mph (22.4 m/s) and remains almost a constant at 0.20 for wind velocities between 50 and 115 mph (22.4 and 51 m/s).

Consider for illustration purposes, a circular prismatic-shaped high-rise building having a diameter equal to 110 ft (33.5 m) and a height-to-width ratio of 6 with a natural frequency of vibration equal to 0.16 Hz. Assuming a wind velocity of 60 mph (27 m/s), the vortex-shedding frequency is given by

$$f = \frac{V \times 0.2}{110} = 0.16\text{Hz}$$

where V is in ft/s.

If the wind velocity increases from 0 to 60 mph (27.0 m/s), the frequency of vortex excitation will rise from 0 to a maximum of 0.16 Hz. Since this frequency happens to be very close to the natural frequency of the building, and assuming very little damping, the structure would vibrate as if its stiffness were zero at a wind speed somewhere around 60 mph (27 m/s). Note the similarity of this phenomenon to the ringing of church bells or the shaking of a tall lamppost whereby a small impulse added to the moving mass at each end of the cycle greatly increases the kinetic energy of the system. Similarly, during vortex shedding an increase in deflection occurs at the end of each swing. If the damping characteristics are small, the vortex shedding can cause building displacements far beyond those predicted on the basis of static analysis.

When the wind speed is such that the shedding frequency becomes approximately the same as the natural frequency of the building, a resonance condition is created. After the structure has begun to resonate, further increases in wind speed by a few percent will not change the shedding frequency, because the shedding is now controlled by the natural frequency of the structure. The vortex-shedding frequency has, so to speak, locked in with the building natural frequency. When the wind speed increases significantly above that causing the lock-in phenomenon, the frequency of shedding is again controlled by the speed of the wind. The structure vibrates with the resonant frequency only in the lock-in range. For wind speeds either below or above this range, the vortex shedding will not be critical.

Vortex shedding occurs for many building shapes. The value of S for different shapes is determined in wind-tunnel tests by measuring the frequency of shedding for a range of wind velocities. One does not have to know the value of S very precisely because the lock-in phenomenon occurs within a range of about 10% of the exact frequency of the structure.

Unlike steady flow of wind, which for design purposes is considered static, turbulent wind loads associated with gustiness cannot be treated in the same manner. This is because gusty wind velocities change rapidly and even abruptly, creating effects much larger than if the same loads were static. Wind loads, therefore, need to be studied as if they were dynamic, somewhat similar to seismic loads. The intensity of dynamic component of wind load depends on how fast the velocity varies and also on the response of the structure itself. Therefore, whether pressures on a building due to a wind gust, is dynamic or static entirely depends on the gustiness of wind and the dynamic properties of the building to which it is applied.

Consider, for example, the lateral movement of an 800 ft tall building designed for a drift index of H/400, subjected to a wind gust. Under wind loads, the building bends slightly as its top moves. It first moves in the direction of wind, with a magnitude of, say, 2 ft (0.61 m), and then starts oscillating back and forth. After moving in the direction of wind, the top goes through its neutral position, then moves approximately 2 ft (0.61 m) in the opposite direction, and continues oscillating back and forth progressively with smaller drifts, until it eventually stops. The time it takes a building to cycle through a complete oscillation is known as *period* of the building. The period of oscillation for a tall steel building in the height range of 700–1400 ft (214–427 m) normally is in the range of 10–15 s, whereas for a 10-story concrete or masonry building it may be in the range of 0.5–1 s. The action of a wind gust depends not only on how long it takes for the gust to reach its maximum intensity and decrease

again, but on the period of the building also. If the wind gust reaches its maximum value and vanishes in a time much shorter than the period of the building, its effects are dynamic. On the other hand, the gusts can be considered as static loads if the wind load increases and vanishes in a time much longer than the period of the building. For example, a wind gust that develops to its strongest intensity and decreases to zero in 2 s is a dynamic load for a tall building with a period of considerably larger than 2 s, but the same 2 s gust is a static load for a low-rise building with a period of less than 2 s.

5.5 ASCE 7-05 WIND LOAD PROVISIONS

The full title of this standard is American Society of Civil Engineers *Minimum Design Loads for Buildings and Other Structures*. In one of its 23 Chapters, Chapter 6, ASCE 7-05 gives three procedures for calculating wind loads for the design of buildings main wind-force-resisting systems, and components and cladding.

The methods are as follows:

1. Method 1—Simplified method
2. Method 2—Analytical procedure
3. Method 3—Wind-tunnel procedure

Method 1 applicable to regular-shaped "simple diaphragm buildings," has many constraints on its applicability, and is restricted to 60 ft maximum in height. The designer can use the simplified procedure to select wind pressures directly without calculations when the building is less than 60 ft in height and meets all requirements given in the ASCE Sections 6.4.1.1 and 6.4.1.2. Method 2, the analytical procedure, may be used for buildings of any height that are reasonably regular in shape, provided they are not sensitive to across-wind loading, vortex shedding, or instability due to galloping or flutter; or do not have a site for which channeling effects warrant special consideration. Method 3 is wind-tunnel procedure that may be used for any building and is recommended for buildings that exhibit the following characteristics:

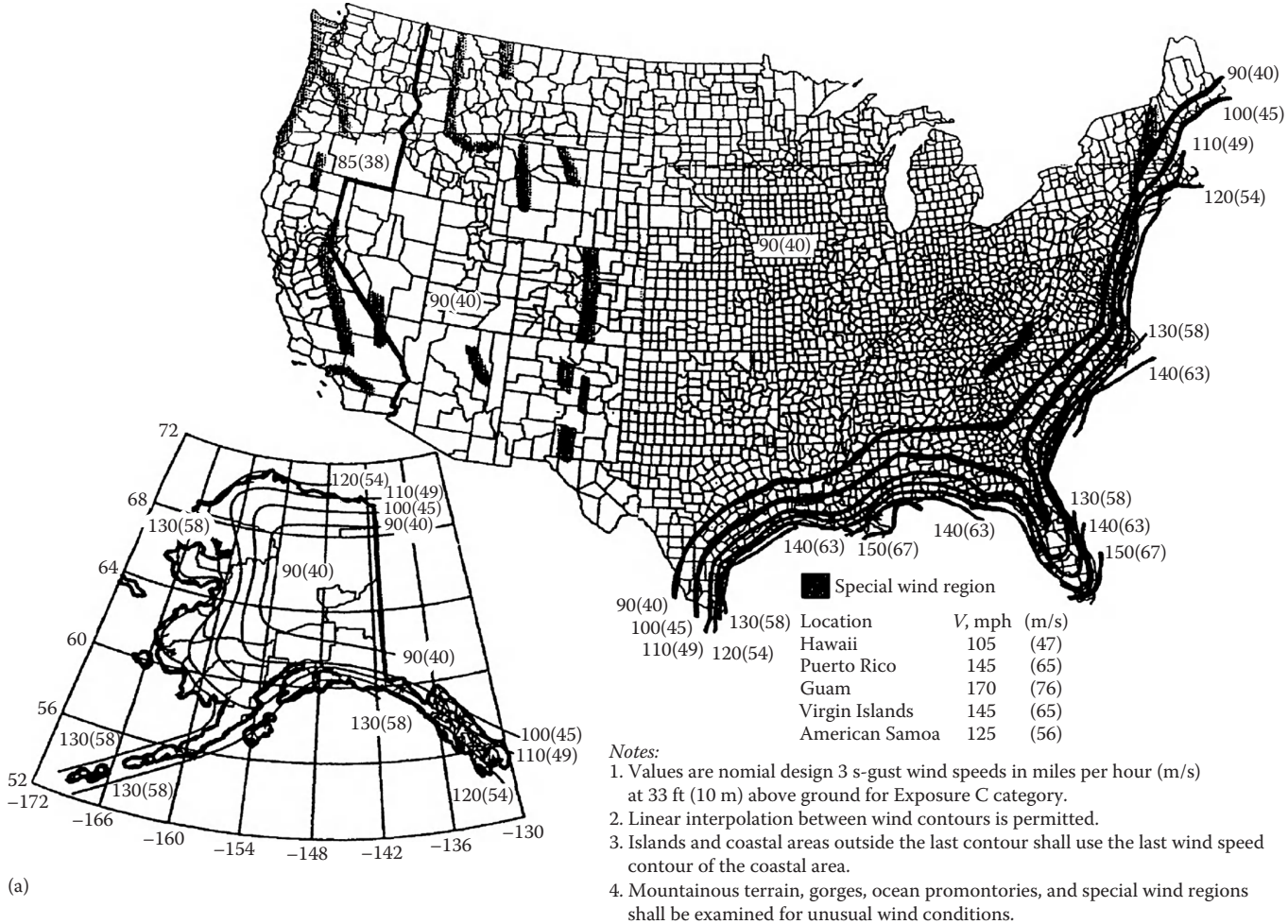
- Have nonuniform shapes
- Are flexible with natural frequencies less than 1 Hz
- Are subject to significant buffeting by the wake of upwind buildings or other structures
- Are subject to accelerated flow of wind by channeling or local topographic features

Basic wind speeds for any location in the continental United States and Alaska are shown on a map of having isotachs (lines of equal pressure) representing a 3 s gust speed at 33 ft (10 m) above the ground (see Figure 5.4). For Hawaii and Puerto Rico, basic wind speeds are given in a table as 105 and 145 mph (47 and 65 m/s), respectively. The map is standardized for a 50 year recurrence interval for exposure C topography (flat, open country, and grasslands with open terrain and scattered obstructions generally less than 30 ft [9 m] in height). The minimum wind speed provided in the standard is 85 mph (38 m/s). Increasing the minimum wind speed for special topographies such as mountain terrain, gorges, and ocean fronts is recommended.

The abandonment of the fastest-mile speed in favor of a 3 s gust speed first took place in the ASCE 7-1995 edition. The reasons are (1) modern weather stations no longer measure wind speeds using the fastest-mile method; (2) the 3 s gust speed is closer to the sensational wind speeds often quoted by news media; and (3) it matches closely the wind speeds experienced by small buildings and components of all buildings.

Observe that the 3 s gust speed, everywhere in the continental United States is 90 mph except the following:

- California, Oregon, and Washington—85 mph
- Hawaii—105 mph
- American Samoa—125 mph



Location	*V*, mph	(m/s)
Hawaii	105	(47)
Puerto Rico	145	(65)
Guam	170	(76)
Virgin Islands	145	(65)
American Samoa	125	(56)

Notes:

1. Values are nomial design 3 s-gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

FIGURE 5.4 Wind speed map for the United States and Alaska. Map of (a) the United States.

(*continued*)

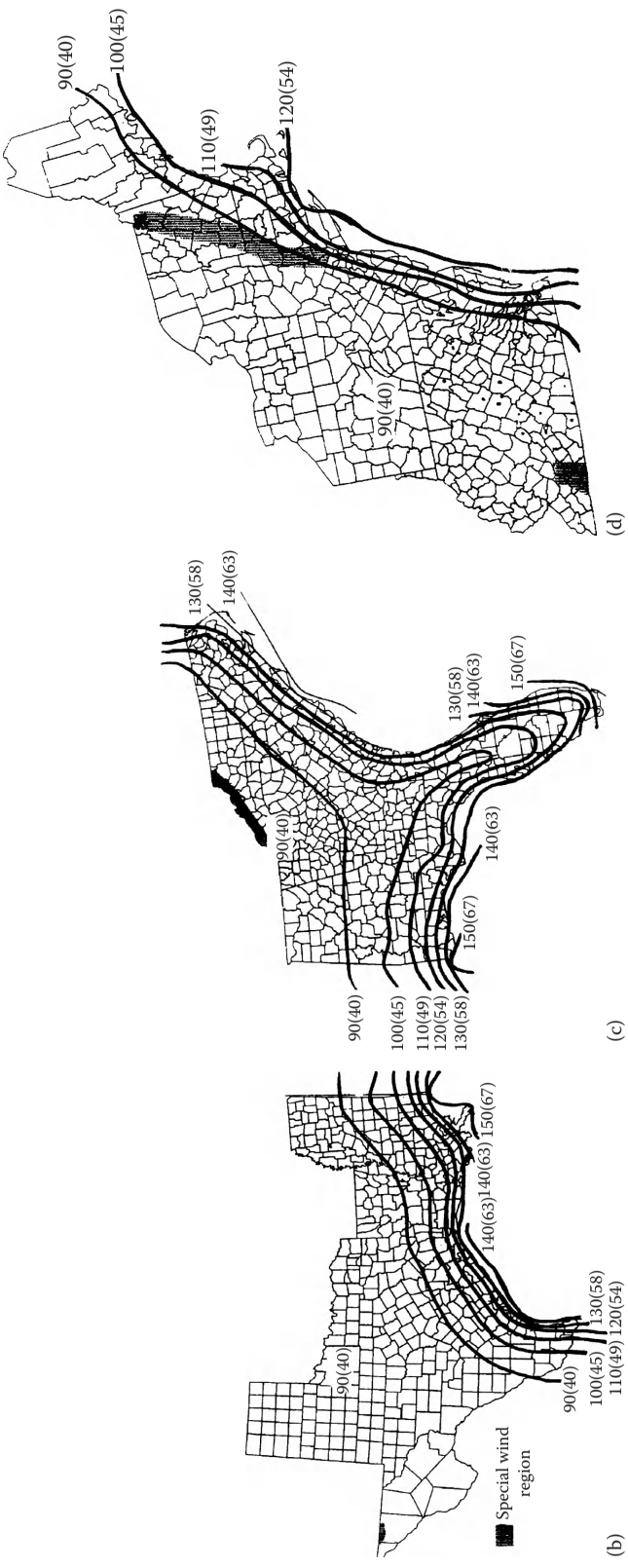


FIGURE 5.4 (continued) (b) Western Gulf of Mexico hurricane coastline (enlarged). (c) Eastern Gulf of Mexico and Southern United States hurricane coastline (enlarged). (d) Mid- and North-Atlantic hurricane coastline (enlarged). (Adapted from ASCE 7-05.)

- Puerto Rico and Virgin Islands—145 mph
- Guam—170 mph
- Hurricane coastline, special wind region—as indicated by shaded areas in ASCE 7-05 wind maps

It is of interest to know that the highest typhoon surface wind speeds ever recorded in the world are a gust wind speed of 189 mph at Muroto, Japan, and a 10 min. average wind speed of 167 mph at Lan-yu, Taiwan. The highest surface wind speed ever measured in the world is 231 mph at Mt. Washington in New Hampshire, United States, on April 12, 1934. All these record winds were caused by terrain effects of mountains.

Method 1, the simplified procedure, is not discussed here. The emphasis is on Method 2. This method, also called the analytical procedure, applies to a majority of buildings. It takes into account:

1. Basic wind speed
2. Mean recurrence interval of the wind speed considered appropriate for the design
3. Characteristics of the terrain surrounding the building
4. Height at which the wind load is being determined
5. Directional properties of the wind climate
6. Size, geometry, and aerodynamics of the building
7. Positions of the area acted on by the wind flow
8. Magnitude of the area of interest
9. Porosity of the building envelope
10. Structural properties that may make the building susceptible to dynamic effects
11. Speed-up effect of topographic features such as hills and escarpments

5.5.1 Analytical Procedure: Method 2, Overview

The more robust design procedure is the analytical method, which applies to most "rigid" structures of any height and shape. It can even design "flexible" structures with some limitations. Most practicing engineers use this method.

A significant benefit of this method is its applicability for a large variety of structures. It provides pressure coefficients for myriad shapes and heights of any type of regular-shaped buildings. In this method, internal pressure coefficients related to enclosure classification are determined and then combined algebraically with external coefficients, both of which can be positive or negative relative to the exterior and interior building surfaces. However, in determining wind forces for the design of the main wind force resisting systems (MWFRS) of enclosed buildings, we may ignore the effects of internal pressures. This is because the pressure pushes or pulls equally on all surfaces and therefore cancels out. However, roofs are directly affected by internal pressure and it must be accounted for. And all partially enclosed buildings always need to account for internal pressures on all surfaces.

The design equation for determining the wind pressure, P_z, using method 2 given in ASCE 7-05 is based the well-known equation

$$P_z = \frac{1}{2}\rho(V_{33})^2 \times k_z$$

where

P_z is the pressure induced on a particular surface at height z above mean ground level, psf

V_{33} is the basic 3 s gust speed in airport exposure (exposure category C) at reference height 33 ft (10 m) above mean ground level, mph

ρ is the mass density of air = 0.0765 lb/ft^3

k_z is the exposure coefficient to account for variation in velocity with height z above mean ground level as influenced by terrain exposure

This equation, which converts the chaotic nature of wind into a more manageable formulation, is attributed to the scientist Daniel Bernoulli (1700–1792). Since then, our understanding of the effects of wind has improved most notably due to studies conducted on scaled building models in wind tunnels. The tendency has been to introduce refinements (and attendant complexities) with an increasing degree of sophistication considered proper for codifying wind loads. The resulting relationship as defined in ASCE 7-05, is summarized as follows:

$$\begin{bmatrix}\text{Velocity}\\ \text{pressure}\end{bmatrix} = \begin{bmatrix}\text{Reference}\\ \text{velocity}\\ \text{pressure}\end{bmatrix} \times \begin{bmatrix}\text{Exposure}\\ \text{height}\\ \text{factor}\end{bmatrix} \times \begin{bmatrix}\text{Topographic}\\ \text{factor}\end{bmatrix} \times \begin{bmatrix}\text{Directionality}\\ \text{factor}\end{bmatrix} \times \begin{bmatrix}\text{Importance}\\ \text{factor}\end{bmatrix}$$

$$q_z = \tfrac{1}{2}\rho(V_{33})^2 \times k_z \times k_{zt} \times k_d \times I$$

$$[\text{wind pressure}]_{\text{windward face}} = q_z \times G_h \times (C_p - C_{pi})$$
$$[\text{wind pressure}]_{\text{non-windward face}} = q_h \times G_h \times (C_p - C_{pi})$$

The internal pressure coefficient, C_{pi}, has no effect on enclosed buildings. Therefore,

$$[\text{wind pressure}]_{\text{windward face}} = q_z \times G_h \times C_p$$
$$[\text{wind pressure}]_{\text{non-windward face}} = q_h \times G_h \times C_p$$

In ASCE 7-05, q_z is referred to as velocity pressure and is given by

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \quad (q_z \text{ in psf}, V \text{ in mph})$$

where

q_z is the velocity pressure also referred to as dynamic pressure, at height z above ground level
K_z is the velocity exposure coefficient
K_{zt} is the topographic factor
K_d is the directionality factor
V is the wind speed at an elevation 33 ft (10 m) above ground in flat open county (Exposure C), as given in Figure 5.4
I is the wind importance factor commonly denoted as I_w

The constant 0.00256 reflects the mass density of air for the standard atmosphere, that is, temperature of 59°F and sea level pressure of 29.92 in of mercury and dimensions associated with wind speed in mph. The constant is obtained as follows:

$$\text{constant} = \tfrac{1}{2}\left[\left(0.0765 \text{ lb/ft}^3\right)/\left(32.2 \text{ ft/s}^2\right)\right] \times [(\text{mi/h})(5280 \text{ ft/mi}) \times (1 \text{ h}/3600 \text{ s})]^2 = 0.00256$$

The wind directionality factor, K_d, accounts for the directionality of wind. Directionality refers to the fact that wind rarely, if ever, strikes along the most critical direction of a building. Additionally,

wind direction changes from one instant to the next. Thus, wind can be only instantaneous along the most critical direction because at the very next instant, it will not be blowing from the same direction. In the olden days (1990s), this effect was taken into account through the use of a relatively low load factor of 1.3 on the effect of wind in determining strength design load combinations. But then, ASCE 7 received comments that engineers using allowable stress design could not take advantage of the directionality effects. The decision to include $K_d = 0.85$ in the definition of the wind pressure was in direct response to these comments. To retain the same strength design values, the load factor for wind was adjusted up, from 1.3 to 1.6.

The basic wind speed V given in Figure 5.4 corresponds to a 50 year mean recurrence interval. It represents the speed from any direction at an elevation 33 ft (10 m) aboveground in flat open country (Exposure C).

The velocity pressure exposure coefficient K_z adjusts the basic wind speed V to height z and, terrain roughness (i.e., exposure category). Three exposure categories—B, C, and D—are defined. Exposure A, meant for heavily built-up city centers, was deleted in the 2002 edition of ASCE 7.

Exposure B corresponds to surface roughness typical of urban and suburban areas, while exposure C represents surface roughness in flat open country. Exposure D is representative of flat unobstructed area and water surface outside hurricane-prone regions. Exposure C, the default category applies to all cases where exposures B and D do not. Interpolation between exposure categories permitted for the first time in the ASCE 7-02 is still valid.

The wind importance factor I, commonly denoted as I_w is a factor that accounts for the degree of hazard to human life and damage to property. For category II buildings, representative of typical occupancy, $I_w = 1.0$. For category I buildings representing low hazard in the event of failure (e.g., agriculture facilities), $I_w = 0.87$ or 0.77, depending upon whether the building site is located in hurricane-prone regions. For buildings in category III posing a substantial hazard to human life in the event of failure (e.g., buildings where more than 300 people congregate in one area, and essential facilities such as fire stations), $I_w = 1.15$. For category IV buildings deemed as essential facilities, $I_w = 1.15$, the same as for category III.

The topographic factor K_{zt} is given by

$$K_{zt} = (1 + K_1K_2K_3)^2$$

It reflects the speed-up effect over hills and escarpments. The multipliers K_1, K_2, and K_3 are given in Figure 6.4 of the ASCE Standard, reproduced here as Figure 5.5 and Table 5.2.

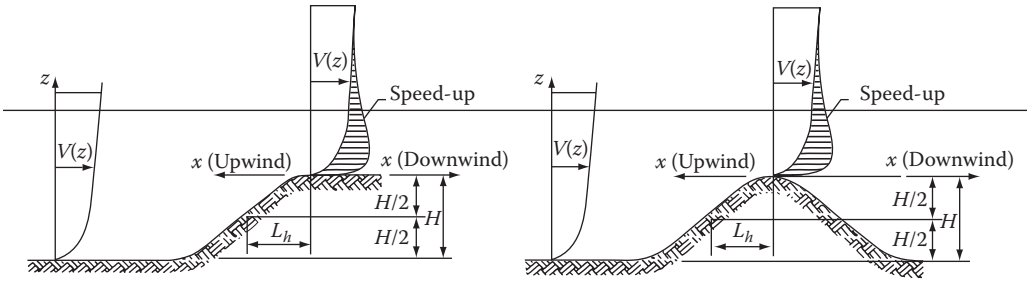
Internal pressures and suctions do not affect wind load calculations for the MWFRS. Therefore, external pressures and suctions that we use for calculating wind loads on the MWFRS may be calculated using the following simplified equations:

$$p_z = q_z G_f C_p \text{ (windward positive pressures, varies up the height)}$$

$$p_z = q_h G_f C_p \text{ (leeward negative pressures calculated at roof height, remains constant for the entire building height)}$$

The overall wind load is the summation of positive and negative pressures on the windward and leeward surfaces, respectively. Using typical value of $G_f = 0.85$, and $C_p = 0.8$ and 0.5 for the windward and leeward walls, the overall wind pressure at z for a typical squarish building is given by

$$p_z = 0.85(0.8q_z + 0.5q_h)$$



Topographic multipliers for exposure C										
H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2D Ridge	2D Escarp.	3D Axisym. hill		2D Escarp.	All other cases		2D Ridge	2D Escarp.	3D Axisym. hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.35	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h, and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_i and substitute $2H$ for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - H: Height of hill or escarpment relative to the upwind terrain, in ft (m).
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in ft (m).
   - K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3: Factor to account for reduction in speed-up with height above local terrain.
   - x: Distance (upwind or downwind) from the crest to the building site, in ft (m).
   - z: Height above local ground level, in ft (m).
   - μ: Horizontal attenuation factor.
   - γ: Height attenuation factor.

FIGURE 5.5 Topography factor k_{zt}.

5.5.2 Analytical Method: Step-by-Step Procedure

Design wind pressure or suction on a building surface is given by the equation:

$$p_z = q_z \times G_f \times C_p$$

where

p_z is the design wind pressure or suction, in psf, at height z, above ground level

q_z is the velocity pressure, in psf, determined at height z above ground for positive pressure, and at roof height h for suction

TABLE 5.2
Topography Factor K_{zt} Based on Equations

Parameters for Speed-Up over Hills and Escarpments

	$K_1/(H/L_h)$				μ	
	Exposure					
Hill Shape	B	C	D	γ	Upwind of Crest	Downwind of Crest
2D ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2D escarpments	0.75	0.85	0.95	2.5	1.5	4
3D axisym, hill	0.95	1.05	1.15	4	1.5	1.5

$K_{zt} = (1 + K_1K_2K_3)^2$

K_1 determined from table

$K_2 = \left(1 - \dfrac{|x|}{\mu L_h}\right)$

$K_3 = e^{-\gamma z/L_h}$

Source: (From Figure 6.4 in ASCE 7-05).

G_f is the gust effect factor, dimensionless, denoted as G for rigid buildings, and G_f for flexible buildings

C_p is the external pressure coefficient, which varies with building height acting as pressure (positive load) on windward face, and as suction (negative load) on non windward faces and roof

The values of C_p are shown in Figure 5.6, and along with building schematics in Figure 5.7. Effect of buildings' width to depth ratios on C_p is shown in Figure 5.8 in a graphical format.

The velocity pressure and suction q_z and q_h are given by

$$q_z = 0.00256K_zK_{zt}K_dV^2I \quad \text{(positive pressure on windward face)}$$

$$q_h = 0.00256K_hK_{zt}K_dV^2I \quad \text{(negative pressure or suction on non windward face and roof)}$$

where

K_h and K_z are the combined velocity pressure exposure coefficients (dimensionless), which take into account changes in wind speed aboveground and the nature of the terrain (exposure category B, C, or D)

K_{zt} is the topographic factor

I is the importance factor, a dimensionless parameter that accounts for the degree of hazard to human life and damage to property

V is the basic wind speed in mph that corresponds to a 3 s gust speed at 33 ft (10 m) aboveground, exposure category C, for a 50 year mean recurrence interval (see Figure 5.4)

K_d is the wind directionality factor that varies from 0.85 to 0.95 depending on the structure type

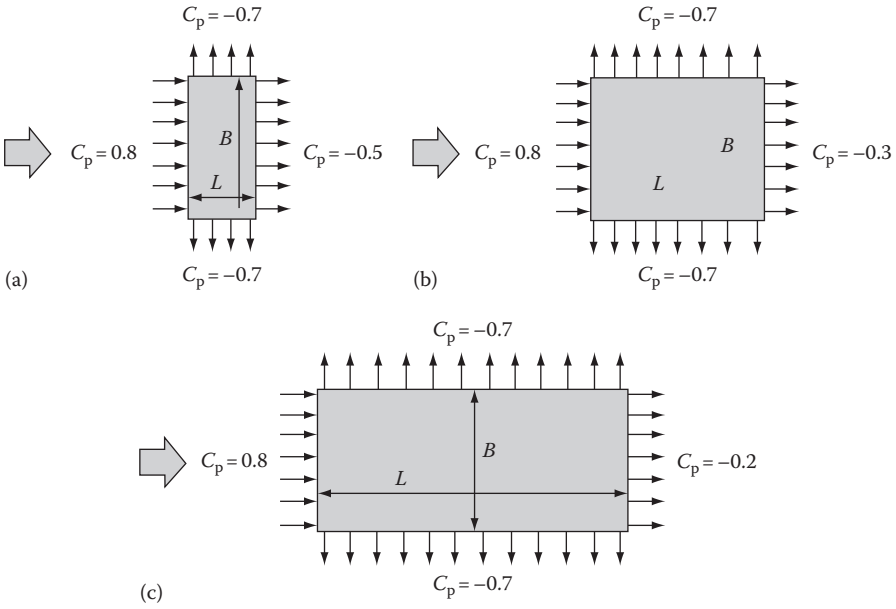


FIGURE 5.6 External pressure coefficient C_p with respect to plan aspect ratio L/B: (a) $0 \leq L/B \leq 1$; (b) $L/B = 2$; (c) $L/B > 4$. *Note*: Linear interpolation permitted.

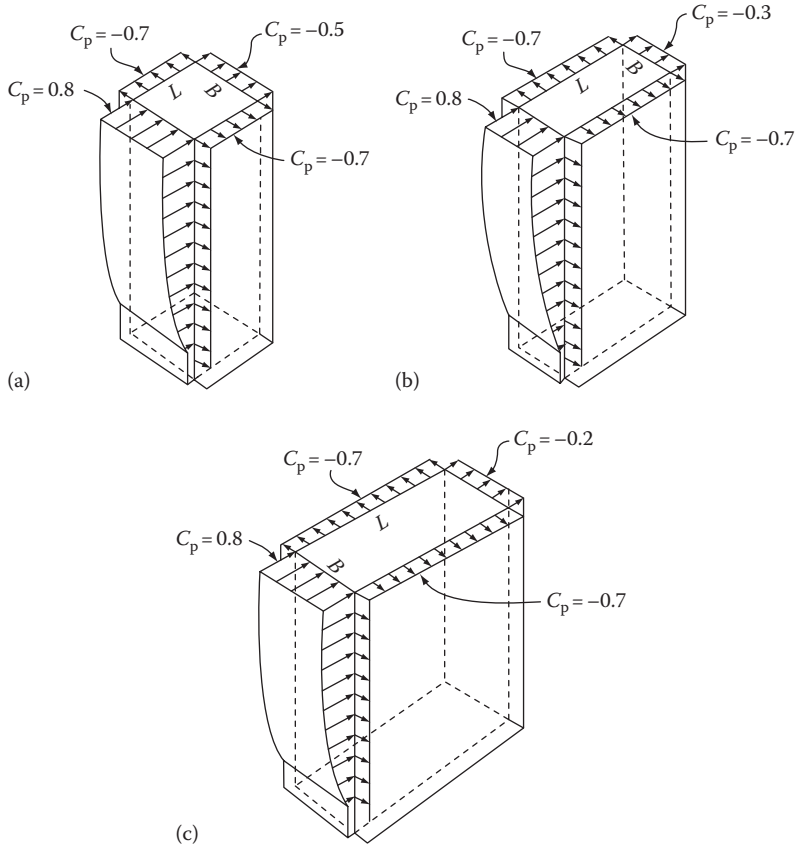


FIGURE 5.7 Schematic building elevations showing variation of C_p.

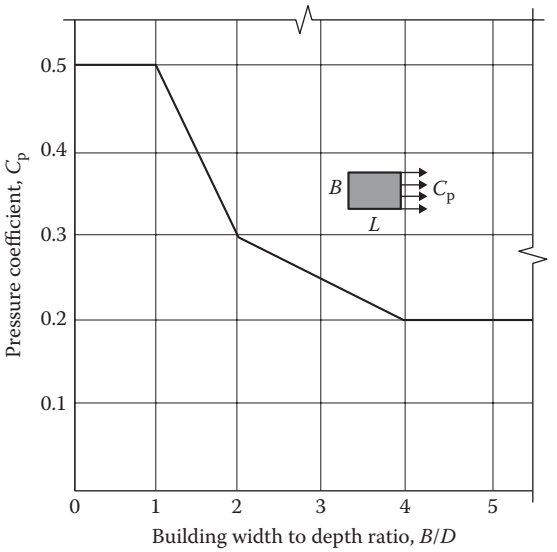


FIGURE 5.8 Leeward suction C_p versus plan aspect ratio L/B.

It should be noted that AISC 7-05 gives only a general formula for the velocity pressure q_z, at any height z, above ground. However, there are many situations in the Standard where a specific value of z is referenced. An example is height, or mean height of roof used in the calculation of suction on leeward side of buildings. Whenever the subscript h is called for, it is understood that z becomes h in the appropriate equations.

The wind directionality factor, K_d, accounts for two effects:

- The reduced probability of maximum winds blowing from any given direction
- The reduced probability of the maximum pressure coefficient occurring for any given direction

5.5.3 Wind Speed-Up over Hills and Escarpments: K_{zt} Factor

As stated previously, the topographic factor K_{zt} accounts for the effect of isolated hills or escarpments located in exposures B, C, and D. Buildings sited on upper half of isolated hill or escarpment experience significantly higher wind speeds than buildings situated at local level ground. To account for these higher wind speeds, the velocity pressure exposure coefficient is multiplied by a topographic factor K_{zt}, determined from the three multipliers K_1, K_2, and K_3 (Figure 5.5). K_1 is related to the shape of the topographic feature and the maximum speed-up with distance upwind or downward of the crest. K_2 accounts for the reduction in speed-up with distance upwind or downwind of the crest, and K_3 for the reduction in speed-up with height above the local ground surface.

The ASCE 7-05 equations used for calculating the topographic factor, K_{zt}, are approximations derived from curve fitting of these equations to the accumulated data from wind-tunnel tests. The equations and curves are complex, and the use of footnotes given in the ASCE 7-05 is somewhat confusing. Even experienced engineers often miscalculate the topographic factor. Much of the confusion in the calculation of K_{zt} is based on how the equations are manipulated when the height, H, of the topographic feature exceeds half of L_h, the half-length of the topographic feature.

5.5.4 Gust Effect Factor

This factor accounts for dynamic amplification of loading in the along-wind direction due to wind turbulence and structure interaction. It does not include allowances for across-wind loading effects,

vortex shedding, instability due to galloping or flutter, or dynamic torsional effects. Buildings susceptible to these effects should be designed using wind-tunnel results.

Three methods are permitted for calculating G. The first two are for rigid structures and the third is for flexible or dynamically sensitive structures.

5.5.4.1 Gust Effect Factor G for Rigid Structure: Simplified Method

For rigid structures (defined as those having a natural frequency of vibration greater than 1 Hz), the engineer may use a single value of $G = 0.85$, irrespective of exposure category.

5.5.4.2 Gust Effect Factor G for Rigid Structure: Improved Method

As an option to using $G = 0.85$, the engineer may calculate a more accurate value by including specific features of the wind environment at the building site and terrain exposure constants given in Table 5.3.

The gust effect factor G is given by

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right)$$

$$I_{\bar{z}} = c(33/\bar{z})^{1/6}$$

where $I_{\bar{z}}$ is the intensity of turbulence at height $\bar{z}$ and where $\bar{z}$ is the equivalent height of the structure defined as $0.6h$ but not less than $z_{\text{min.}}$ for all building heights h. $z_{\text{min.}}$ and c are listed for each exposure in Table 5.3; g_Q and g_v shall be taken as 3.4. The background response Q is given by

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

where B, h are defined in ASCE 7-05, Section 6.3; and $L_{\bar{z}}$ is the integral length scale of turbulence at the equivalent height given by

$$L_{\bar{z}} = l(\bar{z}/33)^{\bar{\varepsilon}}$$

in which l and $\bar{\varepsilon}$ are constants listed in Table 6.2 of ASCE 7-05 (Table 5.3 of this chapter).

TABLE 5.3
Terrain Exposure Constants

Exposure	α	Z_g (ft)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	l (ft)	$\bar{\varepsilon}$	$z_{\text{min.}}$ (ft)[a]
B	7.0	1200	1/7	0.84	1/4.0	0.45	0.30	320	1/3.0	30
C	9.5	900	1/9.5	1.00	1/6.5	0.65	0.20	500	1/5.0	15
D	11.5	700	1/11.5	1.07	1/9.0	0.80	0.15	650	1/8.0	7

Source: From ASCE 7-05, Table 6.2.

[a] $z_{\text{min.}}$ is the minimum height used to ensure that the equivalent height $\bar{z}$ is the greater of $0.6h$ or $z_{\text{min.}}$. For building with $h \le z_{\text{min.}}$, $\bar{z}$ shall be taken as $z_{\text{min.}}$.

Design Example: Gust Effect Factor *G*: Improved Method

Given

A 10-story steel building with the following characteristics:

Height h = 112 ft
Width perpendicular to wind, B = 90 ft
Exposure category = C
Basic wind speed V = 100 mph
Topographic factor $K_{z,t}$ = 1.0
Building depth parallel to wind, L = 95 ft
Building natural frequency n_1 = 1.1 Hz

Required

Gust effect factor G, using the improved method.

Solution

ASCE 7-05 Formulas	Commentary
$\bar{z} = 0.6 \times h = 0.6 \times 112 = 67.2\,\text{ft}$	h = building height = 112 ft, given
$I_{\bar{z}} = C\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{67.2}\right)^{1/6} = 0.1776$	c from Table 6.4
$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\varepsilon}$	l and ε from Table 6.4
$= 500\left(\frac{67.2}{33}\right)^{1/5}$	
$= 576$	
$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$	Q = background response
$= \sqrt{\frac{1}{1 + 0.63\left(\frac{90+112}{576}\right)^{0.63}}}$	
$= 0.87$	
	B = building width perpendicular to wind = 112 ft, given
	G = gust effect factor
$G = 0.925\left(\frac{(1 + 1.7 g_Q I_{\bar{z}} Q)}{1 + 1.7 g_v I_{\bar{z}}}\right)$	
$= 0.925\left(\frac{1 + 1.7 \times 3.4 \times 0.1776 \times 0.86}{1 + 1.7 \times 3.4 \times 0.1776}\right)$	
$= 0.86$	

Observe that the calculated value of G = 0.86 is not much different from the default value of G = 0.85 permitted for rigid structures. This is because the example building's natural frequency of 1.1 Hz, is very nearly equal to 1.0 Hz, the cut-of frequency between rigid and flexible structures.

5.5.4.3 Gust Effect Factor G_f for Flexible or Dynamically Sensitive Buildings

Gust effect factor G_f depends on

- Basic wind speed V
- Exposure category B, C or D
- Building natural frequency
- Building damping

ASCE 7 defines a rigid building as "A building or other structure whose fundamental frequency is greater than or equal to 1 Hz." The commentary of ASCE 7 goes on to state, "When building or other structures have a height exceeding four times the least horizontal dimension or when there is a reason to believe that the natural frequency is less than 1 Hz (natural period greater than 1s), the natural frequency calculated by approximate methods for seismic design and appropriate estimates of natural frequency for it should be investigated." The ASCE 7 commentary explains the difference between the natural frequency calculated by approximate methods for seismic design and appropriate estimates of natural frequency for wind design. Approximate equations of natural frequency developed for seismic design tend to give higher estimates of the natural frequency (lower estimates of the structure's period), as this gives conservative approximation of the seismic base shear. For wind design, the opposite case exists. That is, these higher estimates of the structure's natural frequency can incorrectly categorize very slender buildings as rigid, when they are in fact, flexible. Alternate equations for natural frequency of various building types and comparison of results of these equations to values used in other countries, are given in the ASCE 7-05 commentary. When using ASCE 7, an engineer needs to determine whether the structure can be categorized as rigid.

A building is considered flexible if it contains a significant dynamic response. Resonant response depends on the gust structure contained in the approaching wind, on wind loading pressures generated by the wind flow about the building, and on dynamic properties of the building. Gust energy in wind is smaller at frequencies about 1 Hz; therefore, resonant response of most buildings and structures with lowest natural frequency above 1 Hz will be sufficiently small that resonant response can often be ignored. When buildings have a height exceeding four times the least horizontal dimension or when there is reason to believe that the natural frequency is less than 1 Hz (natural period greater than 1 s), the natural frequency should be investigated.

The formula for calculating G_f is as follows:

$$G_f = 0.925\left(\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_vI_{\bar{z}}}\right)$$

where g_Q and g_v shall be taken as 3.4 and g_R is given by

$$g_R = \sqrt{2In(3600n_1)}+\frac{0.577}{\sqrt{2In(360n_1)}}$$

and where R, the resonant response factor, is given by

$$R = \sqrt{\frac{1}{\beta}R_nR_hR_B(0.53+0.47R_L)}$$

$$R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) \quad \text{for } \eta > 0$$

$$R_\ell = 1 \quad \text{for } \eta = 0$$

where the subscript ℓ in Equation 4.19 shall be taken as h, B, and L, respectively, and where

n_1 is the building natural frequency
$R_1 = R_h$ setting $\eta = 4.6 n_1 h/\bar{V}_{\bar{z}}$
$R_\ell = R_B$ setting $\eta = 6.4 n_1 B/\bar{V}_{\bar{z}}$
$R_\ell = R_L$ setting $\eta = 15.4 n_1 L/\bar{V}_{\bar{z}}$
β is the damping ratio, percent of critical h, B, L are defined in Section 6.3
$\bar{V}_{\bar{z}}$ is the mean hourly wind speed (ft/s) at height $\bar{z}$ determined from equation:

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$$

where

$\bar{b}$ and $\bar{\alpha}$ are constants listed in Table 5.3
V is the basic wind speed in mph

Design Example: Gust Effect Factor G_f: Flexible Structure

Given

Building height h = 600 ft
Building width perpendicular to wind, B = 100 ft
Building depth parallel to wind, L = 100 ft
Building natural frequency n_i = 0.2 Hz
Damping ratio—0.015
Exposure category = C
Basic wind speed V = 140 mph

Required

Gust effect factor G_f

ASCE 7.05 Formulas	Commentary
V = wind speed in ft/s $= V_{mph} \times 1.467$ $= 140 \times 1.467 = 205$ ft/s	V = 140 mph, given
$\bar{z} = 0.6h$, but not less than $z_{min.}$ $= 0.6 \times 600 = 360$ ft $> \bar{z}_{min.} = 15$ ft OK	h = building height = 600 ft, given $\bar{z}_{min.}$ = 15 ft, from Table 6.4 c = 0.20, from Table 6.4
$I_{\bar{z}} = C\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.20\left(\frac{33}{360}\right)^{1/6} = 0.134$	l = 500 ft, $\varepsilon = 1/5$, from Table 6.4

(*continued*)

(continued)

ASCE 7.05 Formulas	Commentary
$L_{\bar{z}} = I\left(\frac{\bar{z}}{33}\right)^{\varepsilon} = 500\left(\frac{360}{33}\right)^{1/5} = 806 \text{ ft}$	
$Q = \sqrt{\frac{1}{1+0.63\left(\frac{g+h}{L_{\bar{z}}}\right)^{0.63}}}$	Q = background response B = building with perpendicular to wind = 100 ft, given
$= \sqrt{\frac{1}{1+0.63\left(\frac{100+600}{761}\right)^{0.63}}}$	
$= 0.796, \quad Q^2 = 0.634$	
$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V = 0.65\left(\frac{360}{33}\right)^{1/6.5} \times 205 = 192 \text{ ft/s}$	$\bar{b} = 0.65, \bar{\alpha} = 1/6.5$, from Table 6.4
$\hat{V}_z = \hat{b}\left(\frac{\bar{z}}{33}\right)^{\hat{\alpha}} V = 1.0 \times \left(\frac{360}{33}\right)^{1/9.5} \times 205 = 264 \text{ ft/s}$	$\hat{b} = 1.0, \hat{\alpha} = 1.0$, from Table 6.4
$N_1 = \frac{n_1 L_z}{\bar{V}_{\bar{z}}} = \frac{0.2 \times 806}{192} = 0.84$	n_1 = natural frequency = 0.2 Hz, given
$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}}$	
$= \frac{7.47 \times 0.84}{(1+10.3 \times 0.84)^{5/3}} = 0.143$	
$\eta_b = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6 \times 0.2 \times 100}{192} = 0.479$	$e = 2.71$
$R_B = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2}(1 - e^{-2\eta_b})$	
$= \frac{1}{0.479} - \frac{1}{2 \times 0.479^2}(1 - 2.71^{-2 \times 0.479})$	
$= 0.739$	
$\eta_h = \frac{4.6 n_1 h}{\bar{v}_{\bar{z}}} = \frac{4.6 \times 0.2 \times 600}{192} = 2.875$	
$R_h = \frac{1}{\eta_k^{\cdot}} - \frac{1}{2\eta_k^2}(1 - e^{-2\eta_k})$	
$= \frac{1}{2.875} - \frac{1}{2 \times 2.875^2}(1 - 2.71^{-2 \times 2.875})$	
$= 0.288$	
$\eta_L = \frac{15.4 n_1 L}{\bar{v}_{\bar{z}}} = \frac{15.4 \times 0.2 \times 100}{192} = 1.604$	L = building breadth parallel to wind = 100 ft, given

(continued)

ASCE 7.05 Formulas	Commentary
$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}(1 - 2.71^{-2\eta_L})$	
$= \frac{1}{1.604} - \frac{1}{2 \times 1.604^2}(1 - 2.71^{-2\times 1.604})$	
$= 0.618$	
$R^2 = \frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)$	β = damping ratio = 0.015, given
$= \frac{1}{0.015} \times 0.143 \times 0.288 \times 0.739(0.53 + 0.47 \times 0.618)$	
$= 1.655$	
$R = \sqrt{1.655} = 1.29$	
$g_R = \sqrt{2\ln(3600 \times n_1)} + \frac{0.577}{\sqrt{2\ln(3600 \times n_1)}}$	In means logarithm to base $e = 2.71$
$= \sqrt{2\ln(3600 \times 0.2)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.2)}}$	
$= 3.789$	
$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R R^2}}{1 + 1.7 g_V I_{\bar{z}}}\right)$	G_f is the gust factor $g_Q = g_V = 3.4$ (defined in the equation for G)
$= 0.925\left(\frac{1 + 1.7 \times 0.134\sqrt{3.4^2 \times 0.634 + 3.789^{-2} \times 1.665}}{1 + 1.7 \times 3.4 \times 0.134}\right)$	
$= 1.185$	

5.5.5 Along-Wind Displacement and Acceleration

Typical modern high-rise buildings are more than likely to be light weight and flexible and, therefore, more prone to wind dynamic motion problems than their earlier counterparts which had heavy cladding and masonry partitions. It is of interest to note that buildings dynamic excitations during earthquakes, insofar as perception of motion by the occupants is concerned, are irrelevant because occupants are thankful to have survived the trauma and are less prone to complain about motion perception. On the other hand, the sentiment when experiencing peak dynamic response of buildings to fluctuating wind forces is quite different. This may be because windstorms occur more frequently and are not as traumatic as earthquakes. Consequently, it has become necessary to evaluate the buildings dynamic behavior related to wind-induced accelerations particularly at top floors to assess occupants comfort.

When considering the response of a tall building to wind gusts, both along-wind and across-wind responses must be considered. These arise from different effects of wind, the former due to buffeting effects caused by turbulence; the latter due to vortex shedding on opposite sides of building. The cross-wind acceleration may be of particular importance with regard to the comfort of occupants because it is likely to exceed along-wind acceleration. This would occur if the building is slender about both axes with a geometric ratio $\sqrt{WD/H}$ less than one-third, where W and D are the across- and along-wind plan dimensions, and H is the building height.

The most important criterion for verifying comfort of building's occupants is the peak acceleration they are likely to experience. It is thus important to estimate the probable maximum accelerations in both the along-wind and across-wind directions. ASCE 7-05 gives a method for predicting along-wind responses, including peak acceleration, but does not provide a procedure for estimating across-wind response. It is of interest to note, however, that the National Building Code of Canada (NBCC) gives such a procedure. Also given in the same reference are methods for calculating along-wind displacements and accelerations.

5.5.6 Summary of ASCE 7-05 Wind Provisions

Although ASCE 7-05 outlines several methods for calculating wind loads, these may be grouped into two broad categories. The first is the *static* approach, which approximates the wind force on a structure by an equivalent static load. The kinetic energy of air in motion is converted into the potential energy of pressure there by creating wind load. The design pressure is calculated as an equivalent static load by considering the effects of velocity of wind, shape of the structure, its exposure, importance factor given as a function of the category, gust effects, and other related factors. For flexible buildings, the gust factor is determined by an analysis that incorporates the dynamic properties of the building. The static method takes into consideration the load magnification effects caused by wind gusts in resonance with along-wind vibrations of the structure but does not include allowances for across-wind, torsional loading, or vortex shedding. However, shielding or the effect of adjacent structures is considered in the exposure category.

The second approach to wind loading is a *dynamic* analysis which involves a determination of the wind loading through wind-tunnel tests and a dynamic analysis of the structure. Whereas the equivalent velocity pressure used in the static approach takes into account the dynamic response of non-slender buildings to gusts in the direction parallel to the wind, it does not provide for the effects of vortex shedding. For buildings whose dynamic properties make them wind-sensitive, a detailed dynamic analysis should be preformed. The dynamic analysis will include wind-tunnel tests which will simulate, to scale, the structure, and the wind velocities.

Extreme winds, such as *thunderstorms*, *hurricanes*, and *tornadoes* are not directly considered in ASCE 7-05 wind provisions.

5.6 WIND-TUNNEL TESTS

Wind tunnels such as those shown in Figure 5.9 are used, among other things, to study the aeroelastic behavior and wind pressures and suctions on buildings.

Tests conducted in wind tunnels typically offer the following benefits.

- Provides predictions of wind-induced building motions (accelerations and torsional velocities) likely to be experienced by occupants of the top occupied floors, and compares the test results to available serviceability criteria. Information provided by tests enables the design team (structural engineers, architect, and developer) to make timely and appropriate modifications, if required, to architectural and structural design.
- Pre-test estimate of cladding pressures and overall loads, based on a review of similar buildings, with appropriate consideration of the local meteorological data. The information can help the engineer, architect, and façade engineer to develop a preliminary designs and initial cost estimates.
- Provides an assessment of expected pedestrian wind comfort along with any conceptual recommendations for improvement to key pedestrian areas (e.g., main entrances, pool decks).
- Because wind-tunnel studies consider the effect of nearby buildings and directional variations in the local wind climate, the overall design wind loads are generally (but not always) lower than code wind loads resulting in lower cost.

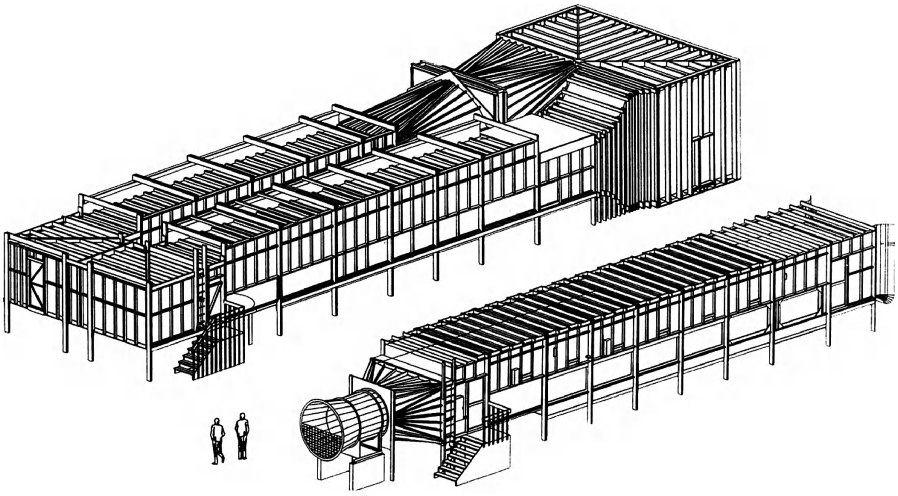


FIGURE 5.9 Schematics of wind tunnels.

Wind-tunnel tests are considered to be properly conducted only if the following conditions are satisfied:

1. The natural atmospheric boundary layer has been modeled to account for the variation of wind speed with height.
2. The length scale of the longitudinal component of atmospheric turbulence is modeled to approximately the same scale as that used to model the building.
3. The modeled building and surrounding structures and topography are geometrically similar to their full-scale counterparts.
4. The projected area of the modeled building and surroundings is less than 8% of the test section cross-sectional area unless correction is made for blockage.
5. The longitudinal pressure gradient in the wind-tunnel test section is accounted for.
6. Reynolds number effects on pressures and forces are minimized.
7. Response characteristics of the wind-tunnel instrumentation are consistent with the required measurements.

5.6.1 Types of Wind-Tunnel Tests

Three basic types of wind-tunnel tests are commonly used:

1. Rigid pressure model (PM)
2. Rigid high-frequency base balance model (HFBB/HFFB)
3. Aeroelastic model (AM)

The rigid PM measures local peak pressures principally for the design of cladding elements. However, the data can be extrapolated to provide pressures for the determination of overall mean loads. The high-frequency model measures overall fluctuating loads for the determination of buildings dynamic response. The AM is used for direct measurement of overall loads, deflections, and accelerations and deemed necessary, when the lateral motions of a building are considered to have a large influence on wind loading.

Shown in Figures 5.10 through 5.13 are representative photographs of rigid PM in wind tunnel, high-frequency force balance model, and AM. The reader is referred to the text "Reinforced Concrete Design of Tall Buildings" by the author, for a complete description of different types of wind-tunnel models and testing techniques.

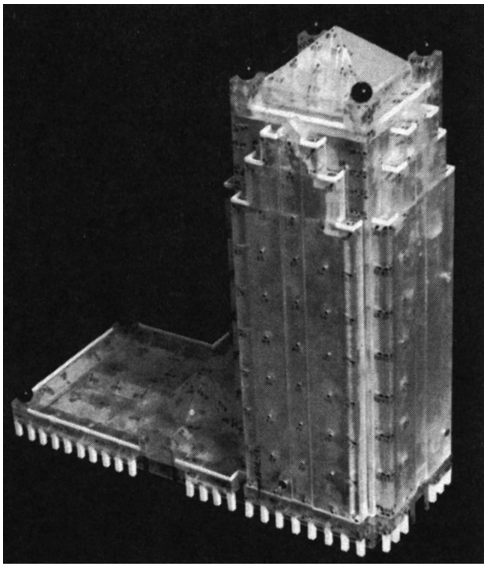


FIGURE 5.10 Rigid pressure model.



FIGURE 5.11 Model in wind tunnel.

It is often difficult to determine quantitatively when aeroelastic study is required. The following factors may be used as a guide in making a decision:

1. The building height-to-width ratio is greater than about 5; that is, the building is relatively slender.
2. Approximate calculations show that there is a likelihood of vortex shedding phenomenon.
3. The structure is light in density on the order of 8–10 lb/ft^3 (1.25 to 1.57 kN/m^3).
4. The structure has very little inherent damping, such as a building with welded steel construction.
5. The structural stiffness is concentrated in the interior of the building, making it torsionally flexible. A building with a central concrete core is one example.
6. The calculated period of oscillation of the building is long, in excess of 4 or 5 s.
7. Existence of nearby buildings that could create unusual channeling of wind, resulting in torsional loads and strong buffeting action.

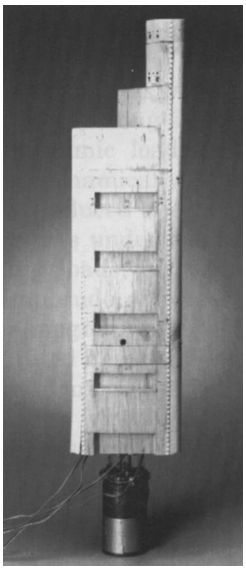


FIGURE 5.12 High-frequency force balance model.

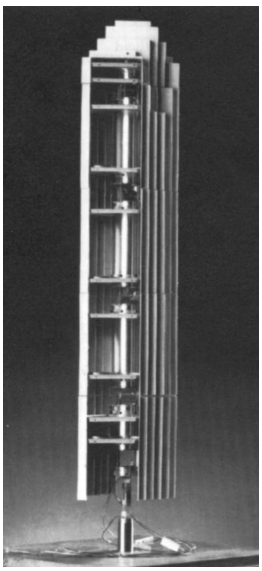


FIGURE 5.13 Aeroelastic model.

8. The building is sited such that predominant winds occur from a direction most sensitive to the building oscillations.
9. The building is a high-rise apartment, condominium, or hotel. Occupants in these buildings are more likely to experience discomfort from building oscillations. This is because residents in these buildings are likely to remain longer in a given location than they would in a typical office setting.

The choice of the AM approach for a tall building depends largely on the complexity of the structural system and the exterior geometry. For many tall buildings, the structural system and geometry are such that the response is primarily in the fundamental sway modes of vibration. However, for

some buildings the contribution of the fundamental torsional mode becomes significant. As well, for some structural systems or geometries there is strong coupling between directions within modes such that the modes are three-dimensional (3D) in character. This latter detail in particular makes it necessary to design an AM that simulates these complicated modes while maintaining consistent scaling parameters in both sway and torsion directions. Such a model is known as a multi-degree-of-freedom AM. The scaling requirements for AM are based on established similarity theory.

5.6.2 Option for Wind-Tunnel Testing

As an option to wind-tunnel procedure, ASCE 7-05 in Section 6.5.2, permits analytical procedure using recognized literature documenting such wind load effects. These effects principally consist of load magnification caused by gusts in resonance with along-wind vibrations of flexible buildings.

5.6.3 Lower Limits on Wind-Tunnel Test Results

Wind-tunnel test frequently measure wind loads that are significantly lower than required by ASCE 7-05. This feature is principally due to the shape of the building, shielding in excess of that implied by exposure categories, and necessary conservatism in enveloping load coefficients. In some cases, adjacent structures may shield the structure sufficiently that removal of one or two structures could significantly increase wind loads. Additional wind-tunnel testing without specific nearly buildings (or with additional buildings if they might cause increased loads through channeling or buffeting) is an effective method for determining the influence of adjacent buildings. It would be prudent to test any known conditions that change the test results and to discuss the results among the owner, architects, and wind consultants. However, it is impossible to anticipate all possible changes to the surrounding environment that could significantly impact pressure for the MWFRS and for cladding pressures. Also, additional testing may not be cost-effective. Suggestions provided in the ASCE commentary for placing lower limit on wind-tunnel results are given in the following sections.

5.6.3.1 Lower Limit on Pressures for Main Wind-Force Resisting System

Forces and pressure determined by wind-tunnel testing shall be limited to not less than 80% of the design forces and pressures that would be obtained in Section 6.5 of ASCE 7-05 for the structures unless specific testing is performed to show that it is the aerodynamic coefficient of the building, rather than shielding from nearby structures, that is responsible for the lower values. The 80% limit may be adjusted by the ratio of the frame load at critical wind directions as determined from wind-tunnel testing without specific adjacent buildings, but including appropriate upwind roughness, to that determined by Section 6.5.

5.6.3.2 Lower Limit on Pressures for Components and Cladding

The design pressures for components and cladding on walls or roofs shall be selected as the grater of the wind-tunnel test results or 80% of the pressure obtained for Zone 4 for walls and Zone 1 for roofs as determine in Section 6.5 of ASCE 7-05, unless specific testing is performed to show that it is the aerodynamic coefficient of the building, rather than shielding from nearby structures, that is responsible for the lower values. Alternatively, limited tests at a few wind directions without specific adjacent buildings, but in the presence of an appropriate upwind roughness, may be used to demonstrate that the lower pressures are due to the shape of the building and not due to shielding.

5.7 BUILDING DRIFT

Drift (lateral deflection) in a building is a serviceability issue primarily from the effects of wind and seismic. Drift limits are imposed on buildings to minimize damage to cladding and to nonstructural walls and partitions. Lateral frame deflection is evaluated for the building as a whole, where the

applicable parameter is the total building drift (defined as the lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ/H). For each floor, the applicable parameter is *interstory drift* [defined as the lateral deflection of a floor relative to that of the floor immediately below, divided by the distance between floors, $(\delta - \delta_{n-1})/h$].

Typical drift limits most widely used in wind design are H (or h)/400 to H (or h)/500. An absolute limit on interstory drift is sometimes imposed in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the interstory drift exceeds about 3/8 in. (10 mm), unless special details are provided to accommodate larger movements. It should be noted that many cladding components can accept deformations that are significantly larger.

It is important to recognize that frame racking or shear distortion is the real cause of damage to building elements such as cladding and partitions, and that drift control limits by themselves in wind-sensitive buildings do not provide comfort of the occupants under wind load.

5.8 HUMAN RESPONSE TO WIND-INDUCED BUILDING MOTIONS

Designers of wind-sensitive buildings have long recognized that perception of building motion under the action of wind may be described by various physical quantities including maximum displacement, velocity, acceleration, and rate of change of acceleration (sometimes called "jerk"). Acceleration has, however, become the standard for evaluation because it is readily measured in the field and can be calculated analytically. Human response to building motion is a complex phenomenon involving many psychological and physiological factors. Perception and tolerance thresholds of acceleration as a measure of building motion are known to depend on factors such as frequency of the building, occupant gender, age, body posture (sitting, standing, or reclining), body orientation, expectation of motion, body movement, visual cues, acoustic clues, and the type of motion (translational or torsional). Different thresholds and tolerance levels exist for different people and responses can be very subjective. It is known that some people will become accustomed to building motion and tolerate higher levels than others. Limited research exists on this subject but certain standards have been applied for design as discussed below.

Acceleration in wind-sensitive buildings maybe expressed as either root mean square or peak acceleration. Both measures are used in practice and there is no clear agreement as to which is the more appropriate measure of motion perception. Target peak accelerations of 20 to 25 milli-g (0.020 to 0.025 times the acceleration of gravity) for commercial buildings (occupied mostly during daylight hours) and 15 to 20 milli-g for residential buildings (occupied mostly during the entire day) under a 10 year mean recurrence interval wind storm have been successfully used in practice for many tall building designs. The target is generally more strict for residential buildings because of their continuous occupancy, the belief that people are less sensitive and more tolerant at work than at home, the fact that there is more turnover in commercial buildings, and that commercial buildings are more easily evacuated for peak wind events.

It is important to recognize that perception to building motion is strongly influenced by building mass and available damping as well as stiffness. For this reason, building drift limits by themselves should not be used as the sole measure of controlling building motion. Damping levels for use in building motion under wind events are generally taken as approximately 1% of critical damping for steel buildings, 1.5% for composite buildings, and 2% for reinforced concrete buildings.

Every building must satisfy a strength criterion typically requiring each member be sized to carry its design load without buckling, yielding, or fracture. It should also satisfy the intended function (serviceability) without excessive deflection and vibration. While strength requirements are traditionally specified, serviceability limit states are generally not included in building codes. The reasons for not codifying the serviceability requirements are several. Failure to meet serviceability limits is generally not a catastrophe, is a matter of judgment that entails the perceptions and expectations of the user or owner, and because the benefits themselves are often subjective and difficult to quantify. However, the fact that serviceability limits are not codified should not diminish their importance.

A building that is designed for code loads may nonetheless be too flexible for its occupants and may also cause portions of building cladding to fall, potentially injuring pedestrians below.

5.9 STRUCTURAL PROPERTIES REQUIRED FOR WIND-TUNNEL DATA ANALYSIS

For a rigorous interpretation of wind-tunnel test results, certain dynamic properties of a structure are required. These are furnished by the structural engineer and consist of

1. Natural frequencies of the first six modes of vibration
2. Mode shapes for the first six modes of vibration
3. Mass distribution, mass moments of inertia, and centroid location for each floor
4. Damping ratio
5. Miscellaneous information such as origin and orientation of the global coordinate system, floor heights, reference elevation for "base" overturning moments

5.9.1 Natural Frequencies

The natural frequencies (the reciprocal of periods) are the fundamental result of a dynamic analysis of the structure. Usually, this is performed by a 3D computer program, which has the capability of performing "eigenvalue" analysis. Generally, only the first three modes are used as these will correspond to the fundamental modes in each of the sway (x, y) and torsional (z) directions. It should be noted that if the structure or mass distribution is unsymmetrical, then at least two of these components will be coupled together in some modes. Normally, the higher modes (four through six) are required only to insure that all of the fundamental directions have been included.

5.9.2 Mode Shapes

Each mode of vibration is described by both a natural frequency and a shape. The mode shapes consist of tabulated values of the x, y, and z deformations of each degree-of-freedom in the structure. For wind-tunnel purposes, the floor diaphragm is typically considered rigid, and a single set of x, y, z deformations is established for each floor. It should be noted that today's buildings of structural complexity commonly feature 3D mode shapes coupling the three-degrees-of-freedom, two translational and one torsional, at each floor.

Mode shapes have no units. They are of intermediate magnitude and can be scaled to any desired size. However, it should be remembered that when multi-dimensional mode shapes include both translational and rotational (twist) components, the same sealing factor must be applied to all components.

Another aspect of mode shapes concerns the reference system used in conveying the mode shapes. Most commercial programs specify the components with respect to the center of mass of each floor. If the shape consists of coupled twist and displacement, then the displacement magnitude is dependent on the location of the reference origin. If the centers of mass do not align on a straight vertical axis—as in setbacks or shear wall drop-offs—then the displacements will contain offsets or "kinks." It is essential, therefore, that the wind engineer and the structural engineer sign off on the reference system used in generating the modal data.

5.9.3 Mass Distribution

The mass and moment of inertia are required at each floor which typically includes the structure's dead weight. Typically the moment of inertia is taken about a vertical axis through the centroid (center of mass) of each floor.

All of the mass in the structure should be included since it will affect the natural frequency, which in turn, will influence the loads determined from the wind-tunnel tests. As a crude rule of thumb, an $x\%$ change in the natural frequency may cause the loads to change by $0.5x–2x\%$.

The mass distribution is needed for two reasons. First, the mass and mode shape are used together with the natural frequency, to determine the generalized stiffness of each mode. The stiffness is combined with the generalized load (measured on the wind-tunnel model) to determine the fluctuating displacement response in each mode. This is needed to evaluate the acceleration likely to be experienced at the top floors of the building.

Second, the static-equivalent loads from the wind-tunnel analysis consist of mean, background, and resonant contributions. The resonant contributions (which in many cases are the single largest contributor) are applied to the structure as concentrated forces at each concentrated mass, and are in proportion to the mass (and also to the modal displacement of that mass). Thus, the accuracy of the wind-tunnel load distribution (i.e., the floor-by-floor forces) is dependent on the relative mass throughout the structure.

5.9.4 Damping Ratio

This parameter is perhaps the most difficult one to asses since there is currently no known method to compute it. The best we can do is to make assumptions based on limited field data. Customary practice in many parts of the world is to use a value of 0.01 for a steel frame structure, and 0.02 for a concrete structure for the prediction of 50–100 year loads. For special structures, for example, a mixed or composite frame, or those with extreme aspect ratios, other values may be appropriate. It should be noted that wind design is based on different principles from earthquake design, for which very high values of damping, usually 0.05, are considered. This value comes from those schooled in seismic design, based on ultimate conditions that do not apply to wind design. Therefore, wind engineers strongly encourage structural engineers to consider a lower value. As stated above, a service-level damping ratio of 0.01 or 0.02 is still the most used to determine the service load effects. An extreme damping level (say, 0.03) in combination with wind speeds that have recurrence interval of approximately 500–1000 years is also used. For the prediction of low return period accelerations such as a 10 year return period, it may be appropriate to use a lower damping value.

5.9.5 Miscellaneous Information

- *Global coordinate system*. The engineer typically creates and uses this system when inputting nodal point locations in the horizontal (usually x, y) plane. Computer output will report the center of masses using this same system. A supplemental sketch is required to define both the origin and coordinate directions relative to the structure.
- *Floor heights*. Provide a tabulation of the relative floor heights and some reference to an absolute height (datum).
- *Reference "base" elevation*. In addition to concentrated floor forces and torques to be applied at each floor level, a summary of information will be provided by the wind engineer as "base" overturning moments and torques. These moments are typically evaluated at "ground" level or "foundation" level (top of footing or pile caps, etc.) but they can be reported by the wind engineer at any elevation desired by the structural engineer.

5.10 PERIOD DETERMINATION FOR WIND DESIGN

Many simple formulas have been proposed over the years to estimate a tall building's fundamental periods, for preliminary design purposes, until a rigorous calculation can be made. Sometimes this rough approximation is the only calculation of fundamental period that can be made when the

wind-tunnel engineer is asked to perform a "desktop" prediction of the eventual loads. The formula most widely recognized today is

$$T = \frac{H\,(\text{ft})}{150}$$

which is apparently in reasonable agreement with many field measurements. However, this value is not in good agreement with the generally accepted analyses. The calculated periods are twice as much as the computer eigenvalue prediction.

It is not known if this observed discrepancy is due to

1. Errors (or other in-applicability) of the field measurements
2. The need for a formula thought to be conservative for seismic application but in fact unconservative for wind purposes
3. Computer modeling inaccuracies and oversimplified modeling assumptions

Wind-tunnel engineers are typically hesitant to "outguess" the design engineer or substitute their own estimate of the structure's period. They are most likely to produce loads consistent with the modal properties provided the engineer. So, this is an issue worthy of further research. Until then, it is appropriate for discussion between the wind-tunnel engineer and design engineer.

Another consideration that goes hand in hand with the determination of building periods is the value of damping for the structure. Damping for buildings is any effect that reduces its amplitude of vibrations. It results from many conditions ranging from the presence of interior partition walls, to concrete cracking, to deliberately engineered damping devices. While for seismic design, 5% of critical damping is typically assumed for systems without engineered damping devices, the corresponding values for wind design are much lower as buildings subject to wind loads generally respond within the elastic range as opposed to inelastic range for seismic loading.

The ASCE 7-05 Commentary suggests a damping value of 1% for steel buildings and 2% for concrete buildings. These wind damping values are typically associated with determining wind loads for serviceability check. Without recommending specific values, the commentary implicitly suggests that higher values may be appropriate for checking the survivability states.

Given this scenario, what values are design engineers supposed to use for ultimate level wind loads? Several resources are available that provide values of damping for service and ultimate loads. The values vary greatly depending upon which reference cited in the commentary, is used. Depending upon the type of lateral force resisting system, the damping value may vary from a low of 0.5% to a high of 16% or more.

The level of damping has only a minor effect on the overall base shear for wind design for a large majority of low- and mid-rise buildings. Nevertheless, where serviceability criteria govern, such as accelerations for tall buildings, a more in-depth study of damping criteria is typically warranted.

The computation of the fundamental building period is essential for calculating lateral load effects due to both seismic and wind forces. Code prescribed empirical equations for calculating approximate building periods are now provided for both seismic and wind design. The following paragraphs explain the background and assumptions behind these equations.

While the use of the fundamental building period for seismic design calculations is well established, the parameters used for wind design have not been as clear. For wind design, the building period is only relevant for those buildings designated as "flexible" (having a fundamental building period exceeding one second). When a building is designated as flexible, the natural frequency (inverse of the building's fundamental period) is introduced into the gust-effect factor, G_f.

Prior to ASCE 7-05, designers typically used either the approximate equations within the seismic section or the values provided by a computer eigenvalue analysis. The first can actually be unconservative because the approximate seismic equations are intentionally skewed toward shorter

building periods. Thus, for wind design, where longer periods equate to higher base shears, their use can provide potentially unconservative results. Also, the results of an eigenvalue analysis can yield building periods much longer than those observed in actual tests, thus providing potentially overly conservative results.

We now have a new formula in the ASCE 7-05 commentary that is based solely on the building height. This equation, $T = H$ (ft)/150, according to some wind engineers, is quite unconservative by as much as a factor of 2 and, therefore, is not appropriate for wind studies.

5.11 ASCE 7-10 WIND LOAD PROVISIONS

The following significant changes have been incorporated in ASCE 7-10:

1. Complete reorganization of wind provisions
2. New wind speed maps
3. New wind-borne debris region
4. Reintroduction of Exposure D for water surfaces in hurricane prone regions
5. New simplified procedure for buildings less than 160 ft in height

ASCE 7-10 has new Chapters (Chapters 26 through 31) as opposed to a single chapter, Chapter 6 in ASCE 7-05. A flow chart is included at the end of each chapter to clarify the applicability of wind provisions. The titles of the new chapters are as follows:

Chapter 26: General Requirements
Chapter 27: Main Wind Force Resisting System, MWFRS, Directional Procedure
Chapter 28: MWFRS Envelope Procedure
Chapter 29: MWFRS Other Structures and Appurtenances
Chapter 30: Components and Cladding
Chapter 31: Wind-Tunnel Procedure

New wind speed maps are given for each building category as follows:

1. Category I buildings—300 year return period
2. Category II buildings—700 year return period
3. Category III buildings—1700 year return period

Observe that for wind load calculations, Wind Importance Factor is no longer required. Serviceability maps for 10, 25, 50, and 100 year return intervals have been added in Appendix C.

5.11.1 New Wind Speed Maps

Wind speed maps have changed-again. Last time that happened was with the 1998 edition. In that year, ASCE 7 switched from the "fastest-mile" wind speed to the "3 s gust" wind speed. For instance, a 75 mph fastest mile became a 90 mph, 3 s gust wind speed overnight. The end result, however, did not necessarily change because the formulas used to translate wind speeds into design loads were also adjusted at the same time.

In the ASCE 7-10, two things are happening to the wind maps at once. On the one hand, the wind speeds are bumped up simply to incorporate the "load factor" that hitherto we were doing as a separate step. For example, in the load and resistance factor design, when you do your wind loading calculations, you multiply the wind loads by a load factor of 1.6. Now in the ASCE 7-10, that load factor effect is put into the wind map. So, instead of multiplying by 1.6, you multiply by one. You would get exactly the same answer using the new method as you had using the old method given in ASCE 7-05.

5.11.2 Return of Exposure *D*

New wind speed information has been put into new prediction models for hurricane wind probabilities. The latest analytical model is believed to be more complete and more accurate about the relationship between storm eyewall winds and eye pressures. It appears that wind speeds farther from a given storm's core actually tend to be lower than the previous model has assumed. So the likelihood of destruction at various parts of the shoreline is now predicted to be less than in past models. The new wind maps show these reduced wind speeds.

There is, however, one big exception to this general reduction in wind loads that applies to the strip of land right on the beach, within perhaps 600–1000 ft off the shore. New research data and modeling techniques have overturned another assumption about storms—the idea that high waves stirred by hurricanes slow the wind speed and reduce the wind hazard to buildings. It seems for very strong winds, the waves do not slow the wind by much—hurricane winds turn the water surface into a low friction, "froth," that has negligible effect in protecting buildings from the force of wind storms. The result of this new finding means Exposure Category D is back.

5.11.3 Wind-Borne Debris

The changing of wind speed values has some effect on certain locations near the beach—because they effect the triggers for wind-borne debris requirement. Under ASCE 7-10, the parts of coastline where buildings need impact-resistant glazing or storm shutters have shrunk.

The new standard has just come out in print starting the process of adopting the latest update into model building codes, and finally the adoption of those codes at state and local level.

6 Seismic Design

PREVIEW

Earthquakes are catastrophic events that occur mostly at the boundaries of portions of the earth's crust called tectonic plates. When movement occurs in these regions, along faults, waves are generated at the earth's surface that can produce very destructive effects.

Aftershocks are smaller quakes that occur after all large earthquakes. They are usually most intense in size and number within the first week of the original quake. They can cause very significant re-shaking of damaged structures, which makes earthquake-induced disasters more hazardous. A number of moderate quakes (6+ magnitude on the Richter scale) have had aftershocks that were very similar in size to the original quake. Aftershocks diminish in intensity and number with time. They generally follow a pattern of being at least 1 large (within magnitude 2 on the Richter scale) aftershocks, 100 within magnitude 3 on the Richter scale, and so on. The Loma Prieta earthquake had many aftershocks, but the largest was only magnitude 5.0, with the original quake being magnitude 7.1.

Some of the most destructive effects caused by shaking as a result of the earthquake are those that produce lateral loads in a structure. The input shaking causes the foundation of a building to oscillate back and forth in a more or less horizontal plane. The building mass has inertia and wants to remain where it is, and, therefore, lateral forces are exerted on the mass in order to bring it along with the foundation. For analysis purposes, this dynamic action is simplified as a group of horizontal forces that are applied to the structure in proportion to mass and to the height of the mass above the ground. In multistory buildings with floors of equal weight, the loading is further simplified as a group of loads, each being applied at a floor line, and each being greater than the one below in a triangular distribution. Seismically resistant structures are designed to resist these lateral forces through inelastic action and must, therefore, be detailed accordingly. These loads are often expressed in terms of a percent of gravity weight of the building and can vary from a few percent to near 50% of gravity weight. There are also vertical loads generated in a structure by earthquake shaking, but these forces rarely overload the vertical-load-resisting system. However, earthquake-induced vertical forces have caused damage to structures with high dead load compared to design live load. These vertical forces also increase the chance of collapse due to either increased or decreased compression forces in the columns. Increased compression overloads columns while decreased compression reduces column bending strength.

In earthquake engineering, we deal with random variables and therefore the design must be treated differently from the orthodox design. The orthodox viewpoint idealizes variables as deterministic. This simple approach is valid when applied to design under only mild uncertainty. But when confronted with the effects of earthquake design, we must contend with appreciable probabilities, particularly so when dealing with building failures, that may occur in the near future. Otherwise, all the wealth of this world would prove insufficient to fill our needs; the most modest structures would be fortresses. We must also face uncertainty on a large scale while designing engineering systems—whose pertinent properties are still debated, about whose characteristics we know even less.

Although over the years, experience and research have diminished our uncertainties and concerns regarding the characteristics of earthquake motions and its manifestations, it is unlikely, though, that there will be such a change in the nature of knowledge to relieve us of the necessity of dealing openly

with random variable. In a way, earthquake engineering is a parody of other branches of engineering. Earthquake effects on structures systemically bring out the mistakes made in design and construction, even the minutest mistakes. Add to this the undeniable dynamic nature of disturbances, the importance of soil–structure interaction and the extremely random nature of it all; it could be said that earthquake engineering is to the rest of the engineering disciplines what psychiatry is to other branches of medicine. This aspect of earthquake engineering makes it challenging and fascinating, and gives it an educational value beyond its immediate objectives. If structural engineers are to acquire fruitful experience in a brief span of time, expose them to the concepts of earthquake engineering, even if their interest in earthquake-resistant design is indirect. Sooner or later, they will learn that the difficulties encountered in seismic design are technically intriguing and begin to exercise that nebulous trait called engineering judgment to make allowance for these unknown factors.

Seismic design entrails determination of maximum response of a structure subjected to ground motions that vary in magnitude and direction. Design of structures to withstand earthquakes is technically invigorating because major challenges lie in the prediction of future earthquakes, their character and energy content imparted to the structure at the base, and the dynamic behavior of the structure itself.

A dynamic analysis differs from its counterpart in that it requires, instead of a single solution, a series of solutions at time intervals covering the entire duration of ground motions.

We begin this chapter with a detailed discussion of structural dynamics to demonstrate its use in the determination of elastic response of idealized buildings subject to prescribed ground motion. Using a single-bay, single-story shear frame as an analytical model, we describe fundamental concepts, such as dynamic equilibrium, dynamic load factor (DLF), and free and forced vibrations. Then we introduce the response spectrum concept, which is central to earthquake engineering, together with procedures to determine the peak response of systems directly from the response spectrum.

In the next section, we study how buildings respond when subjected to ground accelerations at the base. The influence of the building configuration and the dynamic characteristics of the soil it rests upon are discussed to set the stage for our understanding of why and how these are treated in seismic standards. Also discussed are the underlying concepts of ductility, redundancy, and damping. Next, we consider the design of diaphragms with an explanation of why the forces used in their design are substantially different from those used for the design of lateral-force-resisting systems. Also included is a discussion of responses of elements attached to buildings.

An overview of the seismic design requirements as prescribed in the ASCE 7-05 is the subject of the final section. Included in this section is a catalog of seismic design requirements for buildings assigned to Seismic Design Categories (SDCs) A, B, C, D, E, or F.

6.1 STRUCTURAL DYNAMICS

The primary purpose of this section is to present methods for analyzing the stresses and deflections developed in any given type of structure when it is subjected to an arbitrary dynamic loading. In one sense, the static-loading condition may be looked upon merely as a special form of dynamic loading. However, it is convenient to distinguish between the static and the dynamic components of the applied loading, to evaluate the response to each type of loading separately, and then to superpose the two response components to obtain their total effect. When treated thusly, the static and dynamic methods of analysis are fundamentally different in character.

The term *dynamic* may be defined simply as time varying; thus, a dynamic load is any load of which its magnitude, direction, and/or position varies with time. Similarly, the structural response to a dynamic load, that is, the resulting stresses and deflections, is also time varying or dynamic.

Even though seismic ground motions are highly oscillatory and irregular in character, the resulting loads may be considered as *deterministic dynamic loads*, as opposed to *random dynamic loads*. Therefore, the emphasis here is on development of methods of deterministic dynamic analysis, rather than on random or nondeterministic analysis that require statistical information about dynamic response quantities.

In general, structural response to any dynamic loading is expressed basically in terms of the displacements of the structure. Thus, a deterministic analysis leads directly to displacement time-histories corresponding to the prescribed loading history; other related response quantities, such as stresses, strains, internal forces, etc., are usually obtained as a secondary phase of the analysis. On the other hand, a nondeterministic analysis provides only statistical information about the displacements resulting from the statistically defined loading; corresponding information on the related response quantities are then generated using independent nondeterministic analysis procedures.

From an analytical point of view, it is convenient to divide deterministic dynamic loadings into two distinct categories, periodic and non-periodic. A periodic loading exhibits the same time variation successively for a large number of cycles as typified by the unbalanced-mass effects in a rotating machinery.

Non-periodic loadings, on the other hand, may be either short-duration impulsive loadings or relatively long-duration general forms of loads. A blast or explosion is a typical source of impulsive load; for such short-duration loads, special simplified forms of analysis may be employed. On the other hand, a general, relatively long-duration loading such as might result from an earthquake can be treated only by completely general dynamic analysis procedures.

6.1.1 Dynamic Loads

A good number of loads that occur in buildings may be considered stationary requiring only static analysis. Although almost all loads except dead loads are transient, meaning they change with time, customarily in structural design they are treated as static loads. For example, lateral loads due to transitory wind gusts that are often dynamic, are usually treated as static loads.

In this case, the dynamic characteristics due to the sudden variation of wind velocity are taken into account by including a gust factor in the determination of wind loads. Therefore, the analysis reduces to a static case requiring but one unique solution.

Now consider the same building, instead of being buffeted by wind, subjected to ground motions due to an earthquake. The seismic shock causes the foundation of the building to oscillate back and forth, principally in a horizontal plane. The building would follow the movement of the ground without experiencing lateral loads if the ground oscillation took place very slowly over a long period of time. The building would merrily ride to the new displaced position as if no load was ever applied to it. On the other hand, when the ground moves suddenly as in an earthquake, the building mass, which has inertia, attempts to stay in its pre-earthquake position thus resisting the accelerations of the structure.

In a building with several floors, the inertial forces may be visualized as a group of horizontal forces applied at various floors in proportion to the floor mass, and its height above the ground.

These earthquake forces are considered dynamic, because they vary with time. Since the load is time-varying, the response of the structure, including deflections, axial and shear forces, and bending moments, is also time dependent. Therefore, instead of a single solution, a separate solution is required to capture the response of the building at each instant of time for the entire duration of an earthquake. Because the resulting inertia forces are a function of building accelerations, which are themselves related to the inertia forces, it is necessary to formulate the dynamic problem in terms of differential equations.

6.1.1.1 Concept of Dynamic Load Factor

A special feature of earthquake excitation of structures is that it is applied in the form of support motions that are neither harmonic nor periodic. Defining such excitations that vary arbitrarily with time is perhaps the most difficult and uncertain phase of predicting structural response to earthquake-induced ground motions. However, when these motions have been established, the calculation of response of a given structures is a standard structural dynamics problem. It can be computed by superposing the responses calculated separately for each component of a multiple ground motion.

Simply stated, the DLF is the ratio of the dynamic response such as the deflection of an elastic system to the static deflection that results from the static application of a time-load function, F_t.

In a static problem, time variation of the load has no effect on the response since it assumed that the load is applied in a gradual manner over a long period of time. In dynamic problems however, the time it takes to apply the load has considerable influence on the structural behavior and thus must be given consideration in the analysis.

Shown in Figure 6.1 are some examples of time-load function, F_t, in which load variation is graphed with respect to time t. The graphs are for unit impulse force, step force, ramp or linearly increasing force, triangular and half-cycle sine pulse force. The determination of the dynamic response of simple systems subject to the time-load functions illustrated in Figure 6.1 constitutes the back bone of structural dynamics, as illustrated later in Section 6.1.6.2.

Consider the cantilever beam shown in Figure 6.2 with an empty container of weight W_D at the free end. Ignoring the self weight of the beam and using common notations, the deflection of the cantilever due W_D is given by

$$\Delta_c = \frac{W_D L^3}{3EI}$$

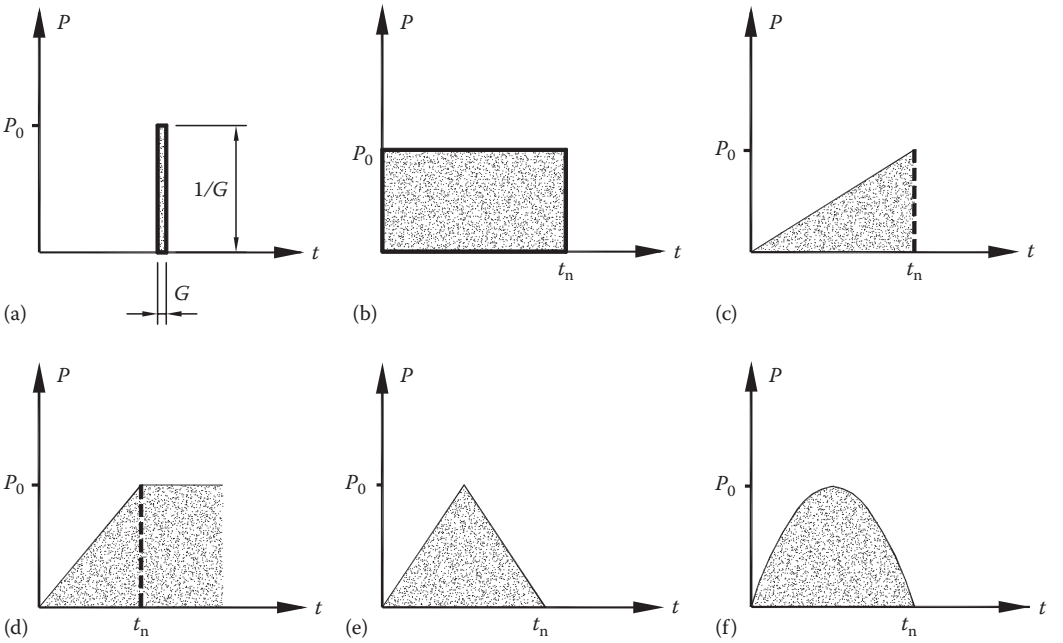


FIGURE 6.1 Time-load functions. (a) Unit impulse force, (b) step force, (c–d) ramp or linearly increasing force, (e) triangular pulse force, and (f) half-cycle sine pulse force.

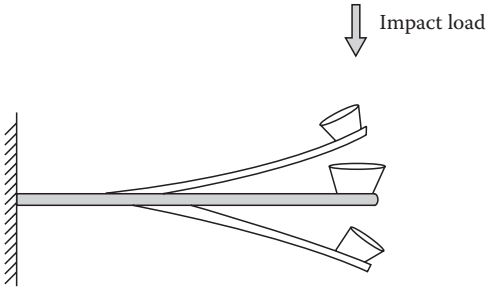


FIGURE 6.2 Dynamic response of a cantilever. *Note*: DLF = 2 for suddenly applied point load.

Let the container be filled with water of weight W_w, flowing gradually from a tap say, drop-by-drop into the container. The additional deflection due to the weight of water W_w is

$$\Delta_w = \frac{W_w L^3}{3EI}$$

Instead of a drop-by-drop loading, if all water is suddenly gushed into the container instantly, the cantilever deflection will no longer be the same as the static deflection Δ_w. We know by intuition, it will be larger than Δ_w (and in our case it can be shown that it is two times the static deflection Δ_w). Hence, DLF = 2. Moreover, the cantilever not only moves downward, it also springs up, and then down and continues to do so about its static position. In other words, the cantilever when subject to a sudden load, exhibits a dynamic response by vibrating typically in a sinusoidal manner. But for the effects damping, it would continue to do so indefinitely as shown schematically in Figure 6.3a, as opposed to damped vibrations shown in Figure 6.3b.

Earthquake ground motions are erratic; they are neither harmonic nor periodic and vary arbitrarily with time and last no more than a few seconds. The longest recorded earthquake lasted for about only 40 seconds although it may have seemed like an eternity for those experiencing the trauma. Although ground motions are unrhythmical in an earthquake, we may for analytical purposes, consider their effects as equivalent to summation of the responses to appropriately scaled forces such as those shown in Figure 6.1.

Thus, the response spectrum of single-degree-of-freedom systems to ground acceleration, as explained presently in this chapter, can be plotted from the DLFs determined for a chosen set of time-load functions such as those shown in Figure 6.1.

It should be noted that the DLF also varies with time (see Figure 6.4). However, in structural engineering we are interested in the maximum design values (such as displacements and stresses) that occur during an earthquake. Therefore, the maximum response, irrespective when it occurs during an earthquake, is of concern. It is worthwhile to note again that when we use response spectra to calculate design values for single-degree-of-freedom systems, it is not necessary to amplify the design values by DLF, because the computationally intensive dynamic analysis has been completed in generating the response spectra.

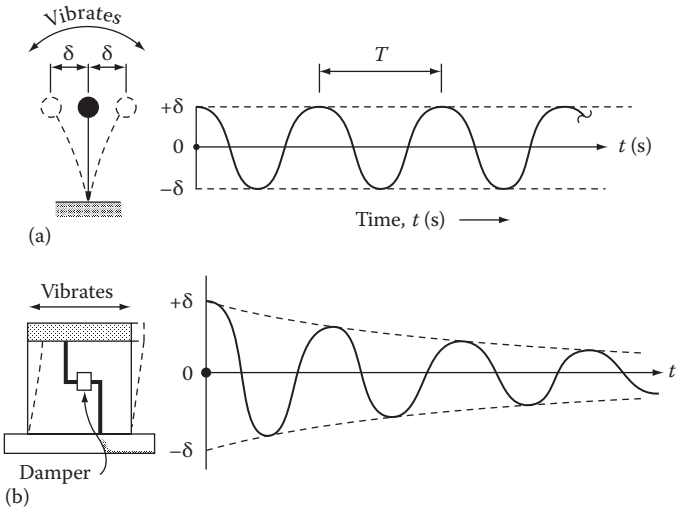


FIGURE 6.3 Vibrations of one-degree-of-freedom system: (a) undamped; (b) damped.

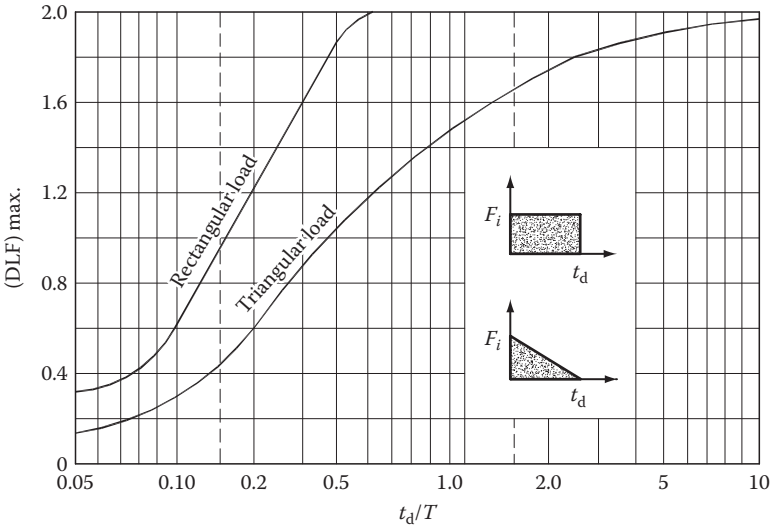


FIGURE 6.4 Maximum response of one-degree elastic systems (undamped) rectangular and triangular pulses having zero rise time. *Note*: T = period of one-degree-of-freedom system and t_d = pulse duration.

6.1.1.2 Difference between Static and Dynamic Analysis

In our day-to-day structural engineering practice, we are quite comfortable with the idea that if a member is overstressed, say in flexure, we simply increase the member size and thus its section modulus, to decrease the flexural stresses. This concept is as old as engineering itself, and has served us quite well in our design of structures subject to static loads. However, this thought of automatically reducing the stresses by increasing member sizes may not always hold together when designing structures subject to seismic ground motions.

In the case of earthquake excitation, the increase in member size resulting in higher stiffness shortens the natural vibration period that may have the effect of increasing the equivalent static force. Whether the corresponding stress decreases or increases by increasing the size of the member depends on the increase in section modules, S, and the increase in the equivalent static force that in turn, depends on the response spectrum.

Example

Given

A 15 ft tall cantilever, an 8 in. nominal diameter standard steel pipe supporting a 40 kip weight at the top as shown in Figure 6.5. The properties of the column are

$$I = 106 \text{ in.}^4$$

$$S = 24.5 \text{ in.}^3$$

$$\text{Weight } W = 43.39 \text{ lb/ft}$$

$$E = 29{,}000 \text{ ksi}$$

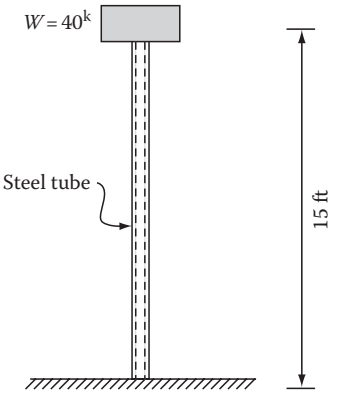


FIGURE 6.5 Cantilever column with weight at top.

Required

1. Determine the peak deformation and bending stress in the cantilever column using the seismic ground motion, acceleration response spectrum, for a damping β = 5%, shown in Figure 6.6.
2. Repeat the calculations using a bigger section, a 12 in. diameter standard pipe. Discuss the pros and cons of using a bigger pipe.

Solution

1. The weight of pipe at 43.39 plf = 15 × 43.39 = 650 lb. Compared to the load of 40 kip at the top of the cantilever, the self weight of the column is small and, therefore, can be ignored. The lateral stiffness of the column is

$$k = \frac{3EI}{h^3} = \frac{3 \times 29{,}000 \times 106}{(15 \times 12)^3} = 1.584 \text{ kip/in.}$$

$$\text{Mass at top} = \frac{W}{g} = \frac{40}{386} = 0.104 \text{ kip-s}^2/\text{in.}$$

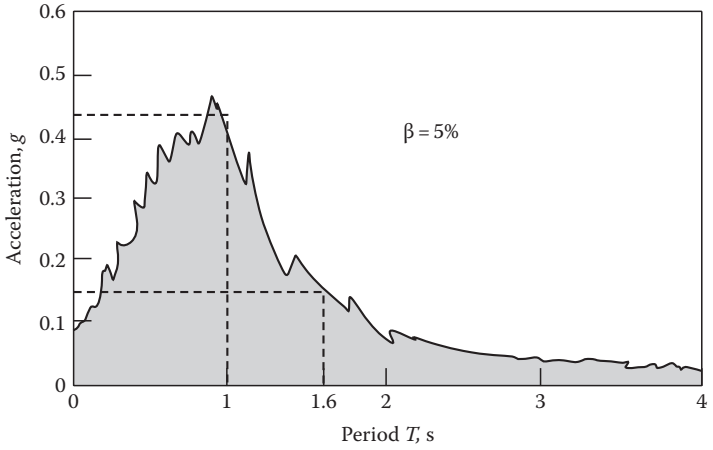


FIGURE 6.6 Acceleration response spectrum.

The natural vibration frequency is given by

$$\omega_n = \sqrt{\frac{k}{m}} = \sqrt{\frac{1.584}{0.104}} = 3.91\,\text{rad}$$

$$\text{Period } T_a = \frac{2\pi}{\omega_a} = \frac{2 \times 3.14}{3.91} = 1.6\text{ s}$$

From the response spectrum curve (Figure 6.6) for $T_a = 1.6$ s, acceleration $a = 0.15g$
The peak value of the equivalent static force is

$$v = \frac{a}{g} w = 0.15 \times 40 = 6\text{ kip}$$

The bending moment at the base of the column is

$$6 \times 15 = 90\,\text{kip-ft}$$

$$\text{Bending stress } f_b = \frac{90 \times 12}{24.5} = 44.08\text{ ksi}$$

2. Because the bending stress is relatively high, the designer elects to try a bigger pipe, a 12 in. diameter standard pipe. Let us verify if the resulting bending stress situation gets any better.
   The properties of the new column are

$$I = 279\text{ in.}^4$$

$$S = 43.8\text{ in.}^3$$

$$\text{Weight} = 49.56\text{ lb/ft}$$

As before, we ignore the self weight of the column.
The lateral stiffness in the new column is

$$k = \frac{3EI}{h^3} = \frac{3 \times 29{,}000 \times 279}{(15 \times 12)^3} = 4.17\text{ kip/in.}$$

$$\text{Mass at top} = 0.104\text{ kip-s}^2\text{/in., as before}$$

The natural vibration frequency is

$$w_n = \sqrt{\frac{k}{m}} = \sqrt{\frac{4.17}{0.104}} = 6.33\,\text{rad}$$

$$\text{Period } T_n = \frac{2\pi}{\omega_n} = \frac{2 \times 3.14}{6.33} = 0.992\text{ s} = \sim 1.0\text{ s}$$

From the response spectrum curve (Figure 6.6) for $T_n = 1$ s, acceleration $a = 0.43g$.

The peak value of the equivalent static force is

$$v = \frac{a}{g} w = 0.43 \times 40 = 17.2 \text{kip}$$

The bending moment at the base of the column is

$$17.2 \times 15 = 258 \text{ kip-ft}$$

$$\text{Bending stress } f_b = \frac{258 \times 12}{43.8} = 70.68 \text{ksi}$$

that is about 60% more than the calculated value for the 8 in. diameter pipe.

The above example (admittedly cooked up to make a point) would make you think twice before you add stiffness to seismic structural systems.

The concept of designing with sufficient strength has been viewed traditionally as a key to more effectively control the behavior of buildings. However, in certain seismic designs, the benefits associated with an increase in strength may be small or even have a negative impact. The doctored-up example given here is not intended to discount strength as an important design consideration but rather to allude to the possible negative impacts. Keep in mind, the excessive strength of the yielding element will impose more demand on the brittle components along the lateral load path. A better design strategy would be to increase ductility; treat ductility as a wealthy person treats money—you cannot have too much of it.

6.1.1.3 Dynamic Effects due to Wind Gusts

Perhaps the reader will recall that in Chapter 5, a brief statement was made regarding the effect of wind gusts on the dynamic response of tall buildings. It was stated that the load on a building due to a wind gust is a dynamic load if the rise time of the guest is considerably shorter that the fundamental period of the building. Conversely, if the rise time is the same as or higher than the building's period, it is a static load.

This characteristic which depends entirely on rise time of wind gust, T_g, is shown in Figure 6.7 for two values of T_g/T_B, the ratio of rise time T_g of the gust, to the fundamental period, T_B, of the building. Observe that for $T_g/T_B = 0.25$, (which signifies a wind gust of relatively long rise time), DLF is nearly equal to 1.

To understand the significance of these figures, consider a tall building with a period, T_B, of 5 s (frequency of 0.166 Hz) subject to a constant wind pressure of 30 psf, and then suddenly to a 1 s gust of 15 psf. A 1 s guest simply means that wind pressure has increased to 30 + 15 = 45 psf in a time-period of 1 s. In other words the 30 psf of constant pressure has increased to 45 psf, with the increase taking place in a short time interval of 1 s, and remains at 45 psf for the time duration of interest.

Referring to Figure 6.7a, for $T_g/T_B = 1/5 = 0.20$, the DLF is equal to 2.0 signifying that the dynamic effect of wind gust is as though it is an impulse load culminating with a maximum DLF.

Next consider the same increase in the wind load but assume that the increase occurs over a relatively long time-interval of say, 12.5 s. As can be seen in Figure 6.7, for the ratio $T_g/T_B = 12.5/5 = 2.5$, the DLF is close to unity, signifying that the dynamic effect can in fact, be ignored for this case.

The concept of DLF presented above for wind gust is equally applicable to seismic design, except the load-time functions are referenced to support accelerations.

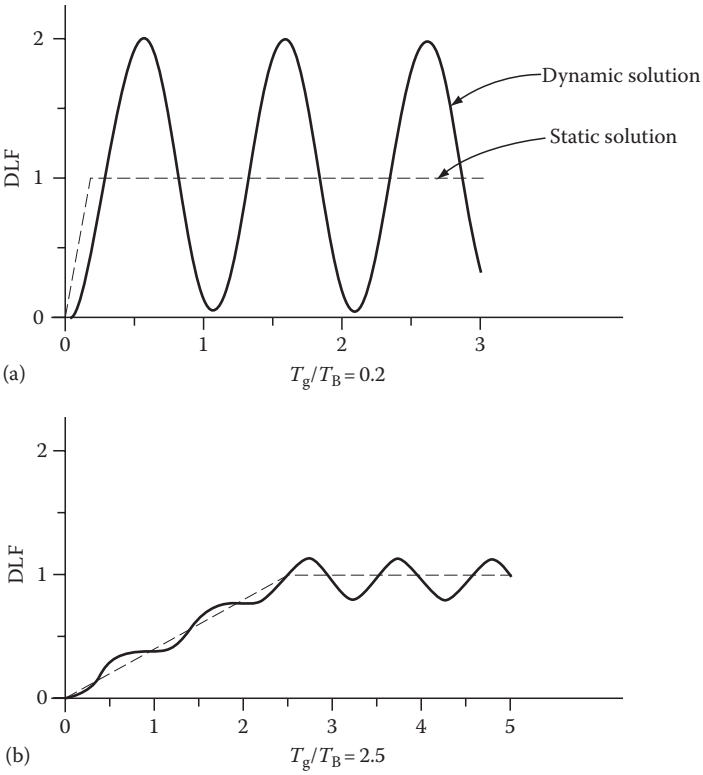


FIGURE 6.7 Dynamic response showing deflection load factor, DLF, for undamped SDF system to step-force with finite time: (a) $T_g/T_B = 0.2$; (b) $T_g/T_B = 2.5$. *Note*: DLF is 2 for 1 s gust and 1.0 for 12.5 s gust.

6.1.2 Characteristics of a Dynamic Problem

A structural dynamic problem differs from its static-loading counterpart in two important respects. The first difference, as implied by the definition, is the time-varying nature of the dynamic problem. Because both loading and response vary with time, it is evident that a dynamic problem does not have a single solution, as a static problem does; instead the engineer must establish a succession of solutions corresponding to all times of interest in the response history. Thus, a dynamic analysis is clearly more complex and time consuming than a static analysis.

The second and more fundamental distinction between static and dynamic problems is illustrated in Figure 6.8. If a single-bay, single-story frame is subjected to a static load F, as shown in Figure 6.8, the internal moments and shears in the columns and deflected shape of the frame depend only upon this load and they can be computed by established principles of force equilibrium. On the other hand, if the load is applied dynamically, as shown in Figure 6.9, the resulting displacements of the frame depend not only upon the load but also upon inertial forces resulting from ground accelerations. Thus, the corresponding internal moments and shears in the columns must equilibrate not only the externally applied force F but also the inertial forces resulting from the accelerations.

Inertial forces which resist accelerations of the structure in this way are the most important distinguishing characteristic of a structural-dynamics problem. In general, if the inertial forces represent a significant portion of the total load equilibrated by the internal elastic forces of the structure, then the dynamic character of the problem must be accounted for in its solution. On the other, if the motions are so slow that the inertial forces are negligibly small, the analysis of response for any desired instant of time may be made by static structural analysis procedures even though the load and response may be time varying.

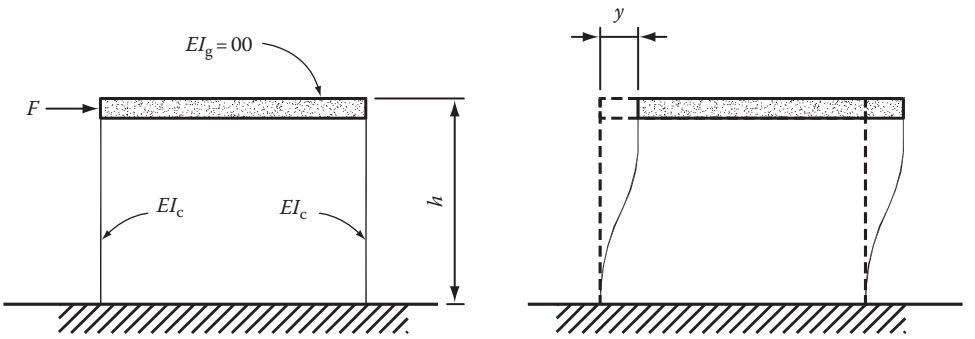


FIGURE 6.8 Portal frame subject to static loads. *Note*: $F = Ky$, $K = 24EI_c/h^3$.

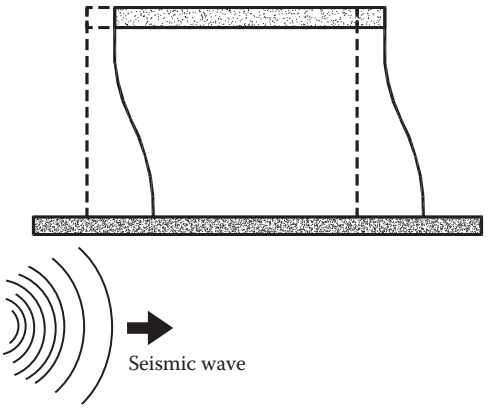


FIGURE 6.9 Portal frame subject to earthquake ground motions.

6.1.3 Multiple Strategy of Seismic Design

The design of structures to perform satisfactorily under expected seismic conditions requires that realistic earthquake loadings be specified and that the structural components be proportioned to resist these and other combined loads within the limits of certain design requirements. In regions of high seismicity, earthquake loading is often critical among the types of loading that must be considered because a great earthquake will usually cause greater stresses and deformations in the various critical components of a structure than will all other loadings combined; yet, the probability of such an earthquake occurring within the life of the structure is very low. In order to deal effectively with this combination of extreme loading and low probability of occurrence, a strategy based on the following dual criteria is adopted in most seismic standards:

1. A moderate earthquake which reasonably may be expected to occur once at the site of a structure during its lifetime is taken as the basis of design. The structure must be proportioned to resist the intensity of ground motion produced by this earthquake without significant damage to the basic system.
2. The most severe earthquake that could possibly ever be expected to occur at the site is applied as a test of structural safety. Because this earthquake is very unlikely to occur within the life of the structure, we as a society have economically justified in accepting significant structural damage; however, collapse and serious personal injury or loss of life must be avoided.

Currently, the trend is to strengthen the second of these criteria for critical and expensive structures by calling for limited repairable damage, thus focusing not only on life safety but on protection of financial investment as well.

A special feature of earthquake excitation of structures, compared with most other forms of dynamic excitation is that it is applied in the form of support motions rather than as external loads; thus, the effective seismic loadings must be established in terms of these motions. Defining the support motions is the most difficult and uncertain phase of the problem of predicting structural response to earthquakes. When these input motions have been established, however, the calculation of the corresponding stresses and deflections in any given structure is a standard problem in structural dynamics.

The earthquake excitation considered to act on a structure is the free-field ground motion at support points. Inherent in this treatment is the assumption that the same free-field ground motion acts simultaneously at all support points of the structure. This assumption is equivalent to considering the foundation soil or rock to be rigid. This hypothesis clearly is not consistent with the concept of earthquake waves propagating through the earth's crust from the source of energy release; however, if the base dimensions of the structure are small relative to the predominant wavelengths, the assumption is acceptable.

When specifying input motions at the base of a structure, it should be recognized that the actual structure base motions during an earthquake may be significantly different from the corresponding free-field motions that would have occurred without the structure being present. This "soil–structure interaction" effect will be of slight importance if the foundation is relatively stiff and the structure is relatively flexible; in this case, the structure transmits little energy into the foundation and the free-field motions are adequate measures of the actual foundation displacements. On the other hand, if a stiff structure is supported on a deep, relatively soft layer of soil, considerable energy will be transferred from the structure to the soil and the base motions will differ from those experienced by the soil under free-field conditions. This soil–structure interaction mechanism is independent of, and in addition to, the effect of local soil conditions on the free-field motions.

6.1.3.1 Example of Portal Frame Subject to Ground Motions

Given

A moment frame with bay lengths $L = 16\,\text{ft}$, height $h = 12\,\text{ft}$, and elastic modulus $E = 29{,}000\,\text{ksi}$. I_c and I_b for columns and beams are, respectively, xx and yy. The columns are fixed at base. The columns are $W8 \times 67$ with $I_c = 272\,\text{in.}^4$

Required

The beams are relatively heavy, $W36 \times 300$ with $I_b = 20{,}300\,\text{in.}^4$ Assuming the beam is infinitely rigid as compared to the column, compute the displacement at top and shear at the base due to an earthquake having an acceleration response spectrum shown in Figure 6.10. Assume the effective weight including the weight of beam is equal to 596 kip.

Solution

The first step is to determine the vibration period of the structure. The circular frequency of the structure is given by

$$\omega = \sqrt{\frac{k}{m}}$$

where

ω is the circular frequency
k is the stiffness of the rigid frame
m is the mass

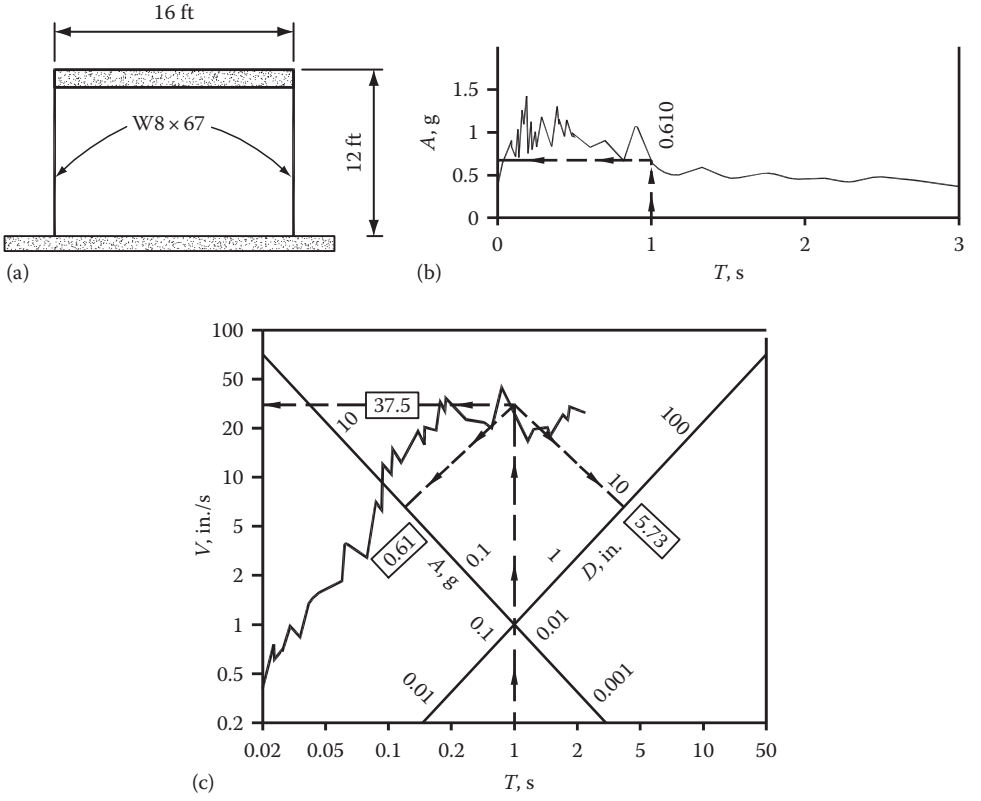


FIGURE 6.10 Example of portal frame subject to ground motions. (a) Portal frame, (b) acceleration spectrum, and (c) tripartite (DVA) spectrum.

Because the beam is considered rigid, that is, its flexural stiffness is large as compared to the column stiffness, the lateral stiffness, k, of the frame is

$$k = \sum \frac{12EI_c}{h^3}$$

$$= \frac{2 \times 12EI_c}{h^3} = \frac{24 \times 29{,}000 \times 272}{(144)^3} = 63.4 \text{ kip/in.}$$

The mass

$$m = \frac{W}{g} = \frac{596}{386.4} = 1.54 \text{ kip-s}^2/\text{in.}$$

$$\omega = \sqrt{\frac{k}{m}} = \sqrt{\frac{63.4}{1.54}} = 6.41 \text{ rad/s}$$

$$\text{Period } T = \frac{2\pi}{\omega} = \frac{2 \times 3.1416}{6.41} = 0.98 \text{ s} \cong 1 \text{ s}$$

From Figure 6.10, the spectral acceleration for the period of 1 s is 0.61*g*. Hence, the base shear

$$V = 0.61 \times 596$$

$$= 363.6 \text{ kip}$$

$$\text{The displacement at top} = \frac{V}{k}$$

$$= \frac{363.6}{63.4} = 5.73 \text{ in.}$$

Shown in Figure 6.10c is a tripartite response spectrum of the same earthquake shown in Figure 6.10b. The procedure for developing tripartite response spectrum is explained presently in Section 6.1.7.5. Suffice here to note that the displacement for the example portal frame can be read directly from the response spectrum without performing the calculations.

6.1.4 Concept of Dynamic Equilibrium

We begin our study of structural dynamics with a simple single-story portal frame shown in Figure 6.11, where the mass is distributed along the girder and only horizontal motions are considered. The supporting structure consisting of the beams and columns is assumed mass-less. The rigid frame is defined as a one-degree system in which only one type of motion is possible, or in other words, the position of the mass at any instant can be defined in terms of a single coordinate, which in our case, is the horizontal displacement u. Thus, the mass can move in a horizontal direction only. As an example of dynamic analysis, let us determine the motion of this mass resulting from the application of a time-varying horizontal force.

A convenient method of deriving the equation of motion is by the use of d'Alambert's principle of *dynamic equilibrium*. Having been trained to think in terms of equilibrium of forces, structural engineers may find this method particularly appealing. This principle states that with *inertia forces* included, a system is in dynamic equilibrium at each time instant. The inertia force, a force equal to the product of mass times its acceleration and acting in a direction opposite to the acceleration, is a fictitious force that allows us to treat structural dynamics in exactly the same manner as a problem in static equilibrium. From Figure 6.12, the equilibrium equation is

$$F_t - ku - M\ddot{u} = 0$$

where

F_t is the time-varying horizontal force
k is the lateral stiffness of the portal frame
δ is the horizontal displacement of the mass
M is the mass of the system = W/g (also denoted by m)
$\ddot{u}$ is the time-dependent acceleration of the mass M

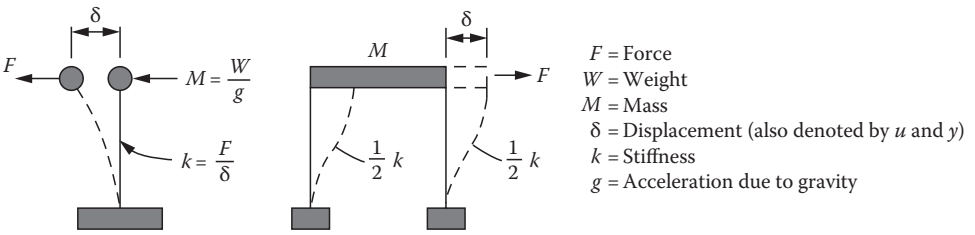


FIGURE 6.11 Idealized single-degree-of-freedom system.

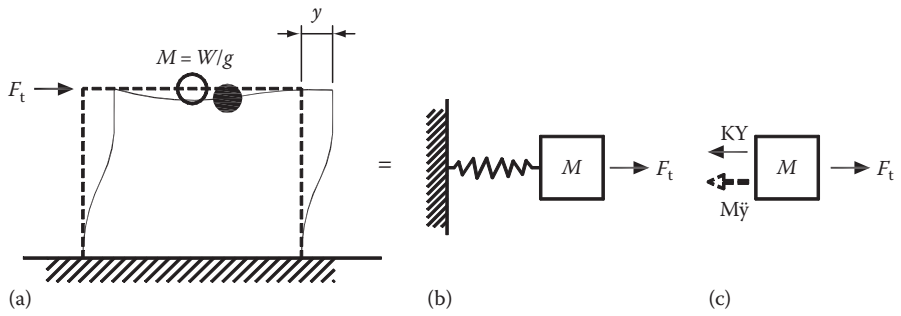


FIGURE 6.12 D'Alembert's principle of dynamic equilibrium: (a) Portal frame subject to dynamic force F_t; (b) equivalent dynamic model; (c) free-body diagram.

Throughout this text, $\dot{u}$ and $\ddot{u}$ will be used to designate the first and second derivatives of displacement u with respect to time t. In other words, $\dot{u} = du/dt$ = the velocity, and $\ddot{u} = du^2/d^2t$ = the acceleration.

The rearranged and expanded form of the equilibrium equation is

$$\frac{\delta^2 u}{dt^2} + \frac{k}{M}u = F_t$$

This is a differential equation of second order which may be solved by using the principles of calculus. However, the solution of the differential equations will not be discussed herein since it is assumed that the reader is familiar with such procedures. The deliberate omission of mathematical techniques in this text enables us to concentrate on the physical phenomena involved and helps us to develop a physical feel and intuition for dynamic response, which is necessary for successful analysis of more complicated dynamic problems.

6.1.5 Free Vibrations

Consider, first, the elementary case where F_t equals zero, that is, there is no external dynamic excitation. Such a structure is said to be undergoing *free vibration* when it is disturbed from its static equilibrium position and then allowed to vibrate. Motion occurs only if the system is given an initial disturbance, such as an initial displacement u_o. Imagine that the mass m is shown in Figure 6.13 is pulled horizontally by a rope and then suddenly released at $t = 0$. The resulting motion unaffected by any external force is the free vibration. The differential equation of motion for this case is given by

$$\ddot{u} + \frac{k}{m}u = 0$$

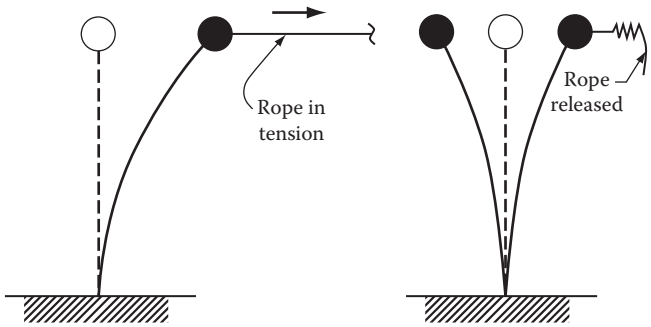


FIGURE 6.13 Concept of free vibrations.

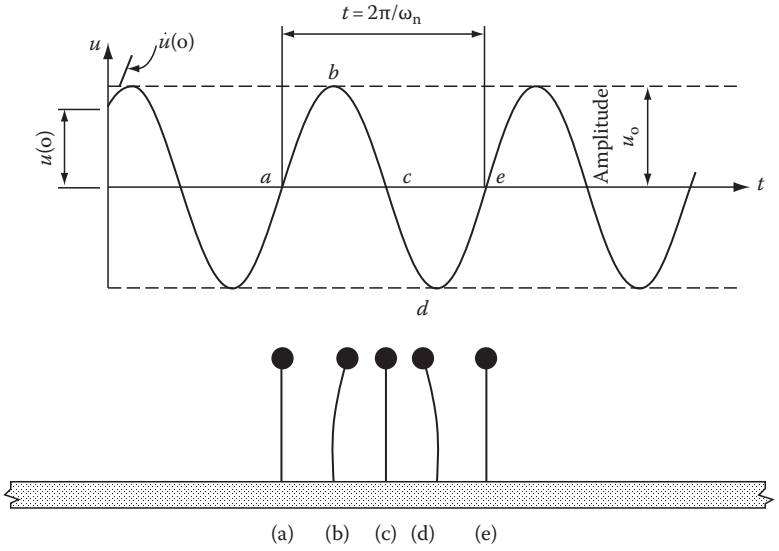


FIGURE 6.14 Displacement plot of free vibrations with damping.

The solution for this equation in terms of displacement u is given by

$$u = \frac{\dot{u}_o}{w} \sin \omega t + u_o \cos \omega t$$

A plot of displacement for initial displacement u_o is given in Figure 6.14. The parameter ω in the above equation is called the natural frequency given by the relation

$$\omega = \sqrt{\frac{k}{M}} \text{ rad/s}$$

As stated previously, d'Alambert's principle, named after its discoverer, the French mathematician Jean le Rond d'Alambert, permits the reduction of a problem in dynamics to one in statics. This is accomplished by introducing a fictitious force equal in magnitude to the product of the mass of the body and its acceleration, and directed opposite to the acceleration. While d'Alambert's principle is merely another way of writing Newton's second of motion, $F = Ma$, it has the advantage of changing a problem in engineering dynamics into a problem in statics.

6.1.6 Earthquake Excitation

The difficulties encountered with the design of structures to withstand earthquake-induced ground motions of the base structure are technically intriguing. The major problem lies in the prediction of the character and intensity of the earthquakes to which a structure might be subjected to during its life. Another difficulty lies in the fact that a realistic analysis for earthquakes should account for in elastic behavior of the structure because very few structures could withstand a strong earthquake without some plastic deformation. In this section, attention will be restricted to a few basic concepts which will provide a foundation for understanding seismic design principles discussed later in this chapter.

6.1.6.1 Single-Degree-of-Freedom Systems

Consider a portal frame subject to a displacement of the ground denoted by u_g. The total displacement u^t of the mass at each instant of time is given by

$$u^t(t) = u(t) + u_g(t)$$

The rigid body component of the displacement of the structure produces no internal forces, only the relative motion u between the base and the mass produces elastic forces.

Thus, the relative displacement $u(t)$ of the structure due to ground acceleration $\ddot{u}(t)$ will be identical to the displacement $u(t)$ of the structure if its base were stationary and if it were subjected to an external force = $m\ddot{u}_g(t)$ acting in a direction opposite to the ground motion. The ground acceleration can therefore be replaced by an *effective earthquake force*

$$F_{eff}(t) = -m\ddot{u}_g(t)$$

This force is equal to mass times the ground acceleration acting opposite to its acceleration.

The dynamic excitation at the base of a structure is neither harmonic nor periodic. Thus, we are interested in studying the dynamic response of structures subject to excitations varying with time.

It was seen earlier that the deflection of the cantilevers due to sudden application of the load is twice as much as the static deflection. If we knew this information, beforehand, we could have obtained the maximum dynamic response without going through the dynamic analysis procedure. However, the procedure, in a manner of speaking, is still a dynamic analysis, because it uses the vibration properties of the structure and the dynamic characteristics of ground motion. It is just that the engineers do not have to carry out any excruciating dynamic analysis. Somebody has done these in telling us that DLF = 2 for this particular load function. The argument also holds good for the response spectrum that we use in earthquake design.

The beam-to-column stiffness ratio, ρ, indicates how much the system may be expected to behave as a frame. For ρ = 0, the beams impose no restraint on joint rotations behaving as if they were pin-connected to the columns. The portal frame behaves as if it consists of two independent cantilever columns fixed at the base. For ρ = 00, the beams restrain completely the joint rotations, and the portal frame behaves as a shear beam with double curvature bending of the columns in each story. An intermediate value of P represents a combination of both the cantilever binding mode of columns and the shear mode deformations due to bending of beams. It should be noted that a frame with stiffer and stronger columns constitutes typical earthquake-resistant construction.

6.1.6.2 Numerical Integration, Design Example

Anyone who tries to explain the concept of numerical integration in less than 30 pages is asking for trouble. Here we take the risk and go for considerably fewer pages by limiting the description to the dynamic response of single-degree-of-freedom, SOF systems. The reader is referred to the vast body of literature that exists in several textbooks for further study.

Structural dynamics is too often taught as a course in advanced mathematics making this approach unnecessarily difficult for practicing engineers. Many engineers may find the mathematical manipulation so intriguing that they fail to develop physical understanding so essential to good design. In this text, we avoid mathematical complexities, which although are useful in understanding advanced dynamic topics, are not necessary for developing an insight into practical seismic designs.

Today in the year 2010, use of closed-form solutions for analyzing practical dynamic problems, very rarely if ever, are undertaken. As in other fields, most often numerical analysis techniques using computers are used to execute solutions to dynamic problems. However, the author believes that summarizing a numerical analysis executed by hand develops a physical feel for dynamic behavior much more rapidly than does the closed-form solution using differential equations.

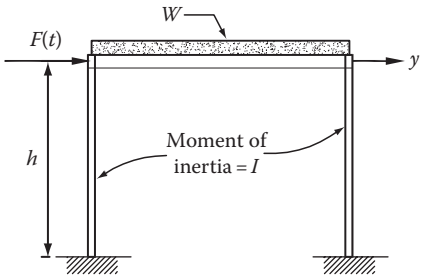


FIGURE 6.15 Portal frame subject to ground motions.

Consider a single-degree-of-freedom system, such as the portal frame shown in Figure 6.15. In terms of dynamic behavior, the system is equivalent to a spring connected to a mass, a model most often illustrated in structural dynamics textbooks. Let us recognize at the outset that this simple system is not only just an ideational model but in fact represents practical single-bay single-story portal frames.

To demonstrate the numerical integration procedure, suppose that the base of the portal frame is subjected to a given set of ground accelerations. Let us assume further, that the accelerations result in an equivalent dynamic load at the top, as shown in Figure 6.16 by the load-time function. To simplify the numerical solution, we will make two assumptions: first, the weights of the columns are negligible, and second, the girder is sufficiently rigid to prevent significant rotation at the top of the columns. These assumptions are not necessary for demonstrating the analysis, but will serve to simplify the problem and, in fact, are essentially correct for many actual frames of this type.

In the load-time function shown in Figure 6.16, the dynamic load is equal to 50 lb at the time $t = 0$ s, rising linearly to 100 lb at $t = 0.10$ s, and dropping back to 5000 lb at time $t = 0.14$ s. The purpose here is to determine the variation of the horizontal displacement of the beam, with time, starting with the system at rest $t = 0$, all the way to $t = 0.14$ s.

The example portal frame is a one-degree-of-freedom system defined as one in which only one type of motion is possible. Or in other words, the position of the system at any instant can be defined in terms of a single coordinate. In our case, the mass m can move in a horizontal direction only.

Before considering a specific example we shall discuss the process of numerical integration in general terms. This is a procedure by which the differential equation of motion is solved step by step, starting at zero time, when the displacement and velocity are presumably known. The timescale is divided into discrete intervals, and one progresses by successively extrapolating the displacement from one time station to the next.

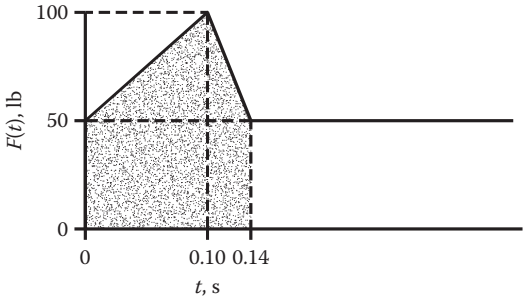


FIGURE 6.16 Equivalent load-time function due to base motions.

There are many such methods available for extrapolating a given mathematical function. One of the simpler versions presented shortly in Section 6.1.6.3 is called the *constant velocity or lumped-impulse procedure*. The reader is referred to the publication "Structural Dynamics" by John M. Biggs for further explanation of this procedure.

To illustrate the numerical integration procedures consider again the portal frame subjected to the load-time function shown in Figure 6.16. It is desired to determine the variation of displacement with time, starting with the system at rest at $t = 0$. Substituting into the equilibrium equation gives earlier in Section 6.1.3.1, and being careful to keep the units consistent, the equation of motion is written as follows:

$$F(t) - 2000y = \frac{64.4}{32.2}y$$

or

$$y = \frac{1}{2}F(t) - 1000y \quad (\textit{Note}: u \text{ and } \delta \text{ are also used to denote displacements.})$$

Thus, knowing the load and the displacement at any time enables one to compute acceleration at the time.

The next step is to select a time interval for the numerical integration. As mentioned above, this should not be greater than one-tenth of the natural period of the system. The *natural period T* of a one-degree system given by

$$T = 2\pi\sqrt{\frac{W}{kg}}$$

which, in this example, is 0.198 s. The *natural frequency f* is the inverse of the natural period, or 5.04 cps. The *natural circular frequency* ω is $2\pi f$, or 31.6 rad/s.

One-tenth of the natural period is approximately 0.02 s, and this value will be used in the computation. However, a second criterion must be considered when selecting the time interval: the interval should be small enough to represent properly the variation of load with time. In this example it will be noted that the stations (i.e., $\Delta = 0.02$) occur at the sudden breaks in the load function and, furthermore, that the interval is small enough so that points at this spacing accurately represent the load function (Figure 6.16). Therefore, the time interval selected is satisfactory.

The result of the complete calculation is plotted in Figures 6.17 and 6.18 where displacement versus time is shown. The ordinate also represents the time variation of the spring force if the displacements are multiplied by the spring constant k. To assist in the interpretation of the result, the hypothetical displacement corresponding to the static application of the load at any instant is also plotted. The maximum displacement which occurs at 0.12 s corresponds to a spring force of 159 lb which is 1.59 times the maximum external load. Subsequent peaks are somewhat smaller and correspond to a spring force of 157 lb the latter peak value would remain constant indefinitely since we have not included damping in the present example. The time interval between successive positive (or negative) peaks is exactly equal to the natural period of the system. After the load becomes constant at 0.14 s, the spring force caries in sinusoidal fashion, the positive and negative peaks being equidistant above and below the value of the external load.

The parameters of the idealized system need in performing the numerical integration are as follows:

$$W = 64.4 \text{ lb}$$

I, s	$1/2F(t)$, ft/s²	$1000y$, ft/s²	y, Eq. ft/s²	$y\,(\Delta t)^2$, ft	y, ft
0.0	25	0	25.0	0.0100	0
0.02	30	5.0	25.0	0.0100	0.0050*
0.04	35	20.0	15.0	0.0060	0.0200
0.06	40	41.0	−1.0	−0.0004	0.0410
0.08	45	61.6	−16.6	−0.0066	0.0616
0.10	50	75.6	−25.6	−0.0102	0.0756
0.12	37.5	79.4	−41.9	−0.0168	0.0794
0.14	25	66.4	−41.4	−0.0166	0.0664
0.16	25	36.8	−11.8	−0.0047	0.0368
0.18	25	2.5	22.5	0.0090	0.0004
0.20	25	−22.8	47.8	0.0191	−0.0228
0.22	25	−29.0	54.0	0.0216	−0.0290
0.24	25	−13.6	38.6	0.0154	−0.0136
0.26	25	17.2	7.8	0.0031	0.0172
0.28	25	51.1	−26.1	−0.0104	0.0511
0.30	25	74.6	−49.6	−0.0198	0.0746
0.32	25	78.3	−53.3	−0.0123	0.0783
0.34	—	—	—	—	0.0607

FIGURE 6.17 Numerical integration results.

$$k = \frac{2EI}{h^3} = 2000 \text{ lb/ft}$$

The spring constant k is simply equal to the inverse of the deflection at the top of the frame due to a unit horizontal load, and the equation employed above may be easily verified by simple elastic analysis. Having these parameters, we would then proceed to determine the dynamic displacements for the given load function. These displacements would be equal to the actual horizontal deflections at the top of the frame. The spring force computed for the ideal system would at all times be equal to the total shear in the two columns, and the maximum column bending moment would be give by $6EIy/h^2$. Thus, the dynamic stresses at any time could be easily determined.

6.1.6.3 Numerical Integration: A Summary

The process of numerical integration, also referred to as time-history analysis is a method in which the differential equation of motion is solved step-by-step, starting at zero time, when the displacement and velocity are presumably known. The time duration of earthquake of interest is divided into discrete intervals, Δt, and we progress by successively extrapolating the displacement from one time station to the next.

There are many methods available for successive extrapolation of displacements but only one of the more simple versions called the constant-velocity technique is discussed here.

Suppose that the curve shown in Figure 6.18B represents the displacement-time variation for a dynamic system. Suppose, further, that a successive numerical analysis for the determination of displacement is in progress with the displacements x_t at time station t, and x_{t-1} at time station $(t - 1)$ previously known. Our task is to determine the next displacement at station $(t + 1)$, by extrapolation. This could be done by the following self-evident formula

$$x_{(t+1)} = x_t + \dot{x}_{ay}\Delta t \tag{6.1}$$

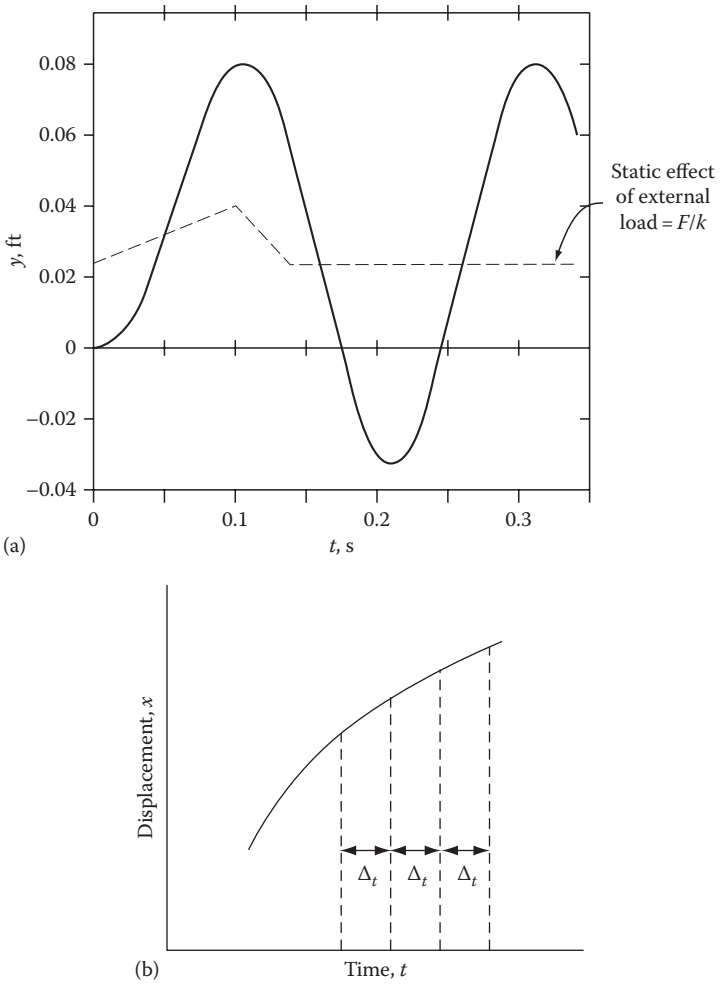


FIGURE 6.18 (a) Dynamic response of portal frame showing plot of displacement versus time. (b) Numerical integration, lumped-impulse procedure.

where

x is the average velocity between time stations t and $(t + 1)$

Δt is the time interval between the stations

$\dot{x}$ is the average velocity between the time interval between the stations

x is given by the relation

$$\dot{x} = \frac{x_t - x_{(t-1)}}{\Delta t} + \dot{x}_t \Delta t \tag{6.2}$$

Substituting Equation 6.2 into Equation 6.1, the following recurrence formula is obtained:

$$x_{(t+1)} = 2x_t - x_{t-1} + \ddot{x}_t (\Delta t)^2 \tag{6.3}$$

The acceleration $\dot{x}_t$ at time station t can be determined by the use of d'Alambert's principle of dynamic equilibrium:

$$F_t - kx_t = M\ddot{u}_t$$

In keeping with the objective stated at the beginning of this section, we will stop right here with our discussion of numerical integration. Suffice it to note that a recurrence formula such as the one given above, plays a pivotal role in time-history analysis. The reader is referred to standard textbooks on structural dynamics for an in-depth discussion of numerical integration techniques.

6.1.6.4 Summary of Structural Dynamics

1. A very convenient way of writing the equation of dynamic motion is by the use of d'Alambert's principle of dynamic equilibrium. In this method, an imaginary force, referred to as inertia force, and equal to the product of mass m and acceleration a is added to the static equilibrium condition in the direction opposite to positive displacement.
2. The natural period T of a one-degree system is given by

$$T = 2\pi\sqrt{\frac{W}{kg}}$$

3. Motion that occurs only if the system is given an initial disturbance such as an initial displacement, and is unaffected by any external force is called free vibration.
4. Motion that occurs as a result of an applied force is called forced vibration.
5. DLF for a given load F_1 is defined as the ratio of dynamic deflection to the deflection which would have resulted from the static application of the load F_1. DLF varies as the ratio t_x/T where t_x is the duration of load and T is the fundamental period of a single-degree-of-freedom system. The time at which the DLF becomes a maximum is also a variant but is of little consequence in seismic design because we are interested only in the maximum values such as deflection and stresses, and not in the time when they occur. Numerical values of DLF for representative load functions are given in Figure 6.22a through d.
6. Damping is conventionally indicated by a dash pot. It produces a force $c\dot{y}$, which opposes the motion and dissipates some of the energy of the system. Damping is the process by which vibration steadily diminishes in amplitude. In damping, the kinetic energy and strain energy of the vibrating system are dissipated by various mechanisms and should be included in the structural idealization in order to incorporate the feature of decaying motion. Damping has the effect of lengthening the period of a system. However, these effects are negligible for damping ratios below 20%. Observe that in seismic design the damping ratios used for most structures is in the range of 5%–10%.
7. The determination of building response by numerical procedures involves the time-wise sequential determination of system's velocities and displacements. This type of numerical procedures in structural engineering is often referred to as a time history analysis. The prediction of response by time history analysis allows the designer to develop a physical feeling as to why a building responds as it does when subjected to ground motion.

In order to develop a physical feeling for dynamic behavior, consider what happens to our portal frame that has columns hinged to the ground, when the ground moves. Assume that you are sitting at the top of the beam and an earthquake occurs. The ground moves, but you do not, at least for the first small interval of time. You will, of course, move along with ground motions right after the first interval of time.

The initial differential displacement, d_o, between the mass (that is you sitting on the beam) and the ground causes strain energy to be stored in the columns. The displacement may be, for analytical purposes, considered as that due to an equivalent force F_o at top.

F_o is given by

$$F_o = Kd_o = \frac{2 \times 3EI}{h^3}(d_o)$$

This force, F_o, which is in dynamic equilibrium, is given by Newton's second law of motion. Force equals mass times acceleration,

$$F_o = M\ddot{x}_o = \frac{W}{g}\ddot{x}_o$$

The acceleration you feel during the ride is given by

$$\ddot{x}_o = \frac{Kd_o}{M}$$

The inertial force imparted on the mass is given by $F = k(d - x)$. Using subscripts to identify sequential increments, the acceleration, $\ddot{x}$, experienced by the mass is $\ddot{x}_i = k/M(d_i - x_i)$.

Once we know the acceleration the mass will experience, we can determine the velocity and displacement this acceleration history will produce on the mass. The velocity ($\dot{x}$) and displacement (x) of the mass can be calculated through the use of integral calculus if the acceleration can be described in terms of a continuous function.

Earthquake motions are erratic. It is not always possible to describe ground accelerations as continuous functions. Therefore, numerical integration is a more appropriate means of predicting successive velocities and displacements. The change in velocity experienced by the mass during a time interval of Δt is then determined by multiplying the average acceleration, which occurs during this time interval by the interval (Δt). The change in displacement is determined by multiplying the average velocity experienced by the mass during the time interval (Δt). Several numerical methods can be used to predict incremental changes in velocity and displacement.

It should be noted that given a small enough time interval, these methods will describe the characteristics of the dynamic response and maximum values sought in structural design, to a degree of accuracy consistent with our ability to predict ground motions.

In order to perform response history analysis, it is necessary to have a digitized ground motion acceleration record. In linear response history analysis, the stiffness of the structure, K, is assumed to be independent of the prior displacement history. In nonlinear response history analysis, the structure's stiffness at an instance of time t, is dependent on the displacement history up to that point in time and varies to account for yielding, buckling, and other behaviors that may have occurred earlier in the structure's response.

Response history analysis is useful because it allows solutions of the deflected shape and force state of the structure at each instant of time during the earthquake. Since each earthquake record has different characteristics, the results obtained from response history analysis are valid only for the particular earthquake record analyzed. Therefore, when performing response history analysis to determine forces and displacements for use in design, it is necessary to run a suite of analyses, each using different ground motion records as input. Present building codes require a minimum of three records. If three records are used, the maximum forces and displacements obtained from any of the analyses must be used for design purposes. If seven or more records are used, the code permits use of the mean forces and displacements obtained from the suite of analyses.

In design practice, linear response history analysis is seldom used. This is because for design purposes, one is usually interested only in the maximum values of the response quantities (forces and displacements) and these quantities can more easily be approximated by an alternative form of analysis known as response spectrum analysis. However, nonlinear response history analysis is an

essential part of the design of structures using seismic isolation or energy dissipation technologies, and it can be quite useful in performance-based design approaches.

6.1.7 Response Spectrum Method

The word spectrum in seismic engineering conveys the idea that the response of buildings having a broad range of periods is summarized in a single graph. For a given earthquake motion and a percentage of critical damping, a typical response spectrum gives a plot of earthquake-related responses such as acceleration, velocity and deflection for a complete range, or spectrum, of building periods. An understanding of the concept of response spectrum is pivotal to performing seismic design.

A response spectrum (Figures 6.19 and 6.20) may be visualized as a graphical representation of the dynamic response of a series of progressively long cantilever pendulums with increasing natural periods subjected to a common lateral seismic motion of the base. Imagine that the fixed base of the cantilevers is moved rapidly back and forth in the horizontal direction, its motion corresponding to that occurring in a given earthquake. A plot of maximum dynamic response, such as accelerations versus the periods of the pendulums, gives us an acceleration response spectrum as shown in Figures 6.19 and 6.20a and c, for the given earthquake motions. In this figure, the absolute value of the peak acceleration occurring during the excitation for each pendulum is represented by a point on the acceleration spectrum curve. As an example, an acceleration response spectra from the 1940 El Centro earthquake is illustrated in Figure 6.21. Using ground acceleration as an input, a family of response spectrum curves can be generated for various levels of damping, where higher values of damping result in lower spectral response.

To establish the concept of how a response spectrum is used to evaluate seismic lateral forces, consider two, single-degree-of-freedom structures: (1) an elevated water tank supported on columns; and (2) a revolving restaurant supported at the top of a tall concrete core (see Figure 6.22). We will neglect the mass of the columns supporting the tank, and consider only the mass m_1 of the tank in the dynamic analysis. Similarly, the mass m_2 assigned to the restaurant is the only mass considered in the second structure. Given the simplified models, let us examine how we can calculate the lateral loads for both these structures resulting from an earthquake, for example, one that has the same ground motion characteristics as the 1940 El Centro earthquake shown in Figure 6.23. To evaluate the seismic lateral loads, we shall use the recorded ground acceleration for the first 30 s. Observe that the maximum acceleration recorded is $0.33g$ occurring about 2 s after the start of the record.

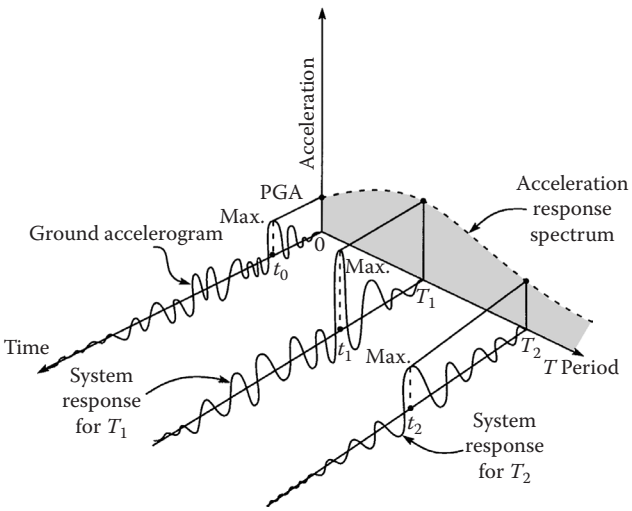


FIGURE 6.19 Graphical description of response spectrum.

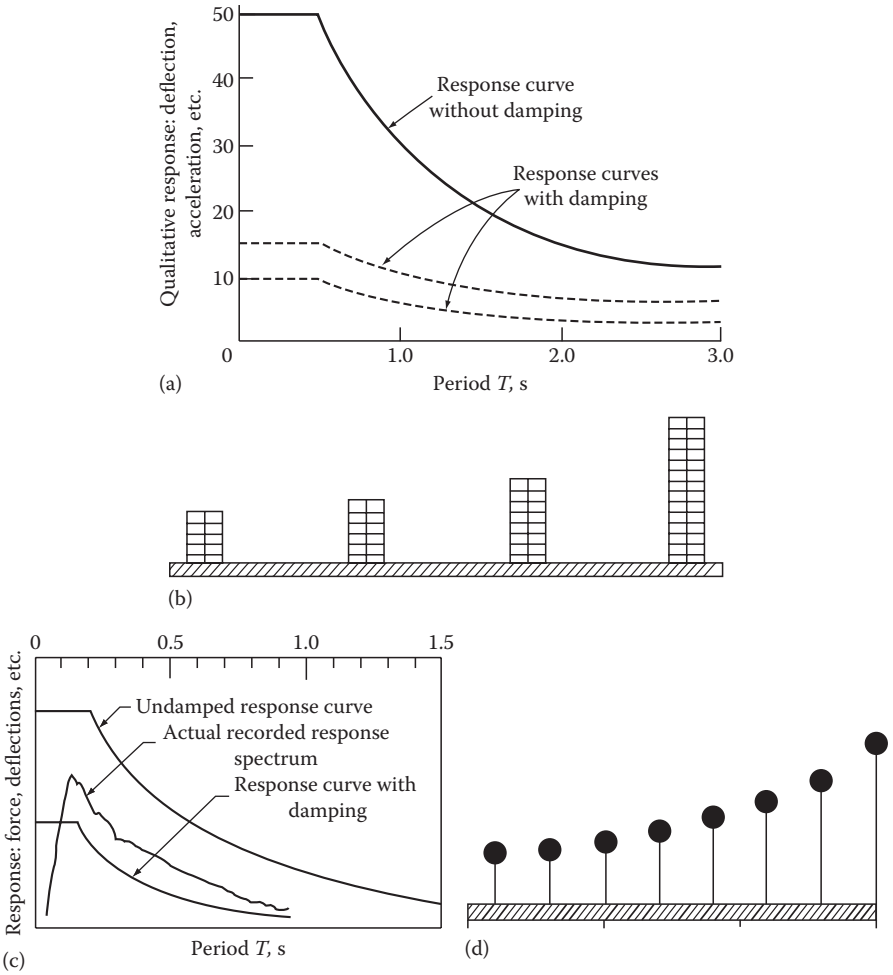


FIGURE 6.20 (a and c) Concept of response spectrum; (b, d) pendulums of varying heights representing progressively taller buildings.

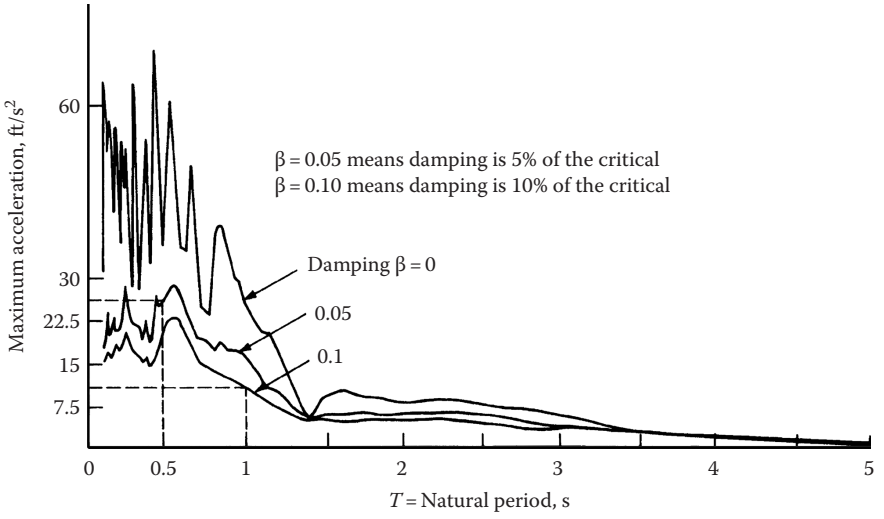


FIGURE 6.21 Acceleration response spectrum: El Centro earthquake.

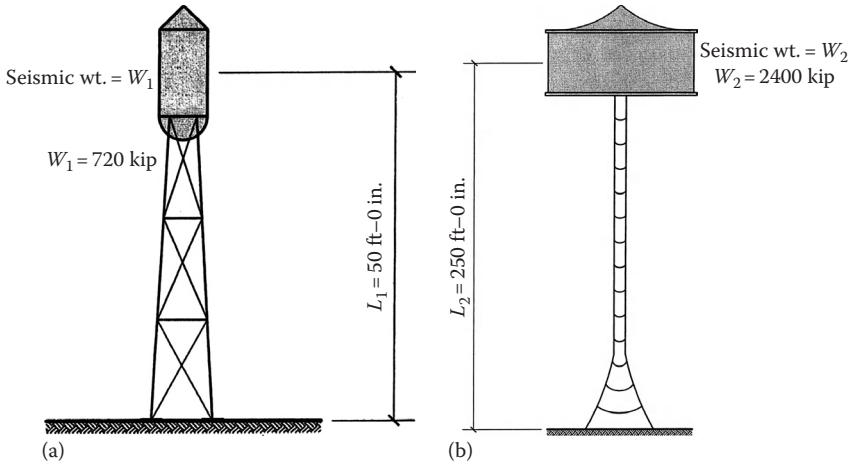


FIGURE 6.22 Examples of SDOF systems: (a) elevated water tank and (b) restaurant atop tall concrete core. Note from Figure 6.21, the acceleration = 26.25 ft/s^2 for T = 0.5 s and β = 0.05 (water tank) and the acceleration = 11.25 ft/s^2 for T = 1.00 s and β = 0.10 (restaurant).

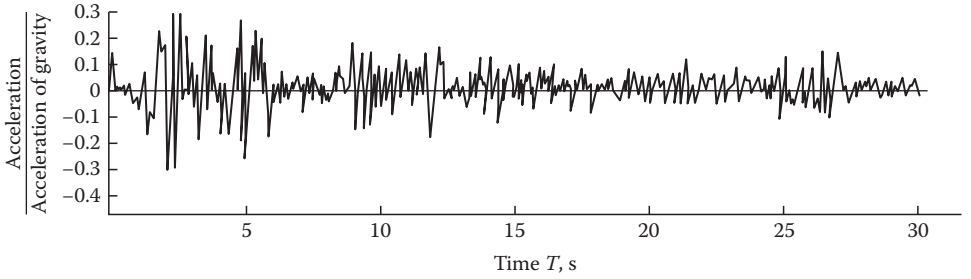


FIGURE 6.23 Recorded ground acceleration: El Centro earthquake.

As a first step, the base of the two structures is analytically subjected to the same acceleration as the El Centro recorded acceleration. The purpose is to calculate the maximum dynamic response experienced by the two masses during the first 30 s of the earthquake. The maximum response such as displacement, velocity, and acceleration response of an SDOF system such as the two examples considered here, may be obtained by considering the earthquake effects as a series of impulsive loads, and then integrating the effect of individual impulses over the duration of the earthquake. This procedure, known as Duhamel integration method, requires considerable analytical effort. However, it is not necessary for us to carry out this procedure when using a response spectrum, because maximum response for postulated earthquakes at a given site is already included in the spectrum. The spectral acceleration response for the north-south component of the El Centro earthquake, shown in Figure 6.23, is one such example.

To determine the seismic lateral loads, assume the tank and restaurant structures weight 720 and 2400 kip, with corresponding periods of vibration of 0.5 and 1 s, respectively. Since the response of a structure is strongly influenced by damping, it is necessary to estimate the damping factors for the two structures. Let us assume that the percentage of critical damping ξ for the tank and restaurant are 5% and 10% of the critical damping, respectively. From Figure 6.21, the acceleration for the tank structure is 26.25 ft/s^2, and the horizontal force in kips, would be equal to the mass at the top times the acceleration.

Thus, the horizontal force, F, for the water tank structure is equal to

$$F = \frac{720}{32.2} \times 26.25 = 587\,\text{kip}$$

Similarly, for the restaurant structure, the force at top is equal to

$$F = \frac{2400}{32.2} \times 11.25 = 838.51\,\text{kip}$$

The two structures can then be designed by applying the seismic loads at the top and determining the associated forces, moments, and deflections. The lateral load, evaluated by multiplying the response spectrum acceleration by the effective mass of the system, is referred to as base shear, and its evaluation forms one of the major tasks in earthquake analysis.

In the example, SDOF structures were chosen to illustrate the concept of spectrum analysis. A multistory building, however, is not typically modeled as a SDOF system because it will have as many modes of vibration as its degrees of freedom. For practical purposes, the distributed mass of a building is lumped at discrete levels to reduce the degrees of freedom to a manageable number. In multistory buildings, the masses are typically lumped at each floor level.

In the two-dimensional (2D) analysis of a building, the number of modes of vibration corresponds to the number of levels, with each mode having its own characteristic frequency. The actual motion of a building is a linear combination of its natural modes of vibration. During vibrations, the masses vibrate in phase with the displacements as measured from their initial positions, always having the same relationship to each other. Therefore, all masses participating in a given mode pass the equilibrium position at the same time and reach their extreme positions at the same instant.

Using certain simplifying assumptions, it can be shown that each mode of vibration behaves as an independent Single-Degree-of-Freedom, SDOF system with a characteristic frequency. This method, called the modal superposition method, consists of evaluating the total response of a building by statistically combining the response of a finite number of modes of vibration.

A building, in general, vibrates with as many mode shapes and corresponding periods as its degrees of freedom. Each mode contributes to the base shear, and for elastic analysis, this contribution can be determined by multiplying a percentage of the total mass, called effective mass, by an acceleration corresponding to that modal period. The acceleration is typically read from the response spectrum modified for damping associated with the structural system. Therefore, the procedure for determining the contribution of the base shear for each mode of a Multi-Degree-of-Freedom, MDOF structure is the same as that for determining the base shear for an SDOF structure, except that an effective mass is used instead of the total mass. The effective mass is a function of the lumped mass and deflection at each floor with the largest value for the fundamental mode, becoming progressively less for higher modes. The mode shape must therefore be known in order to compute the effective mass.

Because the actual deflected shape of a building consists of a linear combination of its modal shapes, higher modes of vibration also contribute, although to a lesser degree, to the structural responses. These can be taken into account through use of the concept of a participation factor. The base shear for each mode is determined as the summation of products of effective mass and spectral acceleration at each level. The force at each level for each mode is then determined by distributing the base shear in proportion to the product of the floor weight and displacement. The design values are then computed using modal combination methods, such as the complete quadratic combination (CQC) and the square root of sum of the squares (SRSS), the preferred method being the former.

6.1.7.1 Earthquake Response Spectrum

Response spectrum analysis is a means of using acceleration response spectra to determine the maximum forces and displacements in a structure that remains elastic when it responds to ground shaking. For SDOF structures, the maximum elastic structural displacement is given by

$$\Delta = \frac{T^2}{4\pi^2} S_a$$

where

T is the structure's period

S_a is the spectral acceleration obtained from the response spectrum plot at period T

The maximum force demand on the structure is given by

$$F = \frac{W}{g} S_a$$

For MDOF structures, the response of the structure can be determined by combining the response quantities for a series of SDOF structures having the same period and mass as each of the structure's modes. For mode i, the maximum inertial force produced in the structure by the earthquake, which is also termed the modal base shear, V_i, is given by

$$V_i = M_i S_a$$

where

M_i is the modal mass for mode i

S_a is the spectral acceleration obtained from the response spectrum at natural period T_i

The results of the analyses conducted for the various modes must be combined in order to obtain an estimate of the structure's actual behavior. Since it is unlikely that peak structural response in all modes will occur simultaneously, statistical combination rules are used to combine the modal results in a manner that more realistically assesses the probable combined effect of these modes. One such combination method takes the combined value as the square root of the SRSS of the peak response quantities in each mode.

When several modes have similar periods, the SRSS method does not adequately account for modal interaction. In this case, the CQC technique is used. Most structural analysis software used in design offices today provides the capability to perform these computations automatically.

For SDOF structures, response spectrum analysis gives exact results, as long as the response spectrum that is used to represent the loading accurately represents the ground motion. However, for MDOF structures, response spectrum analysis is always approximate because the way that the peak displacements and forces from the various modes are combined. It does not accurately represent the way these quantities will actually combine in a real structure subjected to ground shaking. Although the results of response spectrum analysis are approximate, it is universally accepted as a basis for earthquake-resistant design, when properly performed.

An acceleration response spectrum as stated previously is a plot of the maximum acceleration that SDOF structures of different periods, T, would experience when subjected to a specific earthquake ground motion. This plot is constructed by performing response history analyses for a series of structures, each having a different period, T, obtaining the maximum acceleration of each structure from the analysis, and plotting it as a function of T. Linear acceleration response spectra are most common, and are obtained by performing linear response history analysis.

Although the response spectra obtained from each earthquake record will be different, spectra obtained from earthquakes having similar magnitudes on sites with similar characteristics tend to have common characteristics. This has permitted the building codes to adopt standard response spectra that incorporate these characteristics, and envelop spectra anticipated at a building site during a design earthquake. The response spectra contained in the building code are called smoothed design spectra because the peaks and valleys that are common in the spectrum obtained from any single record are averaged out to form smooth functional forms.

Earthquake response spectrum gives engineers a practical means of characterizing ground motions and their effects on structures. Introduced in 1932, it is now a central concept in earthquake engineering that provides a convenient means to summarize the peak response of all possible linear SDOF systems to a particular ground motion. It also provides a practical approach to apply the knowledge of structural dynamics to the design of structures and development of lateral force requirements in building codes.

A plot of the peak value of response quantity as a function of the natural vibration period T_n of the system (or a related parameter such as circular frequency ω_n or cyclic frequency f_n) is called the response spectrum for that quantity. Each such plot is for SDOF systems having a fixed damping ratio ξ. Often times, several such plots for different values of ξ are included to cover the range of damping values encountered in actual structures. Whether the peak response is plotted against f_n or T_n is a matter of personal preference. In this text, we use the latter because engineers are more comfortable in using natural period rather than natural frequency—the period of vibration is a more familiar concept and one that is intuitively appealing. Although a variety of response spectra can be defined depending on the chosen response quantity, it is almost always the acceleration response spectrum, (a plot of pseudo-acceleration, against the period T_n for a fixed damping, ξ) most often used in the practice of earthquake engineering.

A similar plot of displacement u is referred to as the deformation spectrum, while that of velocity $\dot{u}$ is called a velocity spectrum.

It is worthwhile to note that only the deformation $u(t)$ is needed to compute internal forces. Obviously, then, the deformation spectrum provides all the information necessary to compute the peak values of deformation and internal forces. The pseudo-velocity and pseudo-acceleration response spectrum are important, however, because they are useful in relating structural dynamics results to building codes.

6.1.7.2 Deformation Response Spectrum

To explain the procedure for determining the deformation response spectrum, we start with the spectrum developed for El Centro ground motion, which has been studied extensively in textbooks.

The acceleration recorded for the first 30 seconds of ground motion is shown in Figure 6.24a. The corresponding deformation in three SDOF systems of varying periods is presented in Figure 6.24b.

The peak deformations are

$u_o = 2.67$ in. for a system with natural period $T_n = 0.5$ s and damping ratio $\xi = 2\%$
$u_o = 5.97$ in. for a system with $T_n = 1$ s and $\xi = 2\%$
$u_o = 7.47$ in. for a system with $T_n = 2$ s and $\xi = 2\%$

The u_o value so determined for each system provides one point on the deformation response spectrum. Repeating such computations for a range of values of T_n while keeping ξ constant at 2% provides the deformation response spectrum shown in Figure 6.25a. The spectrum shown is for a single damping value, $\xi = 2\%$. However, a complete response spectrum would include such spectrum curves for several values of damping.

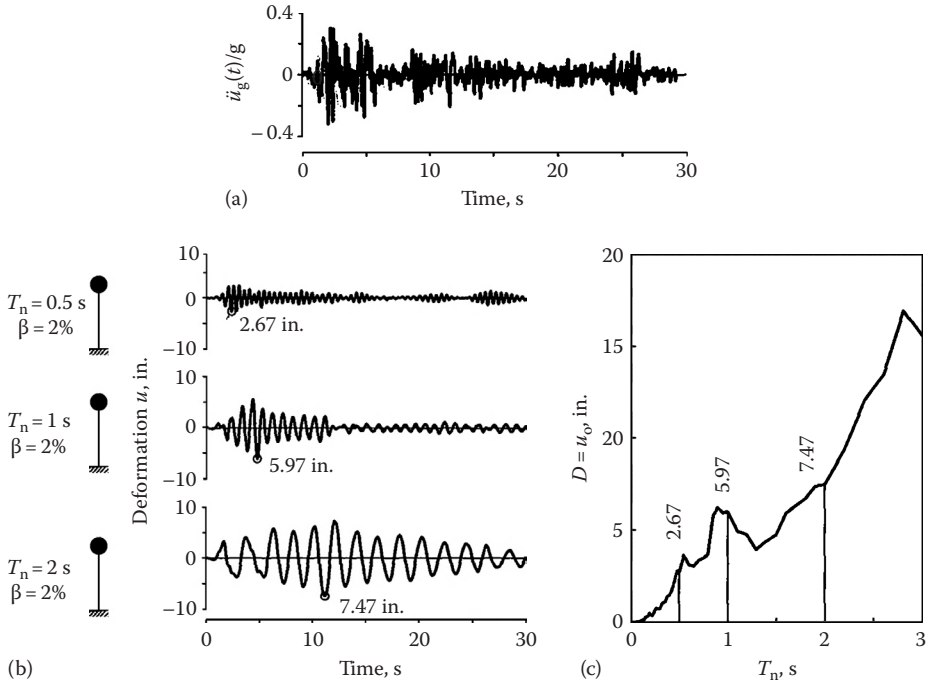


FIGURE 6.24 (a) Ground acceleration; (b) deformation response of three SDOF systems with β = 2% and T_n = 0.5, 1, and 2 s, and (c) deformation response spectrum for β = 2%.

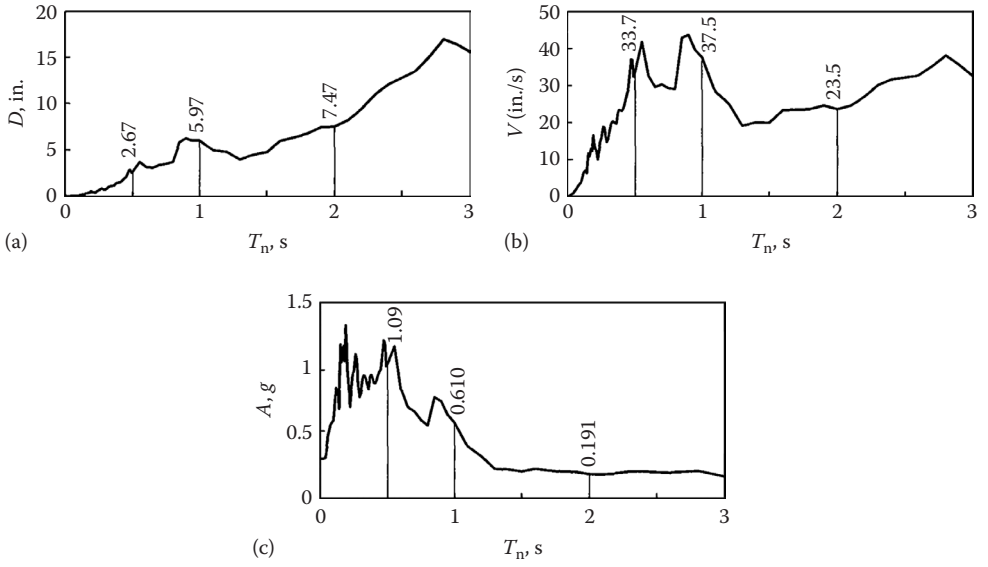


FIGURE 6.25 Response spectra (β = 2%) for El Centro ground motion: (a) deformation response spectrum, (b) pseudo-velocity response spectrum, and (c) pseudo-acceleration.

6.1.7.3 Pseudo-Velocity Response Spectrum

The pseudo-velocity response spectrum is a plot of velocity V as a function of the natural vibration period T_n, or natural vibration frequency f_n, of the system. For a given ground motion, the peak pseudo-velocity V for a system with natural period T_n can be determined from the following equation using the deformation $\xi = 2\%$ of the same system from the response spectrum of Figure 6.25b:

$$V = \omega_a D = \frac{2\pi}{T_n} D$$

As an example, for a system with $T_n = 0.5$ s and $\xi = 2\%$, $D = 2.67$ in.:

$$V = \omega_a D = \frac{2\pi}{T_n} D = \left(\frac{2\pi}{0.5}\right) 2.67 = 33.7 \text{ in./s}$$

Similarly, for $T_n = 1.0$ s and $\xi = 2\%$, $D = 5.97$ in.:

$$V = \left(\frac{2\pi}{1}\right) 5.97 = 37.5 \text{ in./s}$$

And, for $T_n = 2.0$ s and the same damping $\xi = 2\%$, $D = 7.47$ in.:

$$V = \left(\frac{2\pi}{2}\right) 7.47 = 23.5 \text{ in./s}$$

These three values of peak pseudo-velocity V are identified in Figure 6.25b. Repeating such computations for a range of values of T_n while keeping ξ constant at 2% provides the pseudo-velocity spectrum shown in Figure 6.25b. The prefix "pseudo" is used for V because V it is not equal to the peak velocity, although it has the same units for velocity.

6.1.7.4 Pseudo-Acceleration Response Spectrum

It has been stated many times in this chapter that the base shear is equal to the inertia force associated with the mass m undergoing acceleration A. This acceleration A is generally different from the peak acceleration of the system. It is for this reason that A is called the peak pseudo-acceleration; the prefix "pseudo" is used to avoid possible confusion with the true peak acceleration, just as we did for velocity V. The pseudo-acceleration response spectrum is a plot of acceleration A as a function of the natural vibration period T_n, or natural vibration frequency f_n, of the system. For a given ground motion, peak pseudo-acceleration A for a system with natural period T_n and damping ratio ξ can be determined from the following equation using the peak deformation D of the system from the response spectrum:

$$A = \omega_n^2 D = \left(\frac{2\pi}{T_n}\right)^2 D$$

As an example, for a system with $T_n = 0.5$ s and $\xi = 2\%$, $D = 2.67$ in.:

$$A = \omega_n^2 D = \left(\frac{2\pi}{T_n}\right)^2 D = \left(\frac{2\pi}{0.5}\right)^2 2.67 = 1.09g$$

where g is the acceleration due to gravity = 386 in./s^2

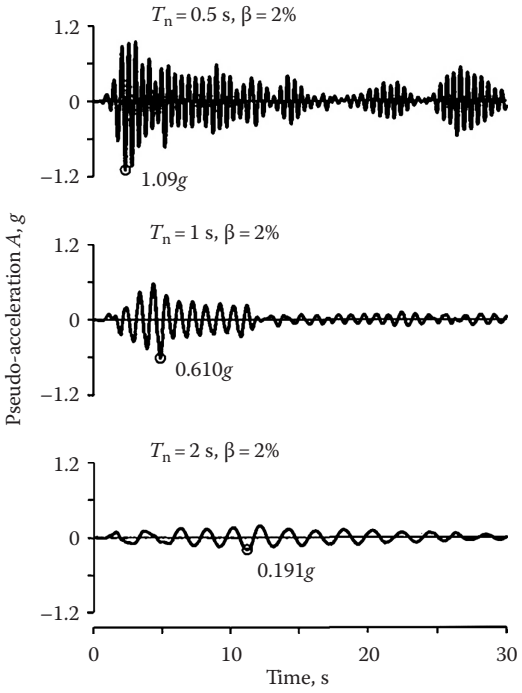


FIGURE 6.26 Pseudo-acceleration response of SDOF systems to El Centro ground motion.

Similarly, for a system with $T_n = 1$ s and $\xi = 2\%$, $D = 5.97$ in.:

$$A = \left(\frac{2\pi}{T_n}\right)^2 D = \left(\frac{2\pi}{1}\right)^2 5.97 = 0.610g$$

And for a system with $T_n = 2$ s and the same damping $\xi = 2\%$, $D = 7.47$ in.

$$A = \left(\frac{2\pi}{2}\right)^2 7.47 = 0.191g$$

The three values T_n of pseudo-acceleration, A, are shown in Figure 6.26. Repeating similar computations for a range of T_n values, while keeping β constant at 2%, yields the pseudo-acceleration spectrum shown in Figure 6.25c.

6.1.7.5 Tripartite Response Spectrum: Combined Displacement–Velocity–Acceleration Spectrum

It was shown in the previous section that each of the deformation, pseudo-velocity, and pseudo-acceleration response spectra for a given ground motion contain the same information, not more and not less. The three spectra are simply distinct ways of displaying the same information on structural response. With the knowledge of one of the spectra, the other two can be derived by algebraic operations using the procedure given in the previous section.

If each of the spectra contains the same information, why do we need three spectra? A good question that has two valid reasons. One is that each spectrum directly provides a physically meaningful quantity: the deformation spectrum provides the peak deformation of a system, the

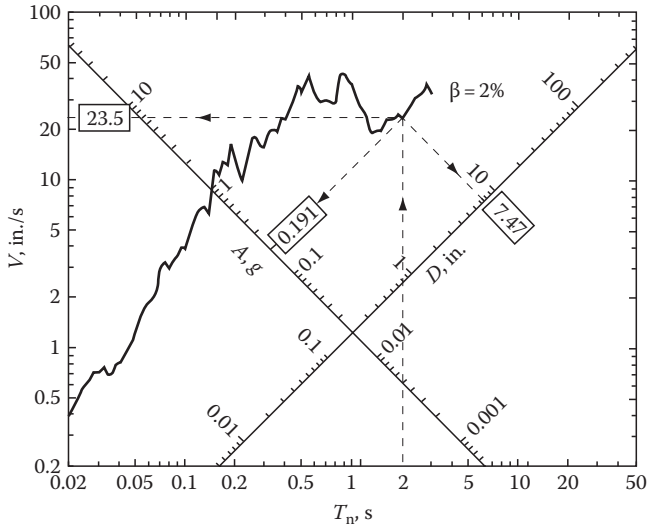


FIGURE 6.27 Combined *DVA* response for El Centro ground motion; β = 2%.

pseudo-velocity spectrum gives the peak strain energy stored in the system during the earthquake, and pseudo-acceleration spectrum yields directly the peak value of the equivalent static force. The second reason lies in the fact that the shape of the spectrum can be approximated more readily for design purposes with the aid of all three spectral quantities rather than any one of them alone. For this purpose, a combined plot showing all three of the spectral quantities is especially useful. This type of plot was developed for earthquake response spectra for the first time by A.S. Veletsos and N.M. Newmark in 1960.

In an integrated DVA spectrum, the vertical and horizontal scales for V and T_n are standard logarithmic scales. The two scales for D and A sloping at +45° and −45°, respectively to the T_n-axis, are also logarithmic scales but not identical to the vertical scale. The pairs of numerical data for V and T_n that were plotted in Figure 6.27 on linear scales are replotted in Figure 6.27 on logarithmic scales. For a given natural period T_n, the D and A values can be read from the diagonal scales. As an example, for $T_n = 2$ s in Figure 6.27, gives $D = 7.47$ in. and $A = 0.191g$. The four-way plot is a compact presentation of the three—deformation, pseudo-velocity, and pseudo-acceleration—response spectra, for a single plot of this form replaces the three plots.

The benefit of the response spectrum in earthquake engineering may be recognized by the fact that spectra for virtually all ground motions strong enough to be of engineering interest are now computed and published soon after they are recorded. From these we can get a reasonable idea of the kind of motion that is likely to occur in future earthquakes. It should be noted that for a given ground motion response spectrum, the peak value of deformation, pseudo-velocity, and base shear in any linear SDOF can be readily read from the spectra without resorting to dynamic analyses. This is because the computationally intense dynamic analysis has been completed in generating the response spectrum.

Given these advantages it makes good sense to have geotechnical engineers provide tripartite response spectrum rather than just an acceleration spectrum, when site-specific studies are commissioned.

An example of tripartite response spectrum is shown in Figure 6.28. The spectrum is for maximum capable earthquake (MCE) of magnitude 8.5 occurring at San Andreas Fault. The project site is in downtown Los Angeles located at a distance of 34 miles from San Andreas Fault.

The response spectrum tells us that the forces experienced by buildings during an earthquake are not just a function of the quake, but are also their dynamic response characteristics to the quake. The response primarily depends on the period of the building being studied. A great deal of

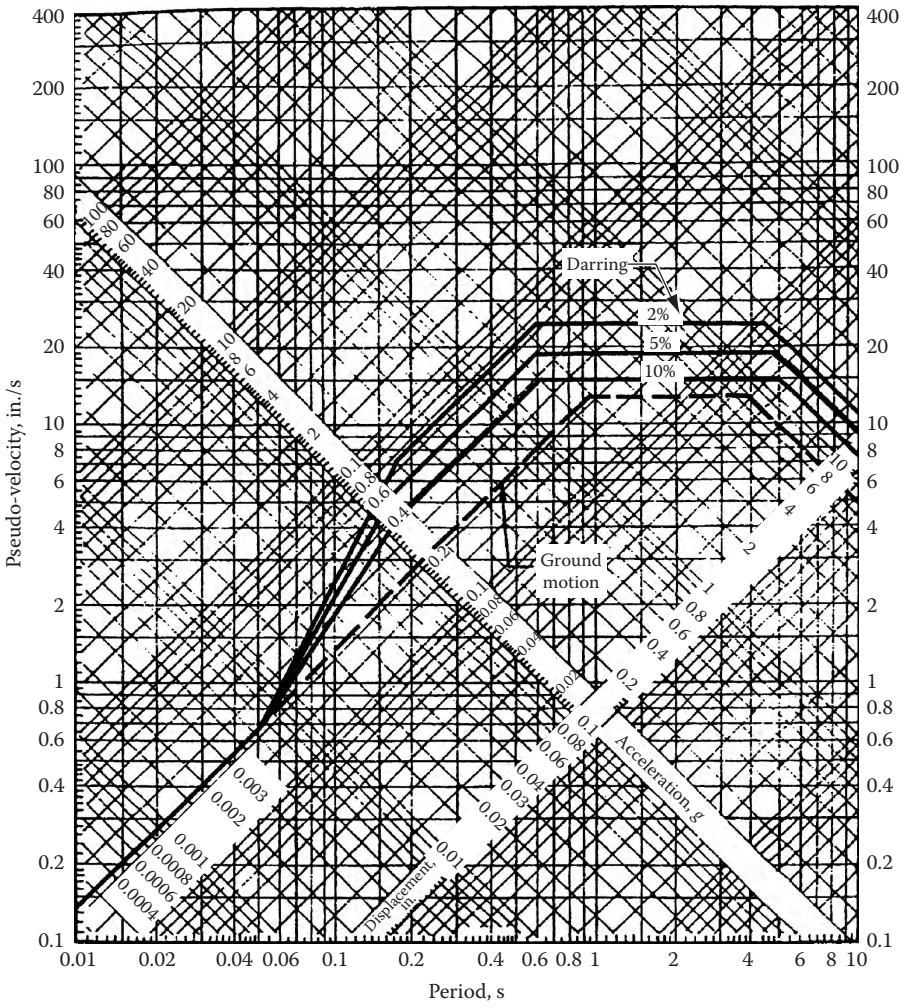


FIGURE 6.28 Tripartite response spectrum.

single-mode information can be read directly from the response spectrum. Referring to Figure 6.27, the horizontal axis of the response spectrum expresses the period of the building being affected by the quake. The vertical axis shows the velocity attained by this building during the quake. The diagonal axis running up toward the left-hand corner reads the maximum accelerations to which the building is subjected. The axis at right angles to this will read the displacement of the building in relation to the support. Superimposed on these tripartite scales are the response curves for an assumed 5% damping.

Let us examine yet another tripartite response spectrum shown in Figure 6.29 to study how buildings of different periods respond to a given spectrum.

If the building is to be studied had a natural period of 1 s, we would start at the bottom of the chart at $T = 1$ s, and reference vertically until we intersect the response curve. From this intersection, point A, we travel to the extreme right and read a velocity of 16 in./s. Following a displacement line diagonally down to the right, we find a displacement of 2.5 in. Similarly following the acceleration line down to the left, we see that the building will experience an acceleration of 0.25 g. If we then move to the 2 s period, point B, in the same sequence, we find that we will have the same velocity of 16 in./s, a displacement of 4 in., and a maximum acceleration of 0.10g. If we then move to 4 s, point C,

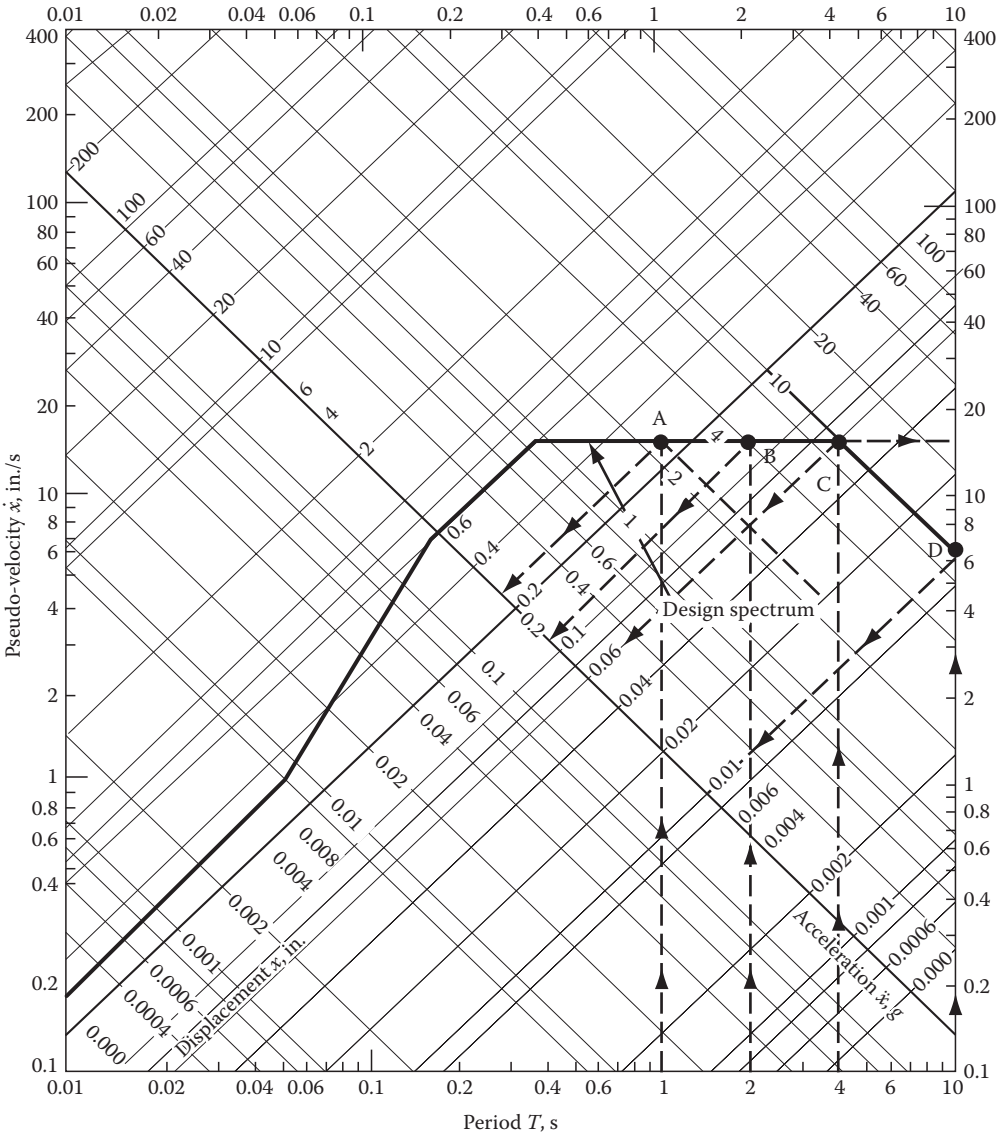


FIGURE 6.29 Velocity, displacement, and acceleration read out from response spectra.

we see a velocity of 16 in./s, a displacement of 10 in., and an acceleration of 0.06 *g*. If we run all out to 10 s, point D, we find a velocity of 7 in./s, a displacement of 10 in. the same as for point C, and an acceleration of 0.01*g*. Notice that the characteristic parameters vary widely, as stated earlier, depending on the period of building.

6.1.7.6 Characteristics of Response Spectrum

We now study certain important properties of earthquake response spectra. For this purpose, we use once again an idealized response spectrum for El Centro ground motion shown in Figure 6.30. The damping, ξ, associated with the spectrum is 5%. The period T_n plotted on a logarithmic scale covers a wide range, $T_n = 0.01$ to 10 s.

Consider a system with a very short period, say 0.03 s. For this system, the pseudo-acceleration *A* approaches the ground acceleration while the displacement *D* is very small. There is a physical reasoning for this trend. For purposes of dynamic analysis, a very short period system is extremely

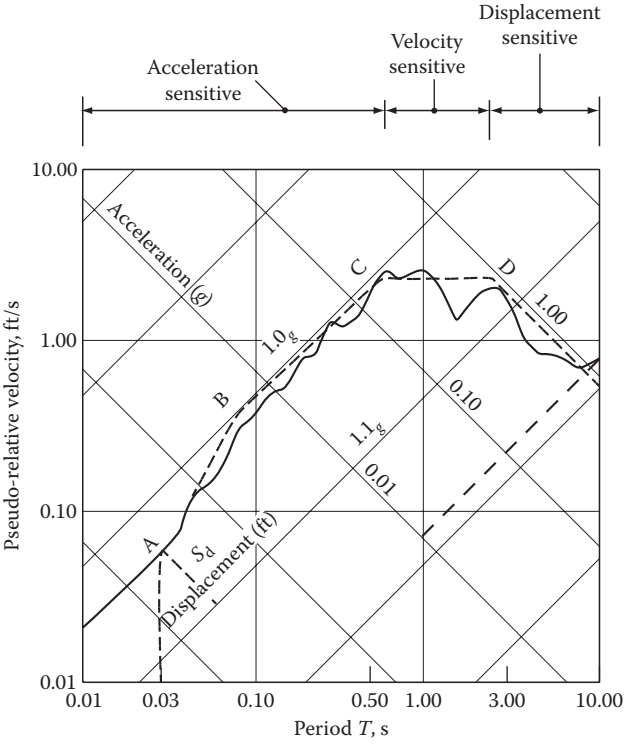


FIGURE 6.30 Idealized response spectrum for El Centro ground motion.

stiff and may be considered essentially rigid. Such a system would move rigidly with the ground as if it is a part of the ground itself. Thus, its peak acceleration would be approximately equal to the ground acceleration as shown in Figure 6.30.

Next, we examine a system with a very long period, say $T_n = 10$ s. The acceleration A, and thus the force in the structure, which is related to mA, would be small. Again there is a physical reasoning for this trend: A very long period system is extremely flexible. Therefore, the mass at top would remain stationary while the base moves with the ground below (see Figure 6.30).

Based on these two observations, and those in between the two periods (not examined here), it is logical to divide the spectrum into three period ranges: (1) The long-period region to the right of point D, called the displacement-sensitive region because structural response is most directly to ground displacement; (2) the short-period region to the left of point C, called the acceleration-sensitive region because structural response is most directly related to ground acceleration; (3) the intermediate period region between points C and D, called the velocity-sensitive region because structural response appears to be better related to ground velocity than to other ground motion parameters.

The preceding discussion should be helpful in recognizing the usefulness of four-way logarithmic plot of the combined deformation, pseudo-velocity and pseudo-acceleration response spectra. These observations would be difficult to discover from the three individual spectra.

We now turn our attention to damping, which has significant influence on the earthquake response of buildings. It reduces the response of a structure, as expected. However, the reduction achieved with a given amount of damping is different in the three spectral regions. In the limit as $T_n \to \infty$, damping does not affect the response because the structural mass stays still while the ground underneath moves. Among the three period regions, the effect of damping tends to be greatest in the velocity-sensitive region of the spectrum. In this spectral region, the effect of damping

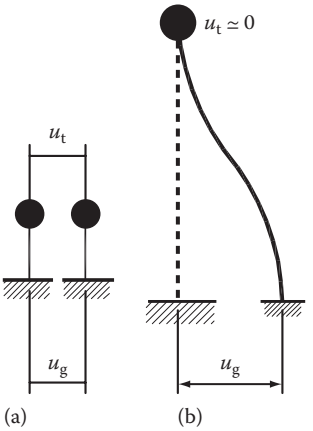


FIGURE 6.31 Schematic response of rigid and flexible systems: (a) Rigid system, acceleration at top is nearly equal to the ground acceleration; (b) flexible system, structural response is most directly related to ground displacement.

depends on the ground motion characteristics. If the ground motion is harmonic over many cycles as it was in the Mexico City earthquake of 1985, the effect of damping would be especially large for systems near resonance.

The motion of structure and the associated forces could be reduced by increasing the effective damping of the structure. The addition of dampers achieves this goal without significantly changing the natural vibration period of the structure. Viscoelastic dampers have been used in many structures; for example, 10,000 dampers were installed throughout the height of each tower of the now nonexistent World Trade Center in New York City, to reduce wind-induced motion to within a comfortable range for the occupants. In recent years, there is a growing interest in developing dampers suitable for structures in earthquake-prone regions. Because the inherent damping in most structures is small, their earthquake response can be reduced significantly by the addition of dampers. These can be especially useful in improving the seismic safety of existing structures.

As stated previously, the response of a SDF system at the extremes of the fundamental periods is intuitively obvious. For example, a system with very a short period, that is, a very stiff system, experiences peak acceleration approaching that of the ground. The distortion of the system, however, is negligible since the motion of the mass is the same as that of the ground. On the other hand, for systems with a very long period, that is, flexible buildings, the mass remains stationary while the ground moves beneath it. The relative displacement between the two is equal to the ground displacement. However, the acceleration of the mass is zero or nearly so. Thus, the force in the structure related to *ma* would also be small. This concept is illustrated in Figure 6.31.

6.1.7.7 Difference between Design and Actual Response Spectra

The response spectrum for a given earthquake is typically a jagged plot representing the peak response of SDF systems. It is a unique plot representative of the particular ground motion. The design spectrum is, however, a smooth plot specifying the level of seismic force as a function of the system's period and damping ratio. This is the first difference. The second difference is the design spectrum is an envelope of two or more different elastic spectra judged to be of consequence in affecting ground motions at a given site. After determining the design spectrum for each of the postulated earthquakes, the design spectrum for the site is developed by enveloping the design spectra for all earthquakes considered for the site, as shown schematically in Figure 6.32a.

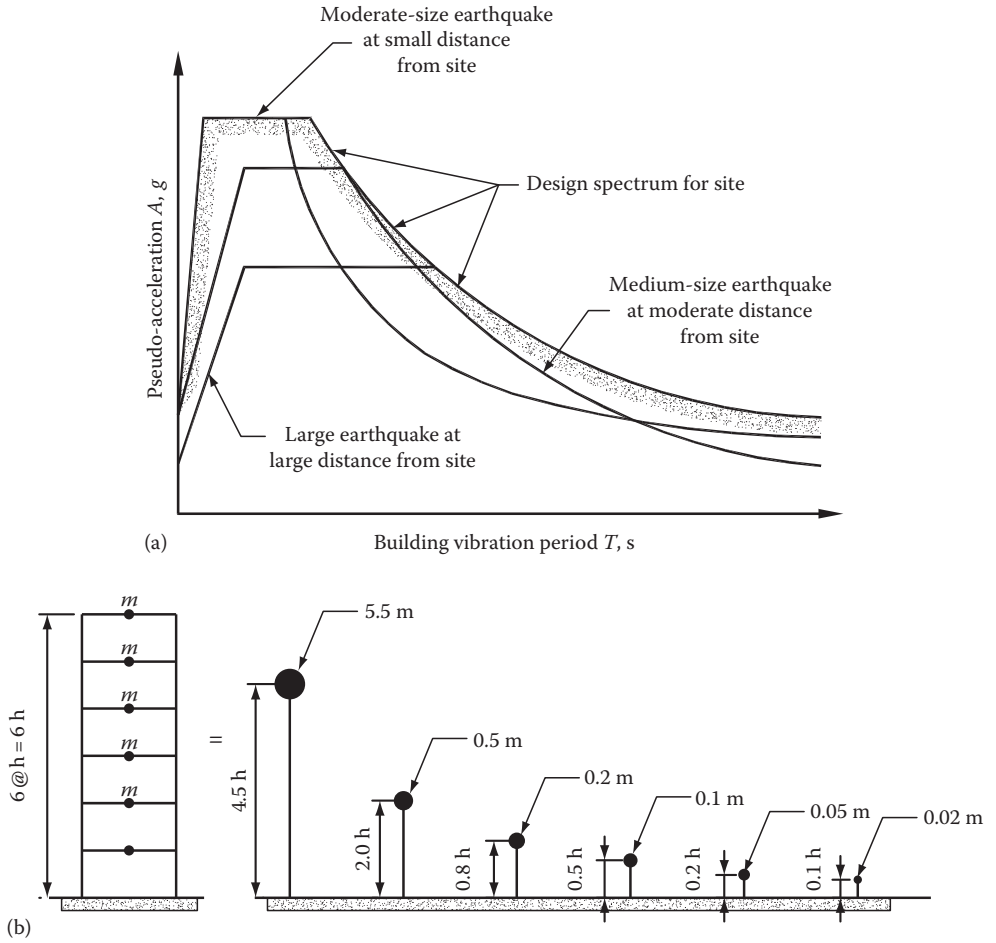


FIGURE 6.32 (a) Comparison of design and actual response spectra. (b) Concept of effective modal masses and effective modal heights.

6.1.7.8 Summary of Response Spectrum Analysis

This is a well-devised method of determining peak response of a system directly from the response spectrum for the ground motion without having to carry a time history analysis. For single-degree-of-freedom systems the method is quite accurate. For multi-degree-of-freedom systems, in a manner of speaking, the result is not exact. However, the result is accepted in practice, as being accurate enough for seismic design applications.

In general, modal responses attain their peaks at different time instants. Therefore, approximations must be used in combining the peak responses determined from response spectrum analysis because no information is available as to when these peak modal values occur. Summing up the absolute values of the maximum values would certainly give us an upper bound solution but is not used in building design practice because the results tend to be too conservative.

In practice, two other modal combination rules, the SRSS and CQC, both of which are based on random vibration theory, are popular. The latter is the preferred method applicable to a wider class of structures as it is also applicable to structures with not-so-well-separated periods.

The response spectrum analysis is a procedure for dynamic analysis of a structure subjected to earthquake ground motions. In a conceptual sense, it reduces dynamic analysis to a series of static analysis. For each mode considered, static analysis of the structure subjected to corresponding modal shears provides the peak modal static response. This procedure avoids the dynamic analysis of SDF

systems necessary for time-history analysis. Thus, the response spectrum analysis may be considered as a pseudo-dynamic analysis procedure, because it indirectly uses the vibration properties of the structure and the dynamic characteristics of the ground motion. The intensive time-history calculations are not required because in developing a smooth design spectrum, somebody has already done these by converting the dynamic loads into equivalent static loads. Conceptually equivalent loads may be thought of as static loads multiplied by their corresponding deflection loud factors.

Response spectrum analysis is a procedure to compute the peak response of a structure during an earthquake directly from the earthquake design spectrum without the need for response history analysis of the structure. The procedure is not an exact predictor of peak response, but it provides an estimate that is considered sufficiently accurate for structural design applications.

To assist us in developing an understanding of the response behavior of multistory buildings a conceptual explanation of modal analysis is given in Figure 6.32b.

It should be evident by now that the effective modal masses and effective modal heights for a given building depend on the characteristics of the response spectrum and the dynamic properties of the building itself. Therefore, the values shown for a six-story example building in Figure 6.32b are for conceptual purposes only and are not based on any given design response spectrum or building properties.

6.1.8 Hysteresis Loop

The term "hysteresis" is often used in engineering mechanics to represent the energy dissipation due to material plasticity. It is derived from an ancient Greek word meaning "lagging behind" or "deficiency."

A simple way to understand this is in terms of a steel bar with weights attached to it. If the bar is hung from a support and the weights are attached to the bar one at a time, it is quite evident that the bar will stretch and continue to do so as the weight is progressively increased.

As the weights are taken off (and here is the crucial point in understanding hysteresis behavior), each weight that produced a specific increase in length as it was loaded will now result in a slightly longer decrease in length as the bar is unloaded.

This is because the bar is inelastic and hence does not obey Hook's law perfectly. In a conceptual sense, the steel bar working in the inelastic range, is harder to stretch when being loaded than when it is being unloaded. In other words, more energy is required during the loading than what is recovered during the unloading. That difference in energy must go somewhere. In our case, it is "lost" or "dissipated" as heat. In structural engineering, the energy dissipation is typically characterized as the area of the hysteresis loop between the loading and unloading cycles.

In a major earthquake, a structure will not remain *elastic*, but will oscilate in the *inelastic* range. The back and forth inelastic motion absorbs significant energy and therefore, the structure can be designed for considerably smaller lateral forces.

Observe that such excursions beyond the elastic range are not permitted in normal operating conditions such as under gravity and wind loads. However, the behavior of structures subject to extreme load conditions such as blast or severe earthquakes are quite important in structural design.

Let us consider an idealized two-dimensional steel frame shown in Figure 6.33 subjected to a lateral load P. If the flexural rigidities of the columns are less than that of the beam and the load is increased progressively, at some point in the loading history, so-called *plastic hinges* will form at the ends of the column. A plot of load P against the displacement x shown in Figure 6.34, shows that x is linear up to the value of P_{y1} (see the line labeled (1) in Figure 6.34) where yielding of the material begins. Subsequently, it curves (see the curve labeled (2)) due to softening of columns at the base and top. Upon unloading, the material rebounds elastically, as indicated by part (3) of the plot in Figure 6.34. The unloading portion is represented by another straight line parallel to the elastic-loading line (part (1) of the plot). If a reverse loading is then applied, the parts of Figure 6.35 labeled

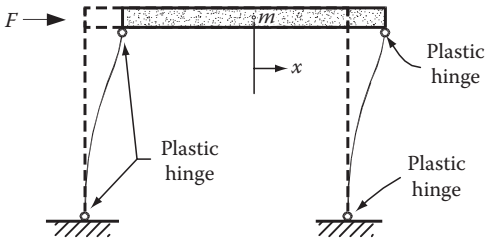


FIGURE 6.33 Idealized steel portal frame subject to load P at top.

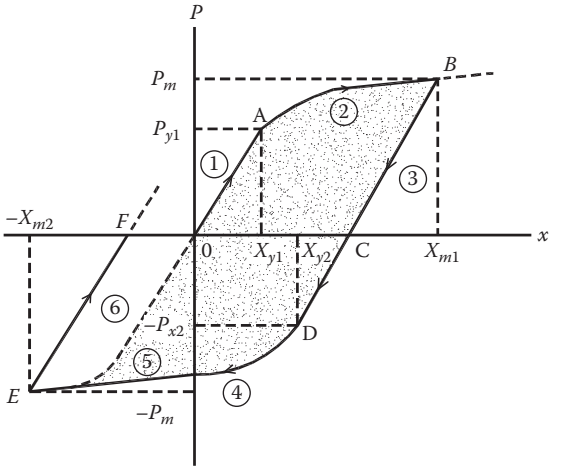


FIGURE 6.34 Displacement plot of load P versus displacement x.

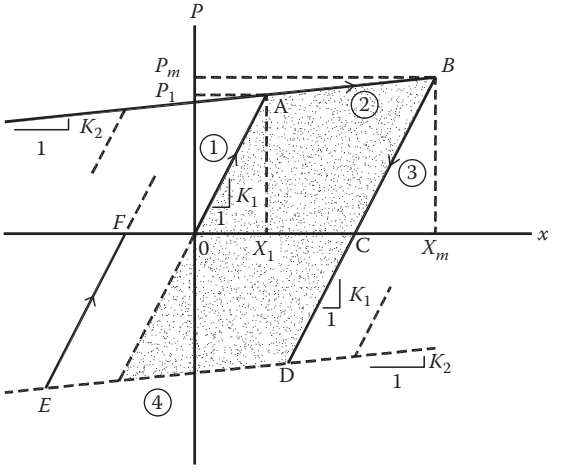


FIGURE 6.35 Plot of idealized displacement x versus load P: observe the curved portion of Figure 6.34 is replaced by straight lines depicting bilinear inelastic behavior.

(4) and (5) will result; and a subsequent unloading produces line (6). If the maximum positive and negative forces P_m and $-P_m$ (the ordinates of points B and E on the diagram) are numerically equal, the *hysteresis loop* formed by cyclic loading is symmetric with respect to the origin.

The curved portions of Figure 6.34 are often replaced by straight lines approximating the true behavior. Figure 6.35 illustrates such a simplified load-displacement diagram, called a *bilinear*

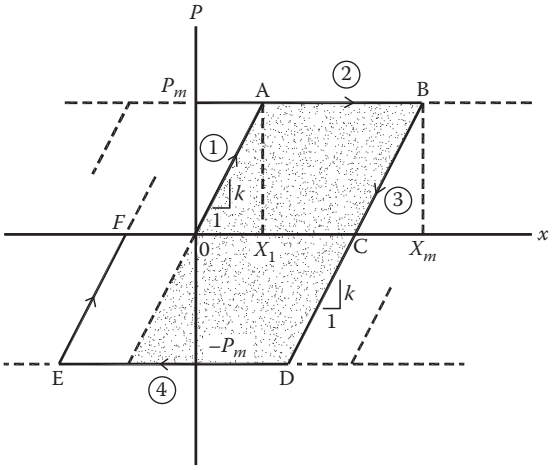


FIGURE 6.36 Elastoplastic hysteresis loop.

hysteresis loop. It consists of two parallel lines (labeled (2) and (4) in Figure 6.35) for inelastic behavior and a family of parallel lines (of which those labeled (1) and (3) are representative) for elastic behavior. If the slopes of line (2) and (4) are zero, as in Figure 6.36, the diagram represents an *elastoplastic hysteresis loop*. That is, the plot of P against x consists of straight-line segments, where the behavior is assumed to be either perfectly elastic or perfectly plastic. For example, let us reconsider the frame in Figure 6.33 and suppose that the load P increases to the value P_m (see point A if Figure 6.33). If the plastic hinges in Figure 6.33 are assumed to form instantaneously, the displacement increases without a corresponding increase of the load, as shown by the horizontal line from A to B in Figure 6.36. A decrease in the load causes a decrease of displacement in accordance with line (3) in Figure 6.37 and so on.

The hysteresis loop inherent in elastoplastic analyses represents a discretized form of structural damping. In this case, all of the dissipated energy is tacitly assumed to be absorbed at the plastic hinges. This dissipative mechanism is often referred to as *elastoplastic damping*, which represents a particular case of *hysteretic damping*.

It is interesting to note that if the slopes of the inclined lines (1) and (2) are taken to be infinite (meaning that the deformation of the portal in its elastic range is assumed to be small in comparison to the plastic deformation), the hysteresis loops specializes to a rigid-plastic dissipative mechanism as shown in Figure 6.37.

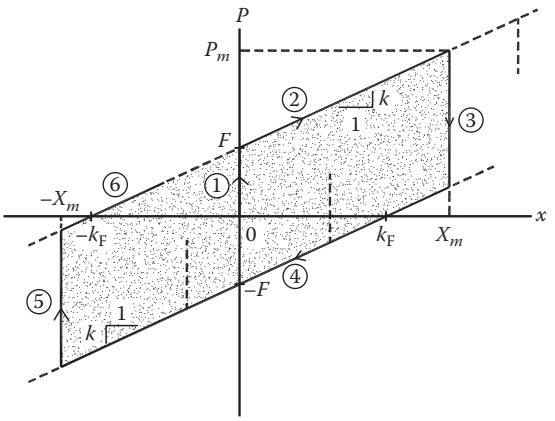


FIGURE 6.37 Rigid-plastic hysteresis loop.

The importance of understanding the use of hysteresis loops in seismic design may be explained as follows:

Inelastic response occurs when the amplitude of earthquake shaking is strong enough to cause forces in a structure to exceed the strength of any of the structure's elements or connections. When this occurs, the structure may experience a variety of behaviors. If the elements that are strained beyond their elastic strength limit are brittle, they will tend to break and lose the ability to resist any further load. This type of behavior is typified by a steel tension member that is stretched such that the force in the brace exceeds the ultimate strength of its end connections or by an unreinforced concrete element that is strained beyond its cracking strength. If the element is ductile, it may exhibit plastic behavior, being able to maintain its yield strength as it is strained beyond its elastic limit. This type of behavior is typified by properly braced, compact section beams in moment frames; by the cores of buckling-restrained braces; and by the shear links in eccentrically braced frames. Even elements that are ductile and capable of exhibiting significant post-yielding deformation without failure will eventually break and lose load-carrying capacity due to low-cycle fatigue if plastically strained over a number of cycles.

Modern structural analysis softwares provide the capability to analyze structures at deformation levels that exceed their elastic limit. In order to do this, these programs require input on the hysteretic (nonlinear force versus deformation) properties of the deforming elements. Hence, the knowledge of how these loops are generated is of importance in seismic design.

6.2 SEISMIC DESIGN CONSIDERATIONS

Although structural design for seismic loading is primarily concerned with structural safety during major earthquakes, serviceability and the potential for economic loss are also of concern. As such, seismic design requires an understanding of the structural behavior under large inelastic, cyclic deformations. Behavior under this loading is fundamentally different from wind or gravity loading. It requires a more detailed analysis, and application of a number of stringent detailing requirements to assure acceptable seismic performance beyond the elastic range.

The seismic analysis and design of buildings has traditionally focused on reducing the risk of loss of life in the largest expected earthquake. Building codes base their provisions on the historic performance of buildings and their deficiencies and have developed provisions around life safety concerns, by focusing their attention to prevent collapse under the most intense earthquake expected at a site during the life of a structure. These provisions are based on the concept that the successful performance of buildings in areas of high seismicity depends on a combination of strength, ductility manifested in the details of construction, and the presence of a fully interconnected, balanced and complete lateral-force-resisting system. In regions of low seismicity, the need for ductility reduces substantially. And in fact, strength may even substitute for a lack of ductility. Very brittle lateral-force-resisting systems can be excellent performers as long as they are never pushed beyond their elastic strength.

Seismic provisions specify criteria for the design and construction of new structures subjected to earthquake ground motions with three goals:

1. Minimize the hazard to life.
2. Increase the expected performance of structures having a substantial public hazard due to occupancy or use.
3. Improve the capability of essential facilities to function after an earthquake.

Some structural damage can be expected as a result of design ground motion because the codes allow inelastic energy dissipation in the structural system. For ground motions in excess of the design levels, the intent for the codes is for structures to have a low likelihood of collapse.

In most structures that are subjected to moderate-to-strong earthquakes, economical earthquake resistance is achieved by allowing yielding to take place in some structural members at

predetermined locations. It is generally impractical to design a structure to respond in the elastic range when subjected to the maximum expected earthquake ground motions. Therefore, in seismic design, yielding is permitted in predetermined structural members or locations, with the provision that the vertical load–carrying capacity of the structure is maintained even after strong earthquakes. However, for certain types of structures such as nuclear facilities, yield cannot be tolerated and as such, the design needs to be elastic.

Structures that contain facilities critical to post-earthquake operations—such as hospitals, fire stations, power plants, and communication centers—must not only survive without collapse, but must also remain operational during and after an earthquake. Therefore, in addition to life safety, damage control is an important design consideration for structures deemed vital to post-earthquake functions.

An idea of the behavior of a building during an earthquake may be grasped by considering the simplified response shown in Figure 6.38. As the ground on which the building rests is displaced, the base of the building moves with it. However, the building above the base is reluctant to move with it because the inertia of the building mass resists motion and causes the building to distort. This distortion wave travels along the height of the structure, and with continued shaking of the base, causes the building to undergo a complex series of oscillations.

Although both wind and seismic forces are essentially dynamic, there is a fundamental difference in the manner in which they are induced in a structure. Wind loads, applied as external loads,

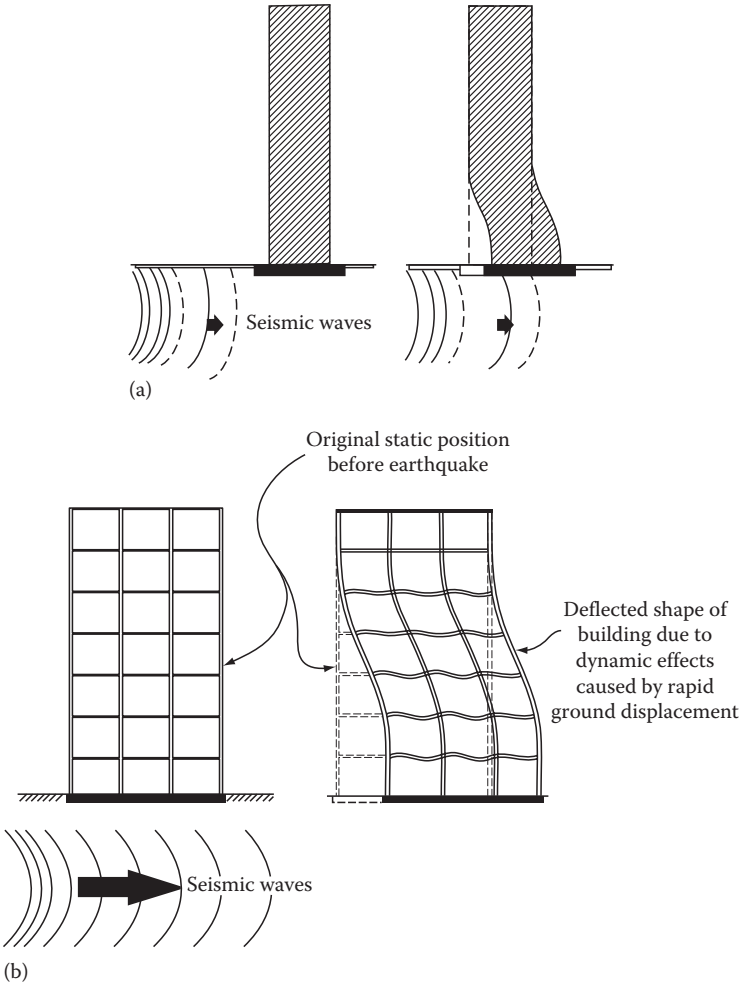


FIGURE 6.38 (a and b) Building behavior during earthquakes.

are characteristically proportional to the exposed surface of a structure, while the earthquake forces are principally internal forces resulting from the distortion produced by the inertial resistance of the structure to earthquake motions. Whereas in wind design, one would feel greater assurance about the safety of a structure made up of heavy sections, n seismic design, this does not necessarily produce a safer design.

6.2.1 Seismic Response of Buildings

Earthquakes can cause structural collapse in several different ways. First, if the structure is not adequately connected and "tied together," the motions induced in the structure by earthquake shaking can allow the building components to pull apart and, if one member is supported by another, to eventual collapse. This type of collapse is observed in bridges and other long structures that incorporate expansion joints.

Another way that earthquakes can cause structures to collapse is by overstressing gravity load-bearing elements such that they lose their load-carrying capacity. As an example, if the overturning loads on the columns in a braced-frame exceed the buckling capacity of the columns, these columns could buckle and lose their ability to continue to support the structure above.

The third way that earthquakes cause collapse is by inducing sufficient lateral displacement into a building to allow $P\Delta$ effects to induce lateral sidesway collapse of the frame. Sidesway collapse can occur in a single story, or can involve multiple stories. Often, it is difficult to distinguish these collapses from the local failures of elements because the large displacements associated with sidesway collapse can often trigger concurrent local collapse.

If the base of a structure is suddenly moved, as in a seismic event, the upper part of the structure will not respond instantaneously, but will lag because of the inertial resistance and flexibility of the structure. The resulting stresses and distortions in the building are the same as if the base of the structure were to remain stationary while time-varying horizontal forces are applied to the upper part of the building. These forces, called inertia forces, are equal to the product of the mass of the structure times acceleration, that is, $F = ma$ (the mass m is equal to weight divided by the acceleration of gravity, i.e., $m = w/g$). Because earthquake ground motion is three-dimensional (3D) (one vertical and two horizontal), the structure, in general, deforms in a highly complex manner. Generally, the inertia forces generated by the horizontal components of ground motion require greater consideration for seismic design since adequate resistance to vertical seismic loads is usually provided by the member capacities required for gravity load design.

The behavior of a building during an earthquake is thus a vibration problem. The seismic motions of the ground do not damage a building by impact, as does a wrecker's ball, or by externally applied pressure such as wind, but by internally generated inertial forces caused by vibration of the building mass. An increase in mass, in this context, has two undesirable effects on the earthquake design. First, it results in an increase in the force, and second, it can cause buckling or crushing of columns and walls when the mass pushes down on a member bent or moved out of plumb by the lateral forces. This effect is known as the $P\Delta$ effect and the greater the vertical forces the greater the movement due to $P\Delta$. It is almost always the vertical load that causes buildings to collapse; in earthquakes, buildings very rarely fall over—they fall down.

In general, tall buildings respond to seismic motions quite differently than low-rise buildings as shown schematically in Figure 6.39. They are invariably more flexible than low-rise buildings, and in general, experience much lower accelerations. But a tall building subjected to ground motions for a prolonged period may experience much larger forces if its natural period is near that of the ground waves. Thus, the magnitude of lateral forces is not a function of the acceleration of the ground alone, but is influenced to a great extent by the type of response of the structure itself and its foundation as well. This interrelationship of building behavior and seismic ground motion also depends on the building period as formulated in the so-called response spectrum, explained elsewhere in this chapter.

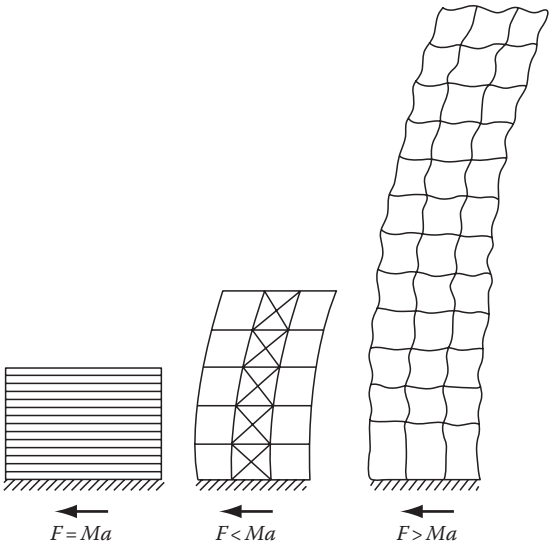


FIGURE 6.39 Schematic magnitude of seismic force. *Note*: The magnitude of seismic force depends on the building mass *M*, ground acceleration *a*, and the dynamic response of the building itself.

An effective seismic design generally includes the following:

1. Selecting an overall structural concept including layout of a lateral-force-resisting system that is appropriate to the anticipated level of ground shaking. This includes providing a redundant and continuous load path to ensure that the building responds as a unit when subject to ground motion.
2. Determining forces and deformations generated by the ground motion, and distributing the former vertically to the lateral-force-resisting system with due consideration to the structural system, configuration, and site characteristics.
3. Analysis of the building for the combined effects of gravity and seismic loads to verify that adequate vertical and lateral strength and stiffness are achieved to satisfy the structural performance and acceptable deformation levels prescribed in the governing building code.
4. Providing details that allow for expected structural movements without damage to nonstructural elements such items as piping, window glass, plaster, veneer and partitions. To minimize this type of damage, special care in detailing, either to isolate these elements or to accommodate the expected movement, is required. Breakage of glass windows can be minimized by providing adequate clearance at edges to allow for frame distortions. Damage to rigid nonstructural partitions can be largely eliminated by providing a detail, which will permit relative movement between the partitions and the adjacent structural elements.
5. In piping installations, the use of expansion loops and flexible joints to accommodate relative seismic deflections between adjacent equipment items and the building floors.
6. Fasten freestanding shelving to walls to prevent toppling.
7. Stairways often suffer seismic damage because they tend to prevent drift between connected floors. This can be avoided by providing a slip joint at the lower end of each stairway to eliminate their bracing effect or by tying stairways to stairway shear walls.

6.2.1.1 Building Motions and Deflections

Earthquake-induced motions, even when they are more violent than those induced by wind, evoke a totally different human response—first, because earthquakes occur much less frequently than windstorms, and second, because the duration of motion caused by an earthquake is generally short,

a few seconds only. People who experience earthquakes are grateful that they have survived the trauma and are less inclined to be critical of the building motion. Earthquake-induced motions are, therefore, a safety rather than a human discomfort issue.

Lateral deflections that occur during earthquakes should be limited to prevent distress in structural members and architectural components. Nonload-bearing in-fills, external wall panels and window glazing should be designed with sufficient clearance or with flexible supports to accommodate the anticipated movements.

6.2.1.2 Building Drift and Separation

Drift is generally defined as the lateral displacement of one floor relative to the floor below. Drift control is necessary to limit damage to interior partitions, elevator and stair enclosures, glass, and cladding systems. Stress or strength limitations in ductile materials do not always provide adequate drift control, especially for tall buildings with relatively flexible moment-resisting frames.

Total building drift is the absolute displacement of any point relative to the base. Adjoining buildings or adjoining sections of the same building may not have identical modes of response and, therefore, may have a tendency to pound against one another. Building separations or joints must be provided to permit adjoining buildings to respond independently to earthquake ground motion.

6.2.1.3 Adjacent Buildings

Buildings are often built right up to property lines in order to make maximum use of space. Historically buildings have been built as if the adjacent structures do not exist. As a result, the buildings may pound during an earthquake. Building pounding can alter the dynamic response of both buildings and impart additional inertial loads to them.

Buildings that are the same height and have matching floors are likely to exhibit similar dynamic behavior. If the buildings pound, floors will impact other floors, so damage usually will be limited to nonstructural components. When floors of adjacent buildings are at different elevations, the floors on one building will impact the columns of the adjacent building, causing structural damage. When buildings are of different heights, the shorter building may act as a buttress for the taller neighbor. The shorter building receives an unexpected load while the taller building suffers from a major discontinuity that alters its dynamic response. Since neither is designed to weather such conditions, there is potential for extensive damage and possible collapse.

One of the basic goals in seismic design is to distribute yielding throughout the structure. Distributed yielding dissipates more energy and helps prevent the premature failure of any one element or group of elements. For example, in moment frames, it is desirable to have strong columns relative to the beams to help distribute the formation of plastic hinges in the beams throughout the building and prevent a story collapse mechanism.

6.2.2 Continuous Load Path

A continuous load path, or preferably more than one path, with adequate strength and stiffness should be provided from the origin of the load to the final lateral-load-resisting elements. The general path for load transfer is in reverse to the direction in which seismic loads are delivered to the structural elements. Thus, the path for load transfer is as follows: Inertia forces generated in an element, such as a segment of exterior curtain wall, are delivered through structural connections to a horizontal diaphragm (i.e., floor slab or roof); the diaphragms distribute these forces to vertical components such as moment frames and braced frames, and finally, the vertical lateral-force-resisting system (LFRS) transfers the forces into the foundations. While providing a continuous load path is an obvious requirement, examples of common flaws in load paths include: a missing collector, or a discontinuous chord because of an opening in the floor diaphragm, or a connection that is inadequate to deliver diaphragm shear to LFRS.

The horizontal elements such as floor and roof slabs distribute lateral forces to the LFRS acting as horizontal diaphragms. In special situations, horizontal bracing may be required in the plane of diaphragms to transfer large shears from discontinuous walls or braces.

A complete load path is a basic requirement. There must be a complete gravity and lateral-force-resisting system that provides a continuous load path between the foundation, and all potions of the building. If there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the elements. Interconnecting the elements needed to complete the load path is necessary to achieve the required seismic performance. Examples of gaps in the load path in addition to those stated earlier would include a shear wall that does not extend to the foundation, a missing shear transfer connection between a diaphragm and vertical elements, a discontinuous chord at a diaphragm's notch or a missing collector.

A good way to remember this important design strategy is to ask yourself the question, "How does the inertia load get from here (meaning the point at which it originates) to there (meaning the shear base of the structure, typically the foundations)?"

6.2.3 Building Configuration

A building with an irregular configuration may be designed to meet all code requirements, but it will not perform as well as a building with a regular configuration. If the building has an odd shape that is not properly considered in the design, good details and construction are of a secondary value.

Two types of structural irregularities, vertical and plan irregularities, are typically defined in most seismic standards. These irregularities result in building responses significantly different from those assumed in the equivalent static force procedure, and to a lesser extent from the dynamic analysis procedure. Although seismic provisions give certain recommendations for assessing the degree of irregularity and corresponding penalties and restrictions, it is important to understand that these recommendations are not an endorsement of their design; rather, the intent is to make the designer aware of the potential detrimental effects of irregularities.

Consider, for example, a reentrant corner, resulting from an irregularity characteristic of a building's plan. If the plan configuration has an inside corner, as shown in Figure 6.40, then it has a reentrant corner. It is however unavoidable in buildings of L, H, T, and X plan shapes.

Two problems related to seismic performance are created by these shapes: (1) differential lateral deformation modes between different wings of the building may result in a local stress concentration at the reentrant corner; and (2) torsion may result because of eccentricity between the center of rigidity and center of mass.

There are two alternative solutions to this problem: Tie the building together at lines of stress concentration and locate seismic-resisting elements at the extremity of the wings to reduce torsion, or separate the building into simple shapes. The width of the separation joint must allow for the estimated inelastic deflections for adjacent wings. The purpose of the separation is to allow adjoining portions of buildings to respond to earthquake ground motions independently without pounding on each other. If it is decided to dispense with the separation joints, collectors at the intersection must be added to transfer forces across the intersection areas. Since the free ends of the wings tend to distort most, it is beneficial to place seismic-resisting members at these locations.

The seismic design of regular buildings is based on two concepts. First, the linearly varying lateral force distribution is a reasonable and conservative representation of the actual response distribution due to earthquake ground motions. Second, the cyclic inelastic deformation demands are reasonably uniform in all of the seismic force-resisting elements. However, when a structure has irregularities, these concepts may not be valid, requiring corrective factors and procedures to meet the design objectives.

The impact of irregular parameters in estimating seismic force levels, first introduced into the Uniform Building Code (UBC) in 1973, long remained a matter of engineering judgment. Beginning in 1988, however, some configuration parameters where quantified to establish the condition of irregularity.

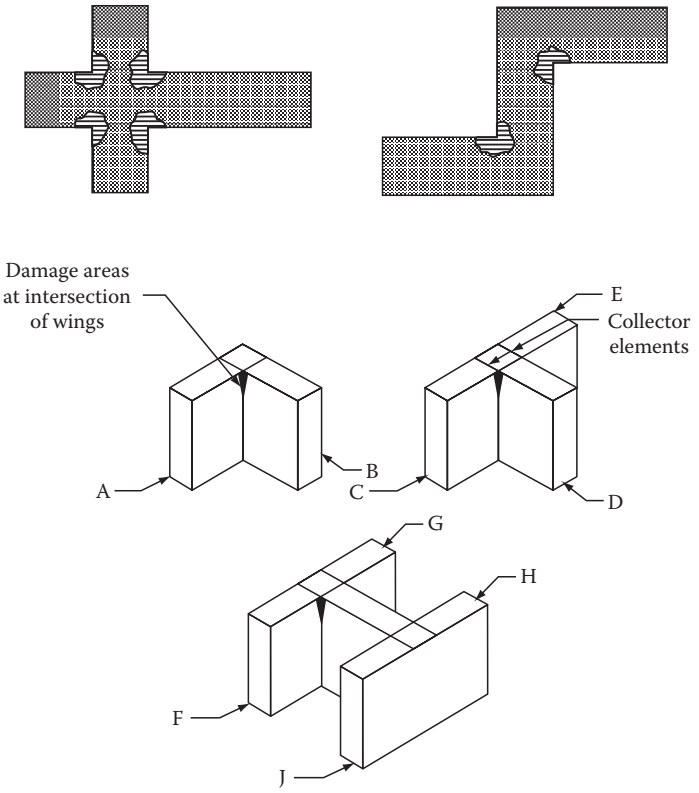


FIGURE 6.40 Reentrant corners in L-, T-, and H-shaped buildings. (As a solution, add collector elements and/or stiffen end walls A, B, C, D, E, F, G, H, and J.)

Additionally specific analytical treatments and/or corrective measures have been mandated to address these flaws.

Typical building configuration deficiencies include an irregular geometry, a weakness in a story, a concentration of mass or a discontinuity in the lateral-force-resisting system. Vertical irregularities are defined in terms of strength, stiffness, geometry, and mass. Although these are evaluated separately, they are related to one another, and may occur simultaneously. For example, a building that has a tall first story can be irregular because of a soft story, a weak story or both, depending on the stiffness and strength of this story relative to those above.

Those who have studied the performance of buildings in earthquakes generally agree that the building's form has a major influence on performance. This is because the shape and proportions of the building have a major effect on the distribution of earthquake forces as they work their way through the building. Geometric configuration, type of structural members, details of connections, and materials of construction all have a profound effect on the structural-dynamic response of a building. When a building has irregular features, such as asymmetry in plan or vertical discontinuity, the assumptions used in developing seismic criteria for buildings with regular features may not apply. Therefore, it is best to avoid creating buildings with irregular features. For example, omitting exterior walls in the first story of a building to permit an open ground floor leaves the columns at the ground floor level as the only elements available to resist lateral forces, thus causing an abrupt change in rigidity at that level. When irregular features are unavoidable, special design considerations are required to account for the unusual dynamic characteristics and the load transfer and stress concentrations that occur at abrupt changes in structural resistance. Examples of plan and elevation irregularities are illustrated in Figures 6.41 and 6.42. Note that plan irregularities are also referred to as horizontal irregularities.

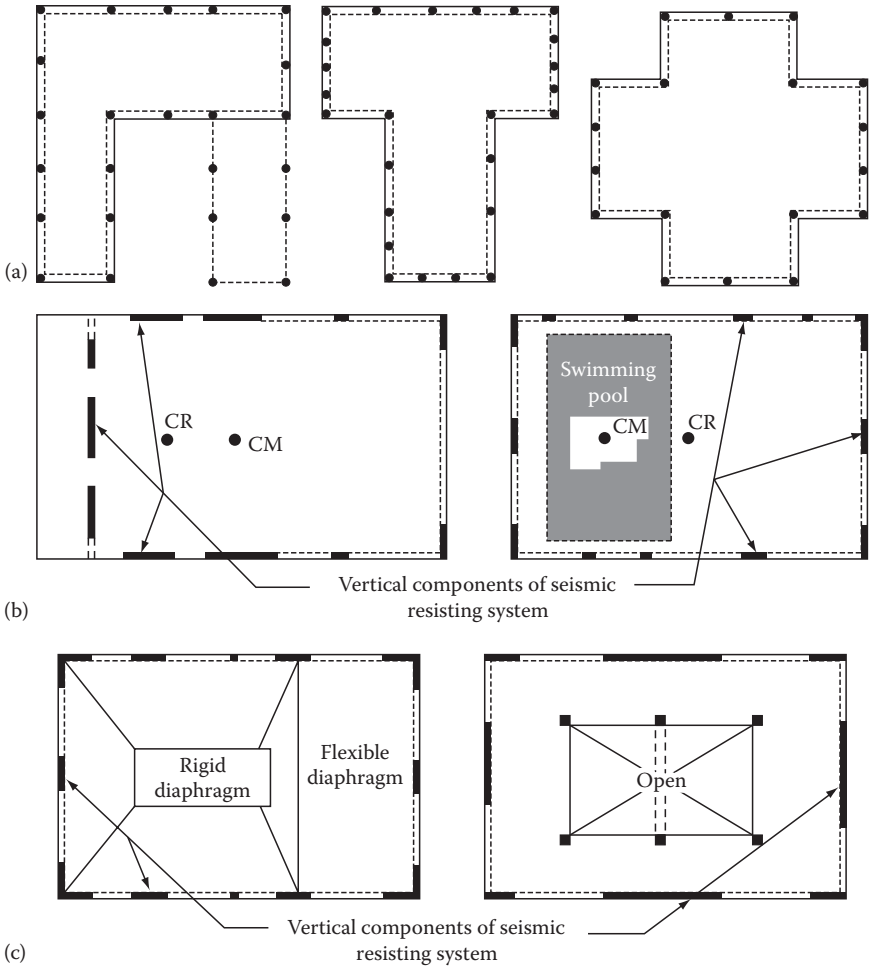


FIGURE 6.41 Plan irregularities: (a) geometric irregularities, (b) irregularity due to mass-resistance eccentricity, and (c) irregularity due to discontinuity in diaphragm stiffness. *Note*: CR, center of resistance, CM, center of mass.

The ASCE 7-05 qualifies irregularity by defining geometrically or by use of dimensional ratios, the points at which a specific irregularity becomes an issue requiring remedial measures. These issues are discussed later in this chapter. It is worth noting that no corrective course is required for certain irregularities, other than performing modal analysis for determining design seismic forces.

As stated previously irregularities are divided into two broad categories: (1) vertical and (2) plan irregularities. Vertical irregularities include soft or weak stories, large changes in mass from floor to floor, and discontinuities in the dimensions or in-plane locations of lateral-load-resisting elements. Buildings with plan irregularities include those that experience substantial torsion when subjected to seismic loads, or have reentrant corners, discontinuities in floor diaphragms, or in the lateral force path, or have lateral-load-resisting elements that are not parallel to each other or to the principal axes of the building.

6.2.4 Influence of Soil

It is quite evident that the intensity of ground motion reduces with the distance from the epicenter of the earthquake. The reduction, called attenuation, occurs at a faster rate for high-frequency

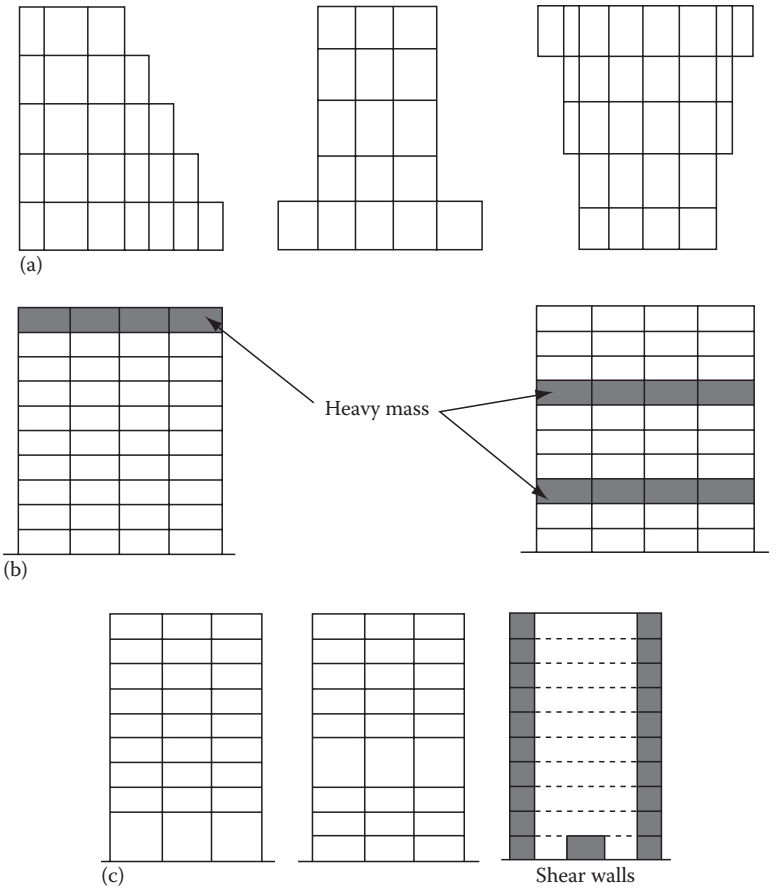


FIGURE 6.42 Elevation irregularities: (a) abrupt change in geometry, (b) large difference in floor masses, and (c) large difference in story stiffnesses.

(short-period) components than for lower-frequency (long-period) components. The cause of the change in attenuation rate is not understood, but its existence is certain. This is a significant factor in the design of tall buildings, because a tall building, although situated farther from a causative fault than a low-rise building, may experience greater seismic loads because long-period components are not attenuated as fast as the short-period components. Therefore, the area influenced by ground shaking potentially damaging to, say, a 50-story building is much greater than for a one-story building.

As a building vibrates due to ground motion, its acceleration will be amplified if the fundamental period of the building coincides with that of the soil it rests upon. It is worth noting that natural periods of soil are typically in the range of 0.5–1.0 s. Therefore, it is entirely possible for a building and ground it rests upon, to have the same fundamental period. This was the case for many 5–10-story buildings that were damaged in the September 1985 earthquake in Mexico City.

Experience in several other earthquakes has confirmed that local soil conditions can have a significant effect on earthquake response. It is perhaps more challenging to picture, but the soil layers beneath a structure have a period of vibration T_{soil} similar to the period of vibration of a building T. Greater structural damage is likely to occur when the period of the underlying soil is close to the fundamental period of the structure. In these cases, a partial resonance effect may develop between the structure and the underlying soil. These conditions are addressed in ASCE 7-05 by classifying soil profiles, into Site Class A through F.

6.2.5 Ductility

Ductility is the property exhibited by certain structural elements, and structures composed of such elements, that enables them to sustain loads when strained beyond their elastic limit. For structures that have well-defined yield and ultimate deformation capacities, ductility, μ, is defined by

$$\mu = \frac{\delta_u}{\delta_y}$$

In this equation, δ_u and δ_y, are the displacements at which failure and yielding, respectively, initiate.

Ductility is an important parameter in computing seismic resistance. It enables structures that do not have adequate elastic strength to survive strong ground motions through inelastic response. The principal benefit of ductile response is that it makes it possible to place ductile elements at key locations in the seismic load–resisting system to protect other nonductile elements from being overstressed. This is a key strategy in design of structures for seismic resistance.

As stated earlier, in seismic design, structures are designed for forces much smaller than those that occur in a linear-elastic structure. The force reduction is possible for a number of reasons. As the structure begins to yield and deform inelastically, the effective period of the structure tends to lengthen resulting in a reduction in strength demand. Furthermore, the inelastic action results in a significant amount of energy dissipation through hysteretic damping.

The energy dissipation resulting from hysteretic behavior as explained previously in Section 6.1.8, can be measured as the area enclosed by the force-deformation curve of the structure as it experiences several cycles of inelastic excitation. Some structures have far more energy dissipation capacity than do others. The extent of energy dissipation capacity is largely dependent on the amount of stiffness and strength degradation of the structure as it experiences repeated cycles of inelastic deformation.

Let us consider the load deformation curves for a beam–column assembly shown in Figure 6.43. Hysteretic curve (a) in the figure is representative of the behavior of substructures that have been detailed for ductile behavior. The substructure can maintain nearly all of its strength and stiffness over a number of large cycles of inelastic deformation. The resulting force-deformation "loops" are quite wide and open, resulting in a large amount of energy dissipation capacity. Hysteretic curve (b) represents the behavior of a substructure that has not been detailed as well for ductile behavior. It rapidly loses stiffness under inelastic deformation and the resulting hysteretic

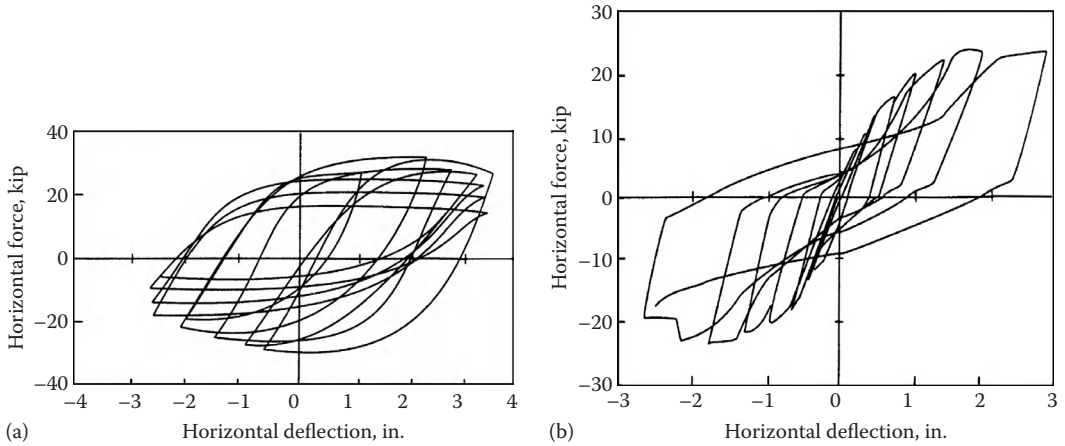


FIGURE 6.43 Hysteric behavior: (a) curve representing large energy dissipation and (b) curve representing limited energy dissipation.

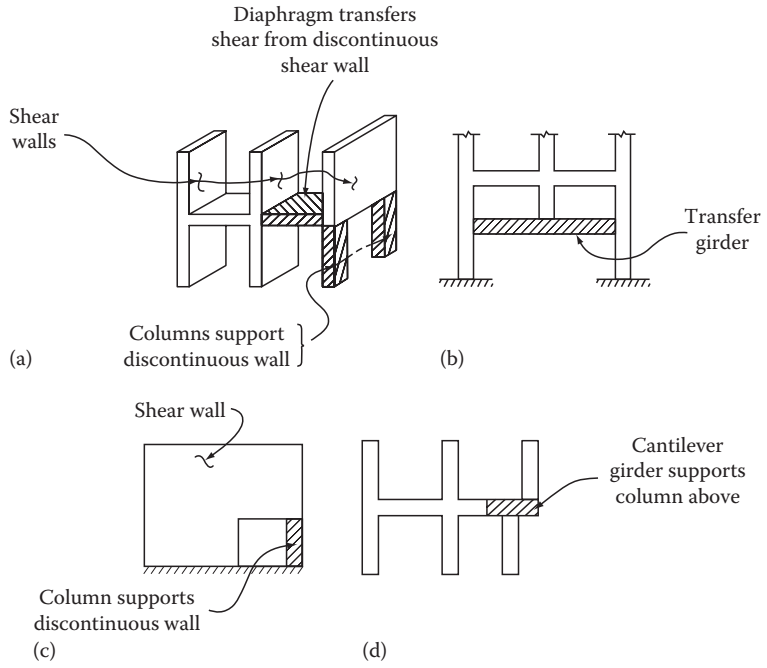


FIGURE 6.44 Examples of nonuniform ductility in structural systems due to vertical discontinuities.

loops are quite pinched. The energy dissipation capacity of such a substructure is much lower than that for the substructure (a). Hence, structural systems with large energy dissipation capacity are assigned higher values for response coefficient, R, resulting in lower design forces, than for systems with relatively limited energy dissipation capacity.

In providing for ductility, it should be kept in mind that severe penalties are imposed by seismic provisions on structure with nonuniform ductility. Some examples are shown in Figure 6.44a.

6.2.6 Redundancy

Redundancy is a fundamental characteristic for good seismic performance. It provides a building with a spare system such that failure of a single connection or component does not adversely affect the entire lateral stability of the structure

6.2.7 Damping

Buildings do not resonate with the purity of a tuning fork because they are damped; the extent of damping depends upon the construction materials, type of connections and the influence of nonstructural elements on the stiffness characteristics of the building. Damping is measured as a percentage of critical damping.

In a dynamic system, critical damping is defined as the minimum amount of damping necessary to prevent oscillation altogether. To visualize critical damping, imagine a tensioned string immersed in water. When the string is plucked, it oscillates about its rest position several times before stopping. If we replace water with a liquid of high viscosity, the string will oscillate, but certainly not as many times as it in water. By progressively increasing the viscosity of the liquid, it is easy to visualize that a state can be reached where the string, once plucked, will return to its neutral position without ever crossing it. The minimum viscosity of the liquid that prevents the vibration of the string altogether can be considered equivalent to the critical damping.

The damping of structures is influenced by a number of external and internal sources. Chief among them are

1. External viscous damping caused by air surrounding the building. Since the viscosity of air is low, this effect is negligible in comparison to other types of damping.
2. Internal viscous damping associated with the material viscosity. This is proportional to velocity and increases in proportion to the natural frequency of the structure.
3. Friction damping, also called Coulomb damping, occurring at connections and support points of the structure. It is a constant, irrespective of the velocity or amount of displacement.
4. Hysteretic damping that contributes to a major portion of the energy absorbed in ductile structures.

For analytical purposes, it is a common practice to lump different sources of damping into a single viscous damping. For nonbase-isolated buildings, analyzed for code-prescribed loads, the damping ratios used in practice vary anywhere from 1% to 10% of critical. The low-end values are for wind, while those for the upper end are for seismic design. The damping ratio used in the analysis of seismic base-isolated building is rather large compared to values used for nonisolated buildings, and varies from about 0.20 to 0.35 (20%–35% of critical damping).

Base isolation, discussed in Chapter 9, consists of mounting a building on an isolation system to prevent horizontal seismic ground motions from entering the building. This strategy results in significant reductions in interstory drifts and floor accelerations, thereby protecting the building and its contents from earthquake damage.

A level of ground acceleration on the order 0.1g, where g is the acceleration due to gravity, is often sufficient to produce some damage to weak construction. An acceleration of 1.0g, or 100% of gravity, is analytically equivalent, in the static sense, to a building that cantilevers horizontally from a vertical surface as shown in Figure 6.45.

As stated previously, the process by which free vibration steadily diminishes in amplitude is called damping. In damping, the energy of the vibrating system is dissipated by various mechanisms, and often more than one mechanism may be present at the same time. In simple laboratory models, most of the energy dissipation arises from the thermal effect of repeated elastic straining of the material and from the internal friction. In actual structures, however, many other mechanisms also contribute to the energy dissipation. In a concrete building, these include opening and closing of microcracks in concrete, friction between the structure itself and nonstructural elements such as partition walls. Invariably it is impossible to identify or describe mathematically each of these energy-dissipating mechanisms in an actual building.

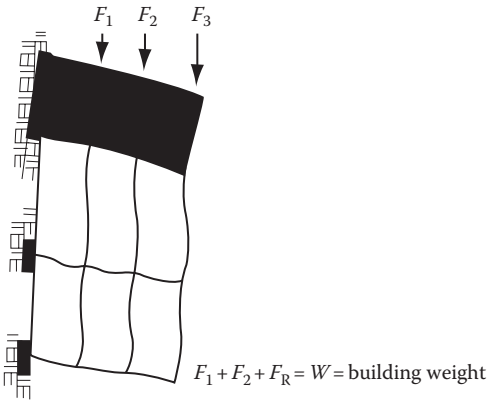


FIGURE 6.45 Concept of 100% g. A building subjected to 1g acceleration conceptually behaves as if it cantilevers horizontally from a vertical surface. *Note*: This is not a misprint!

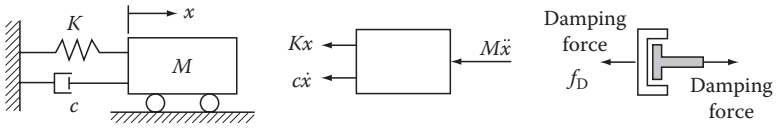


FIGURE 6.46 Linear viscous damper; damping is defined as a force that resists dynamic motion. A simple and yet realistic damping model for analysis purposes is to assume that the damping force, f_D, is proportional to viscous friction of a fluid in a dash pot, and therefore it is called *viscous damping*.

Therefore, the damping in actual structures is usually represented in a highly idealized manner. For many purposes the actual damping in structures can be idealized by a linear viscous damper or dashpot. The damping coefficient is selected so that the vibrational energy that dissipates is equivalent to the energy dissipated in all the damping mechanisms. This idealization is called equivalent viscous damping.

Figure 6.46 shows a linear viscous damper subjected to a force f_D. The damping forces f_D is related to the velocity $\dot{u}$ across the linear viscous damper by

$$f_D = c\dot{u}$$

where the constant c is the viscous damping coefficient; it has units of force × times/length.

Unlike the stiffness of a structure, the damping coefficient cannot be calculated from the dimensions of the structure and the sizes of the structural elements. This is understandable because it is not feasible to identify all the mechanisms that dissipate vibrational energy of actual structures. Thus, vibration experiments on actual structures provide the data for evaluating the damping coefficient. These may be free vibration experiments that lead to measured rate at which motion decays. The damping property may also be determined from forced vibration experiments.

The equivalent viscous damper is intended to model the energy dissipation at deformation amplitudes within the linear elastic limit of the overall structure. Over this range of deformations, the damping coefficient c determined from experiments may vary with the deformation amplitude. This nonlinearity of the damping property is usually not considered explicitly in dynamic analyses. It may be handled indirectly by selecting a value for the damping coefficient that is appropriate for the expected deformation amplitude, usually taken as the deformation associated with the linearly elastic limit of the structure. Additional energy is dissipated due to inelastic behavior of the structure at larger deformations. Under cyclic forces or deformations, this behavior implies formation of a force–deformation hysteresis loop as shown in Figure 6.47. The damping energy dissipated during one deformation cycle between deformation limits $\pm u_o$ is given by the area within the hysteresis loop (Figure 6.47). This energy dissipation is usually not modeled by a viscous damper, especially

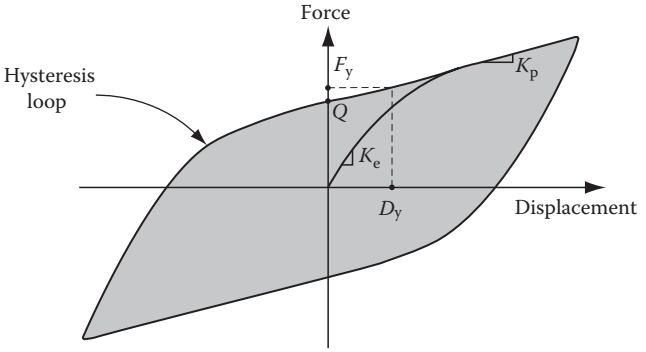


FIGURE 6.47 Force displacement hysteresis loop: the area inside of the loop is a measure of energy dissipation due to nonelastic behavior. *Note*: K_e = initial elastic stiffness, K_p = stiffness in the plastic range, F_y = stress at yield, D_y = deformation at yield.

if the excitation is earthquake ground motion. Instead, the most common and direct approach to account for the energy dissipation through inelastic behavior is to recognize the inelastic relationship between resisting force and deformation. Such force–deformation relationships are obtained from experiments on structures or structural components at slow rates of deformation, thus excluding any energy dissipation arising from rate-dependent effects.

6.2.8 Diaphragms

Buildings are composed of vertical and horizontal structural members that resist lateral forces caused by wind and seismic. The primary purpose of a diaphragm, which consists of the roof and floors of a building, is to transfer the horizontal forces to the vertical elements resisting the lateral loads.

It is customary to consider a diaphragm analogous to a deep plate girder laid in a horizontal plane. In a composite floor system, the metal deck and concrete topping perform the function of the plate girder web. Just as the plate girder flanges carry flexural forces, so must the diaphragm elements located at the perimeter. These edge members are commonly referred to as *chords* using the analogy of a truss spanning between supports. The resulting forces in the chords are either tension or compression acting in a direction perpendicular to the direction of lateral loading under consideration. The diaphragm chord forces at any point along the diaphragm boundary member are equal to the diaphragm moment at that point divided by the depth of the diaphragm at that location. In the absence of such chord members to take the moment couple, the diaphragm must act as a deep plate resisting both bending and shear forces.

Drag force is the tension or compression force in the diaphragm boundary member parallel to the direction of loading under consideration. Where reentrant corner irregularities are present, drag struts (also called *collectors*) are required to prevent localized tearing of the diaphragm. The collector essentially collects the diaphragm shear and delivers it to the vertical elements resisting the lateral loads.

The stiffness of the diaphragm has an important effect on the proportionate distribution of lateral loads to various components of the lateral support system. This effect is displayed in Figure 6.48, which shows how the relative rigidities of diaphragm and lateral support elements influence load distribution. At one extreme is the *rigid diaphragm* which distributes the lateral loads to the supports in proportion to *just* their rigidities. At the other extreme is the flexible diaphragm, which delivers lateral loads based on tributary area. The diaphragm which falls in between the two extremes is the *semi-rigid diaphragm* in which the load distribution involves both aspects of behavior. All three types of diaphragm behavior require effective transfer of bending and shear forces in the plane of the diaphragm, necessitating careful design and detailing of connections between diaphragm elements and the lateral support system.

In steel buildings, floor diaphragms most commonly consist of composite metal deck with concrete topping. Of concern is the design of the diaphragm itself, and the transfer of diaphragm forces into the lateral-load-resisting system.

Depending upon the magnitude of lateral load to be transferred to the vertical system, the designer has two choices in detailing the force transfer: either assign the transfer to occur along the entire frame width or to a selected segment of the frame width. A combination of the two is of course, quite logical.

As stated previously, tension and compression chord forces are developed at the perimeter of the diaphragm due to lateral loads. The topping concrete slab over metal deck can be typically designed to resist the compression chord forces. Tension chord forces can be resisted by spandrel steel beams, continuous steel closure plates, or by mild steel reinforcement placed within the concrete topping.

Earthquake loads at any given level of a building are distributed to the lateral-load-resisting vertical elements through the floor and roof slabs. For analytical purpose, the diaphragms are assumed to behave as deep beams. The slab is the web of the deep beam carrying shear, and the perimeter

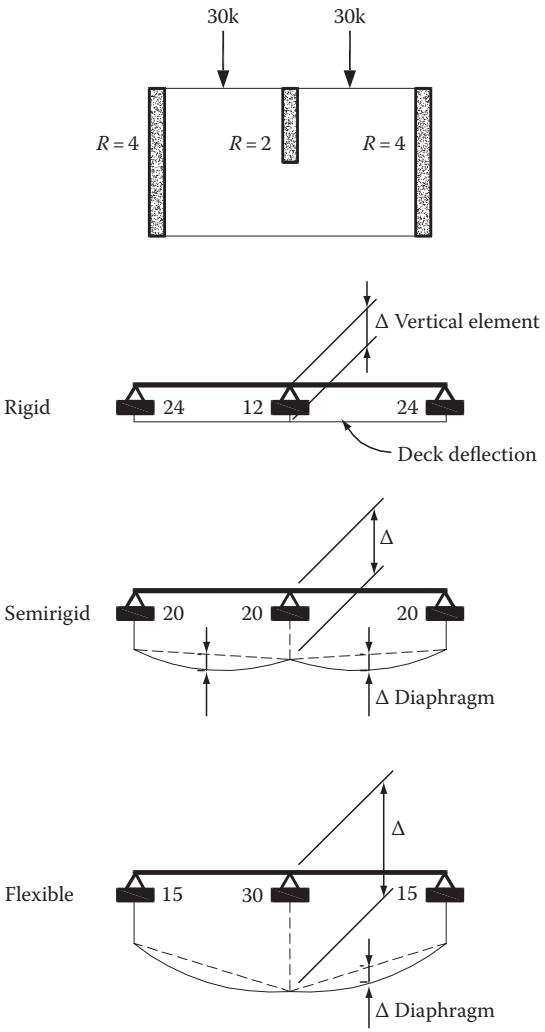


FIGURE 6.48 Relative effects of diaphragm stiffness.

spandrel or shear wall if any, is the flange of the beam resisting bending (see Figure 6.49a and b). In the absence of perimeter members, the slab is analyzed as a plate subjected to in-plate bending.

Three factors are important in diaphragm design:

1. The diaphragm must be adequate to resist both the bending and shear stresses and be tied together to act as one.
2. The collectors and drag members (see Figure 6.49) must be adequate to transfer loads from the diaphragm into the lateral-load-resisting vertical elements.
3. Openings or reentrant corners in the diaphragm must be property placed and adequately reinforced.

Inappropriate location or large-size openings such as for stairs or elevator cores, atriums, skylights, etc. create problems similar to those related to cutting of beam flanges and holes in the beam web of a beam adjacent to the flange. This reduces the ability of the diaphragm to transfer the chord forces and if not designed properly, may cause rupture in the web (Figure 6.50).

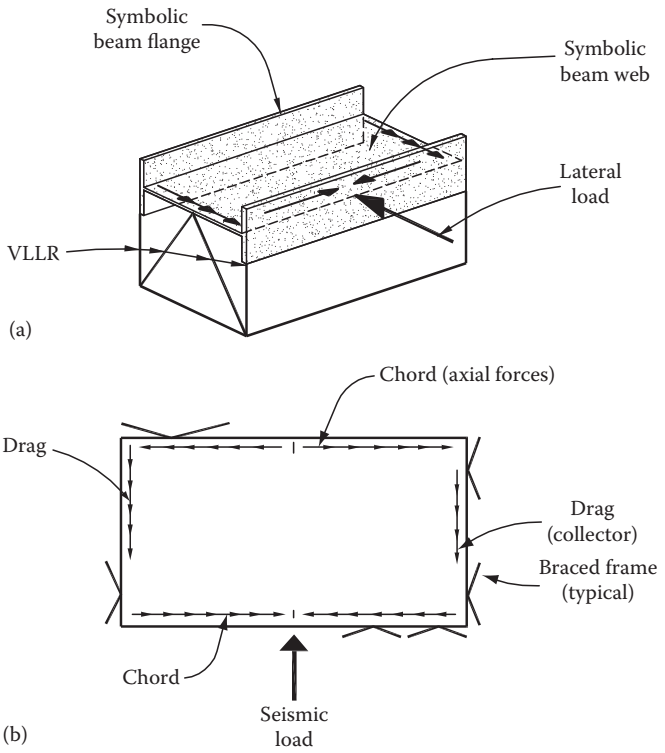


FIGURE 6.49 (a) Diaphragm action of floor or roof system. *Note*: VLLR, vertical–lateral-load-resisting system. (b) Schematic drag and chord for north-south seismic loads.

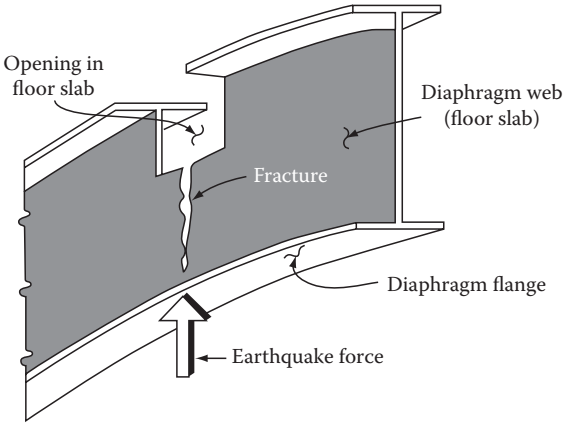


FIGURE 6.50 Diaphragm web failure due to large opening.

6.2.9 Response of Elements Attached to Buildings

Elements attached to the floors of buildings (e.g., mechanical equipment, ornamentation, piping, nonstructural partitions) respond to floor motion in much the same manner as the building responds to ground motion. However, the floor motion may vary substantially from the ground motion. The high-frequency components of the ground motion that correspond to the natural periods of vibrations of the building tend to be magnified. If the elements are rigid and are rigidly attached to the structure, the

forces on the elements will be in the same proportion to the mass as the forces on the structure. But elements that are flexible and have periods of vibration close to any of the predominant modes of the building vibration will experience substantially greater than the forces on the structure.

When individual elements of the structure are analyzed separately, it becomes necessary to consider seismic effects differently than for LFRS or for diaphragms. The reason for this is that certain elements which are attached to the structure respond dynamically to the motion of the structure rather than to the motion of the ground. Resonance between the structure and the attached elements may occur.

Seismic forces and design requirements for nonstructural components are addressed in ASCE 7-05 Sec. 13.3. The component forces also apply to a wide range of architectural, mechanical, and electrical components. Architectural components addressed include interior and exterior nonstructural walls and partitions, parapets, chimneys, veneer, ceilings, cabinets, etc. In fact, in higher SDCs virtually everything on or attached to the structure requires design per ASCE 7 Chap. 13.

ASCE 7-05 Sec. 13.1.4 provides a list of four categories of components exempt from design:

1. In SDC B, all architectural components having component importance factor I_p equal to 1.0 are exempt, except parapets supported by bearing walls or shear walls.
2. In SDC B, all mechanical and electrical components are exempt.
3. In SDC C, all mechanical and electrical components having I_p equal to 1.0 are exempt.
4. In SDC D, E, and F, mechanical and electrical components with I_p equal to 1.0 and meeting certain weight and/or installation restrictions are exempt.

The seismic force on a portion of the structure is given by the following formula:

$$F_p = \frac{0.4 a_p S_{DS} W_p}{R_p / I_p} \left(1 + 2\frac{z}{h}\right)$$

with F_p limited to the range:

$$0.3 S_{DS} I_p W_p \text{ " } F_p \text{ " } 1.6 S_{DS} I_p W_p$$

where

F_p is the component seismic design force centered at the component's center of gravity and distributed relative to the component's mass distribution
S_{DS} is the short-period design spectral acceleration
a_p is the component amplification factor per ASCE 7 Section 13.5-1 or 13.6-1
I_p is the component importance factor per ASCE 7 Section 13.1.3
W_p is the component operating weight
R_p is the component response modification factor per ASCE 7 Table 13.5-1 or 13.6-1
z is the height in structure of point of attachment of component with respect to the structure base. For items attached at or below the base, z/h need not exceed 1.0
h is the average roof height of structure with respect to the structure base

This equation for F_p was developed based on building acceleration data recorded during earthquakes. The term $1 + 2z/h$ allows F_p to vary from one value for components anchored at the ground level to three times the value of components anchored to the roof. This matches the general trend seen in recorded acceleration data. If the elevation at which the component will be anchored is not known, this term can default to 3.

Typically, F_p will be considered in two perpendicular horizontal directions, combined with a vertical force component of plus or minus $0.2 S_{DS} W_p$. Where the component under consideration is a vertically cantilevered system, F_p will be considered to act in any horizontal direction. It is

specifically noted that the redundancy factor ρ will be taken as 1.0 for components, and that use of overstrength Ω_o forces in combination with F_p forces is not required. As with the seismic base shear equations, component forces F_p are at a strength level. Appropriate load factors are to be used in the load combination equations.

6.3 ASCE 7-05 SEISMIC DESIGN CRITERIA AND REQUIREMENTS: OVERVIEW

ASCE provisions for seismic design are much broader than determining only the loads for which the structure must be designed. There are a number of requirements relating to the systems that may be used in high seismic areas, to the detailing required in those systems, and to structural configuration issues.

Experience has proven that *regular structures* (i.e., symmetric structures and structures without discontinuities) perform much better in an earthquake than irregular structures. Therefore, even if a nonlinear procedure is used in design, ASCE 7 penalizes certain *irregular structures* in areas of high seismic risk with *additional design and detailing requirements.*

From structural dynamics it is known that a number of different forces act on a structure during an earthquake. These forces include inertia forces, damping forces, elastic forces and an equivalent forcing function (mass times ground acceleration).

A range of approaches has been developed to analyze the loads and displacements that occur in buildings due to earthquakes. Buildings are complex combinations of structural and nonstructural materials, all of which influence the response of a structure to seismic ground motion. The purpose of analysis for seismic loads is to make sure that the structure being designed meets the minimum strength and deflection requirements of the ASCE 7.

Rather than attempting to define all of the forces acting during an earthquake, the equivalent lateral force procedure takes a simplified approach. This empirical method is one that is particularly easy to visualize. The earthquake force is treated as an inertial problem only. Before the start of an earthquake, a building is in static equilibrium (i.e., it is at rest). Suddenly, the ground moves, and the structure attempts to remain stationary. The key to the problem is, of course, the length of time during which the movement takes place. If the ground displacement were to take place slowly, the structure would simply ride along quite peacefully. However, because the ground movement occurs quickly, the structure lags behind and "seismic" forces are generated.

Seismic forces are generated by acceleration of the building mass. Typical practice is to consider that a lump of mass acts at each story level. This concept results in equivalent static forces being applied at each story level (i.e., at the roof and floors), and for many buildings produces reasonable structural designs.

It should be noted, however, that there are structure configurations for which ASCE 7 will require the use of an analysis method more sophisticated than the equivalent lateral force procedure.

The empirical forces given in the equivalent lateral force procedure are considerably lower that would be expected in a major earthquake.

The key to this singular philosophy lies in *design* and *detailing* of the structure so that there is sufficient *system ductility* for the building to remain structurally safe when forced into the *inelastic* range in a major earthquake.

Therefore, in areas of high seismic risk, *detailing requirements* apply to all of the structural building materials including steel. The term "detailing" here refers to special connection design provisions and to a general tying together of the overall LFRS so that there is a *continuous path* for the *transfer of forces* from the top of the structure down in the foundation.

Chapter 11 of ASCE 7-05 includes introductory material required for establishing seismic design requirements for structures assigned to SDC A through F. It defines how to construct a general design response spectrum by using acceleration parameters S_s and S_1 and explains the procedures for establishing the seismic importance factor, I_E and the SDCs A through F.

The limitation for citing SDC E and F buildings is given in the final sections of Chapter 11 followed by the requirements for the investigation of the building site for potential geologic and seismic hazards.

Tucked in-between the introductory material and the requirements of geotechnical investigation are the seismic design requirements for SDC A buildings. If the building being designed is assigned to SDC A, the designer is not required to comply with the requirements of other seismic chapters.

6.3.1 Seismic Ground Motion Values, S_s and S_1

Two parameters, S_s and S_1, play a key role in the determination of ground motion values used in seismic design. These are derived using the maps given in the ASCE Provisions or by accessing web-based information. The seismicity maps showing the contours of 5% damped 0.2 s and 1 s spectral acceleration values for the MCE ground motions are based on the 2002 USGS probabilistic maps. These maps incorporate improved earthquake data in terms of updated fault parameters (such as slip rates, recurrence time, and magnitude), additional attenuation parameters, also referred to as ground motion prediction equations. The interpolated ground motion for the contiguous 48 states by latitude and longitude are calculated using a closer grid spacing in areas of known fault regions.

S_s is the mapped value of the 5%-damped maximum considered earthquake spectral response acceleration, for short period structures founded on Class B, firm rock, sites. The short period acceleration has been determined at a period of 0.2 s. This is because 0.2 s is a reasonable representation of the shortest effective period of buildings that are designed by the ASCE Provisions considering the effects of soil compliance, foundation rocking and other factors typically neglected in structural analysis.

Similarly, S_1, is the mapped value of the 5% damped maximum considered earthquake spectral response acceleration at a period of 1 s on Site Class B. The spectral response acceleration at periods other than 1 s can typically be derived from the acceleration at 1 s. Consequently, these two response acceleration parameters, S_s and S_1 are sufficient to define an entire response spectrum for the period range of importance for most buildings. See Figures 6.51 and 6.52 for mapped ground motion acceleration values S_s and S_1.

In order to obtain acceleration response parameters that are appropriate for sites other than Site Class B, it is necessary to modify the S_s and S_1 values. This modification is performed using two

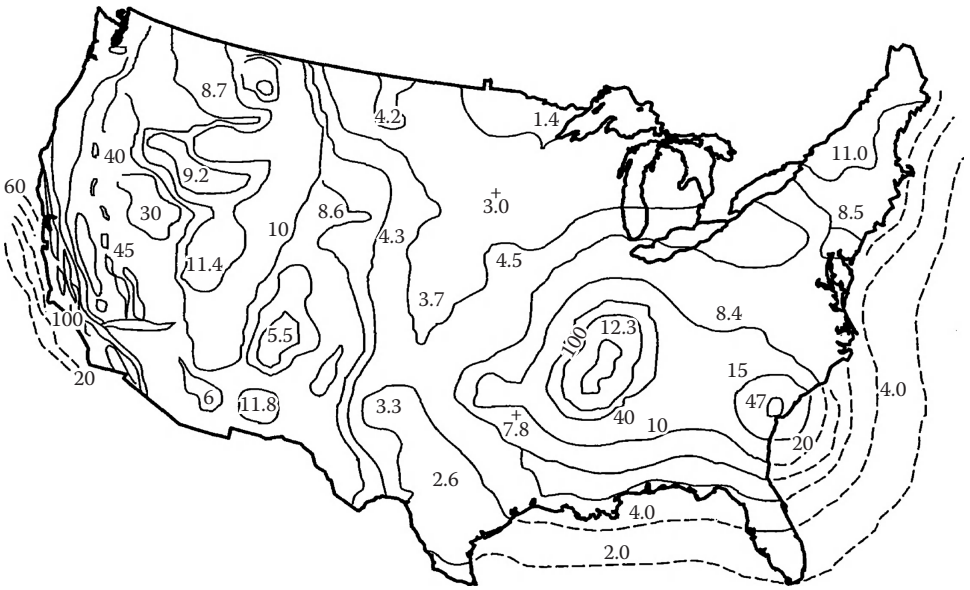


FIGURE 6.51 MCE ground motion for 0.2 s period. Spectral response acceleration, S_s, as a percent of gravity, Site Class B, 5% critical damping.

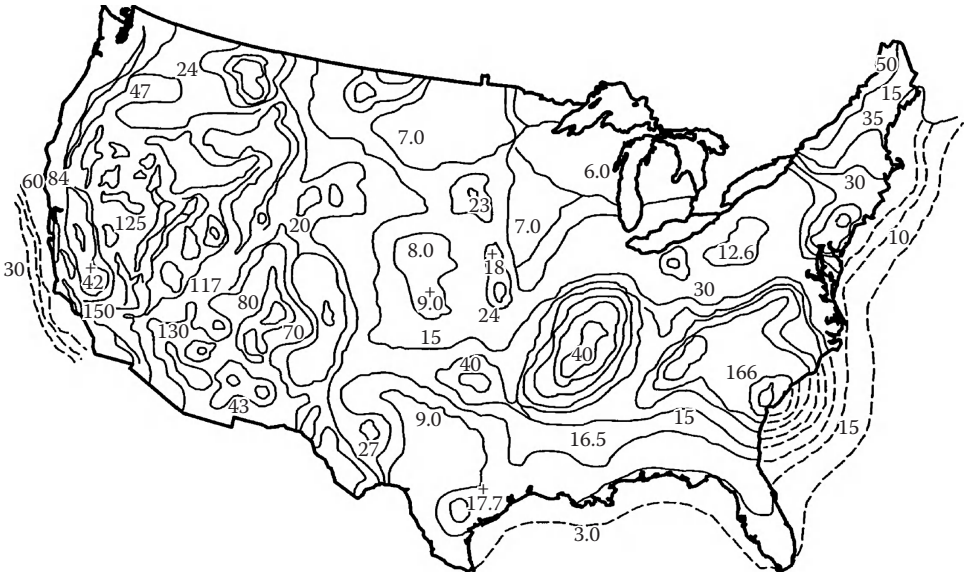


FIGURE 6.52 MCE ground motion for 1.0 s period. Spectral response acceleration, S_1, as a percent of gravity, Site Class B, 5% critical damping.

coefficients, F_a and F_v, to scale the S_s and S_1 values determined for firm rock site. The maximum considered earthquake spectral response accelerations adjusted for site class are designated S_{MS} and S_{M1}, respectively, for short-period and 1 s period response.

6.3.2 Site Coefficients F_a and F_v

Strong-motion recordings obtained on a variety of geologic deposits during the Loma Prieta earthquake of October 17, 1989, provided an important basis for the development of the site coefficients F_a and F_v. The data showed that the peak acceleration at zero-period of the response spectra of the soft clay site relative to rock were extremely high, ranging from about three to six times. Based on the analysis of these strong-motion recordings, ASCE 7-05 has provided values for F_a and F_v in Tables 11.4-1 and 11.4-2. These are shown in Tables 6.1 and 6.2, of this text.

TABLE 6.1
Site Coefficient, F_a

	Mapped MCE Spectral Response Acceleration Parameter at Short Period				
Site Class	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7, ASCE 7-05				

Source: ASCE 7-05, Table 11.4-1.
Note: Use straight-line interpolation for intermediate values of S_s.

TABLE 6.2
Site Coefficient, F_v

	Mapped MCE Spectral Response Acceleration Parameter at 1 s Period				
Site Class	**$S_1 \leq 0.1$**	**$S_1 = 0.2$**	**$S_1 = 0.3$**	**$S_1 = 0.4$**	**$S_1 \geq 0.5$**
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7, ASCE 7-05				

Source: ASCE 7-05.
Note: Use straight-line interpolation for intermediate values of S_1.

6.3.3 Site Class S_A, S_B, S_C, S_D, S_E, and S_F

A set of six site classifications, S_A through S_F, based on the average properties of the upper 100 ft of soil profile are defined in ASCE-05 Table 20.3-1. Since, in practice, geotechnical investigations are seldom conducted to depths of 100 ft, ASCE 7 allows the geotechnical engineers to determine site class based on site-specific data and professional judgment.

The site class should reflect the soil conditions that will affect the ground motion input to the structure or a significant portion of the structure. For structures receiving substantial ground motion input from shallow soils (e.g., structures with shallow spread footings, laterally flexible piles or structures with basements where it is judged that substantial ground motion input to the structure may come through the side walls), it is reasonable to classify the site on the basis of the top 100 ft (30 m) of soils below the ground surface. Conversely, for structures with basements supported on firm soils or rock below soft soils, it is reasonable to classify the site on the basis of the soil or rock below the foundation, if it can be justified that the soils contribute very little to the response of the structure.

Buildings on sloping bedrock sites and/or having highly variable soil deposits across the building area require careful study since the input motion may vary across the building (for example, if a portion of the building is on rock and the rest is over weak soils). Site-specific studies including 2D or 3D modeling may be appropriate in such cases to evaluate the subsurface conditions and site and superstructure response. Other conditions that may warrant site-specific evaluation include the presence of low shear wave velocity soils below a depth of 100 ft (30 m), location of the site near the edge of a filled-in basin, or other subsurface or topographic conditions with strong 2D and 3D site-response effects.

6.3.4 Response Spectrum for the Determination of Design Base Shear

Design base shear is the total lateral force or shear at the base of the building. It is also equal to the sum of the seismic design forces at each level of a building. The symbol V is used to represent the base shear. For the purpose of calculating the base shear, the base of the building is the level at which the earthquake forces are considered to be imparted to the structure or the level at which the structure, as a dynamic vibrator is supported.

The total horizontal base shear, V, is calculated from an expression which is essentially in the form:

$$F = Ma = \left(\frac{W}{g}\right)a = W\left(\frac{a}{g}\right)$$

where

F is the inertia force
M is the mass
a is the acceleration
g is the acceleration of gravity

The ASCE 7 form of this expression is somewhat modified. The (*a*/*g*) term is replaced by a "seismic base shear coefficient C_s, also referred to as the seismic response coefficient."

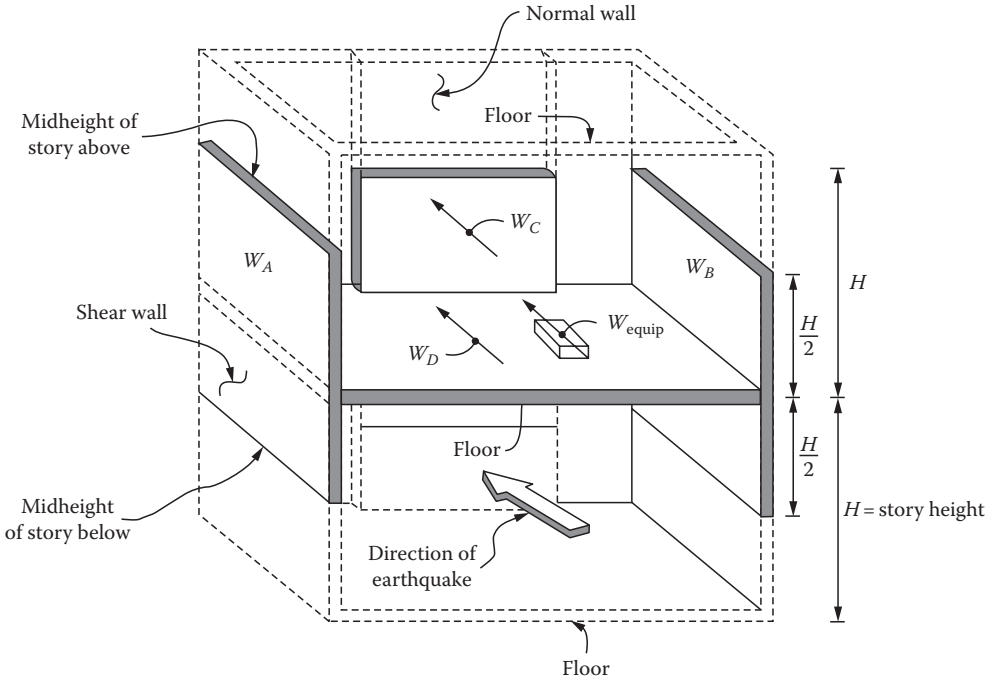
For the *equivalent lateral force procedure*, the base shear formula is:

$$V = C_s W$$

where

V is the base shear equal to the *strength level* horizontal seismic force acting at the base of the structure
W is the weight of structure that is assumed to contribute to the development of seismic forces

See Figure 6.53 for schematics of tributary weights contributing to the seismic dead load calculation. For most structures, this weight is simply taken as the dead load. However, in structures where a large percentage of the live load is likely to be present at any given time, it is reasonable to include at least a portion of this live load in the value of *W*. ASCE 7-05 lists four specific items that are to be included in the weight of the structure, *W*. For example, in storage warehouse, *W* is to include at



Story weight for calculation of lateral forces

W_x = walls + floor + equipment
$= W_A + W_B + W_C + W_D + W_{equip}$
Weight for design of diaphragm W_{Px} = normal walls + floor + equipment
$= W_C + W_D + W_{equip}$
Note: Floor weight W_D includes floor structure, suspended ceiling, mechanical equipment, and an allowance for partitions.

FIGURE 6.53 Tributory weights for seismic dead load calculations.

least 25% of the floor live load. In offices and other buildings where the locations of partitions are subject to relocation, ASCE 7 requires that floors be designed for a live load of not less than 15 psf. However, this 15 psf value is to account for localized partition loads, and it is intended to be used only for gravity load design. For seismic design, it is recognized that the 15 psf loading does not occur at all locations at the same time. Consequently, an average floor load of 10 psf may be used for the weight of partitions in determining W for seismic design.

Roof live loads need not be included in the calculation of W, but 20% of the snow load shall be included if it exceeds 30 psf.

The formulas for base shear V may be rewritten in a compact form as follows:

$$V = C_s W$$

$$C_s = \frac{S_{DS} I}{R} = \begin{pmatrix} \le \dfrac{S_{D1} I}{RT} & \text{for } T \le T_L \\ \le \dfrac{S_{D1} T_L I}{RT^2} & \text{for } T > T_L \\ = 0.044\, S_{DS} & I \ge 0.01 \\ \ge \dfrac{0.5 S_1 I}{R} & \text{for } S_1 \ge 0.6g \end{pmatrix}$$

where

S_{DS} is the short-period design spectral response acceleration
S_{D1} is the 1 s design spectral response acceleration
R is the response modification factor
I is the importance factor (*Note*: I_e in ASCE 7-10)
T is the building period
S_1 is the mapped 1 s spectral acceleration
T_L is the transition period

As mentioned earlier, the design spectral response accelerations S_{DS} and S_{D1} are the primary variables defining the design response spectrum. The maps for these are printed in ASCE 7 Figures 22-1 through 22-14. It is more practical, however, to access spectral response accelerations through web-based information.

ASCE 7 permits the maximum value of S_S used in calculating C_s to be $1.5g$ for regular structures of five stories or less and having a period of 0.5 s or less.

The variable S_S and S_1 are converted to maximum considered spectral response accelerations, S_{MS} and S_{M1}, by multiplying by site coefficients F_a and F_v, defined in ASCE 7 tables. F_a and F_v are a function of Site Classes A through F. Site classes can be assigned as a function of three different soil parameters: shear wave velocity, penetration resistance, or undrained shear strength. Most building designers would need input from a geotechnical engineer in order to determine site class. Site Class D is commonly assumed, provided that the building site is known to not have deep soft soils.

As a next step, S_{MS} and S_{M1} are multiplied by 2/3 to convert from maximum considered spectral response accelerations to design spectral response accelerations. These are designated as S_{DS} and S_{D1}, respectively.

Given a set of values for S_{DS} and S_{D1}, it is quite simple as evidenced in Figure 6.54, to construct a generic response spectrum.

Shown in Figures 6.55 through 6.57 are response spectrum for specific locations in Los Angeles, CA, Boston, MA, and Seattle, WA. Default Site Class D is used in determining the site coefficients F_a and F_v.

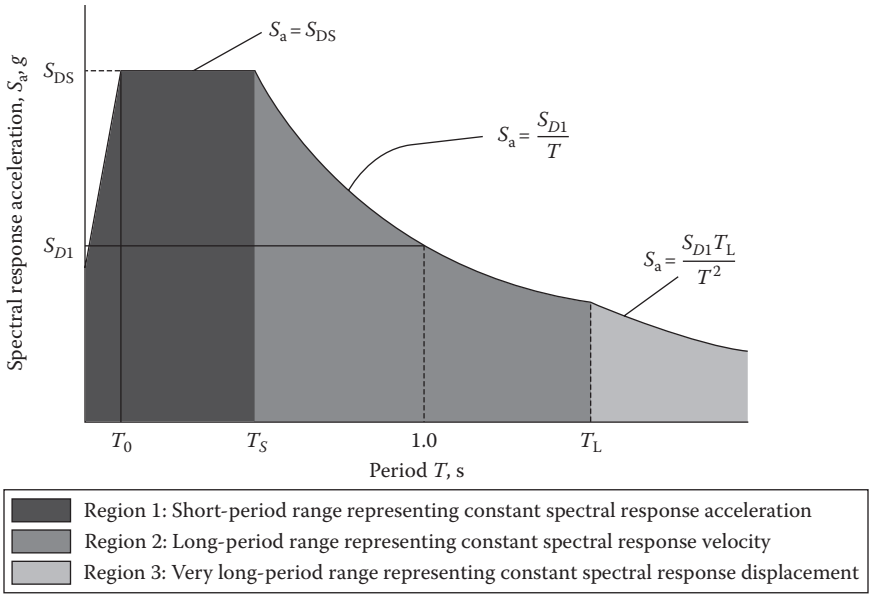


FIGURE 6.54 Generic design response spectrum.

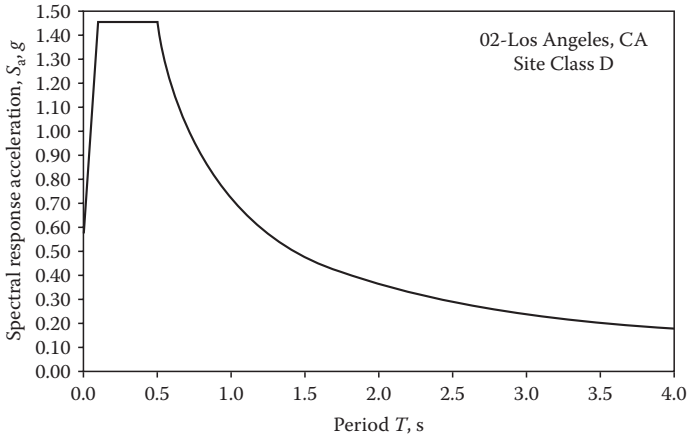


FIGURE 6.55 Response spectrum for a specific site in Los Angeles, CA, latitude 34°3′N, longitude 118°14′W, Site Class D.

The response spectra of Figures 6.54 through 6.57A show the spectral accelerations for buildings of varying periods. The level plateau, defined by S_{DS} can be considered to apply to stiffer buildings. For periods higher than T_s, the downward trend of the curve shows that as buildings become more flexible, they tend to experience lower seismic forces. On the other hand, the more flexible buildings will experience greater deformations and therefore damage to finishes and contents could become an issue.

6.3.5 Site-Specific Ground Motion Analysis

The objective in conducting a site-specific ground motion analysis is to develop ground motions that are determined with higher confidence for the local seismic and site conditions than can be determined from national ground motion maps and the general procedure. Accordingly, such studies must be comprehensive and incorporate current scientific interpretations. Because there is typically more than one scientifically credible alternative for models and parameter values used to characterize

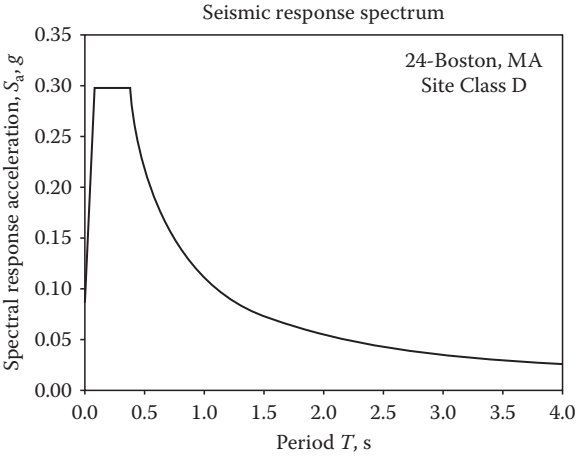


FIGURE 6.56 Response spectrum for a specific site in Boston, MA, latitude 42°22′N, longitude 71°2′W, Site Class D.

seismic sources and ground motions, it is important to formally incorporate these uncertainties in a site-specific probabilistic analysis. For example, uncertainties may exist in seismic source location, its extent and geometry; maximum earthquake recurrence rate; choices for ground motion attenuation relationships; and local site conditions including soil layering and dynamic soil properties as well as possible 2D or 3D dimensional wave propagation effects. The use of peer review for a site-specific ground motion analysis is encouraged.

Near-fault effects on horizontal response spectra include (1) directivity effects that increase ground motions for periods of vibration greater than approximately 0.5 s for fault rupture propagating toward the site; and (2) directionality effects that increase ground motions for periods greater than approximately 0.5 s in the direction normal (perpendicular) to the strike of the fault.

Observe that ASCE 7-05 requires that site-specific geotechnical investigations and dynamic site response analysis be preformed for sites having Site Class F soils.

For purposes of obtaining data to conduct a site response analysis, site-specific geotechnical investigations should include borings with sampling, standard penetration tests (SPT) for sandy soils, cone penetrometer test (CPT) and/or other subsurface investigative techniques and laboratory soil testing to establish the soil types, properties and layering and the depth to rock or rock-like material. For very deep soil sites, the depth of investigation need not necessarily extend to bedrock but to a depth that may serve as the location of input motion for a dynamic site response analysis. It is desirable to measure shear wave velocities in all soil layers. Alternatively, shear wave velocities may be estimated based on shear wave velocity date available for similar soils in the local area or through correlations with soil types and properties.

Development of a site-specific design response spectrum requires reviewing seismological and geological data and performing engineering analyses. ASCE 7-05 allows development of a site-specific design spectrum using probabilistic seismic hazard and site response analyses. However, the design spectrum for structural analysis may not be less than 80% of the general response spectrum developed using Section 11.4 of ASCE 7-05.

Development of the site-specific spectra typically involves the following steps:

1. Select previously recorded time histories of acceleration and scale the records to match the estimate bedrock motions.
2. Perform site response analyses to estimate the response of the site soils to the bedrock motions using a probabilistic seismic hazard analysis (PSHA). PSHA considers all

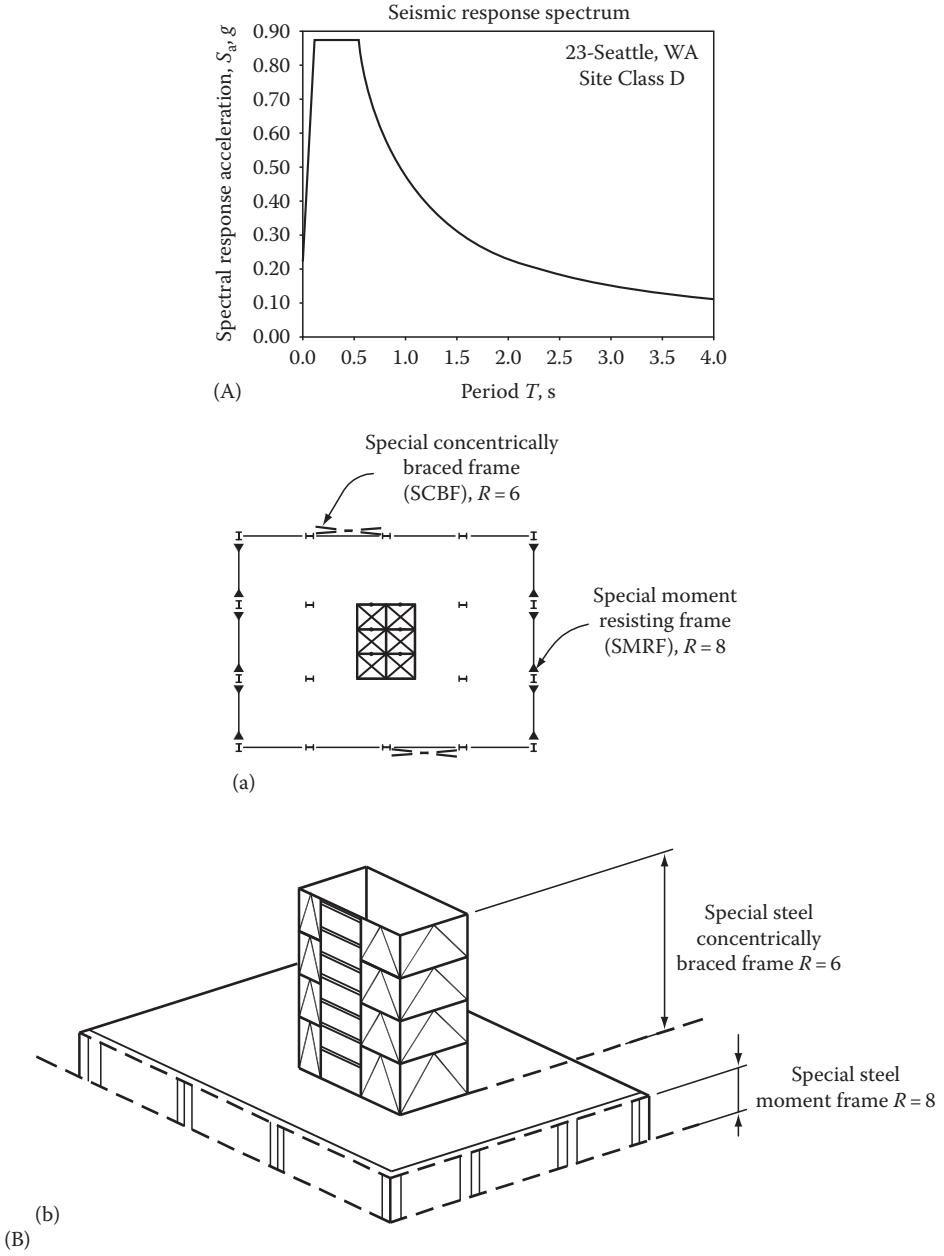


FIGURE 6.57 (A) Response spectrum for a specific site in Seattle, WA, latitude 47°39′N, longitude 122°18′W, Site Class D. (B) (a) Different systems along two orthogonal axes; use corresponding *R* value for each system. (b) Different systems over the building's height. Response modification coefficient, *R*, for any story above, shall not exceed the lowest value, in the direction under consideration.

potential earthquake sources that may contribute to strong ground shaking at a specific site. Magnitude, distance, and probability of occurrence are all factored into the computation. Typically PSHA data generated by the USGS is used for the purpose. Next, rock outcrop motions (i.e., spectral ordinates) are selected for a probability of exceedance of 2% in 50 years and multiplied the values by 2/3 per the ASCE 7-05.

6.3.6 Importance Factor I_E

The purpose of this factor, I_E (referred to as I without the subscript "E" in ASCE 7-05), is to improve the capability of certain types of buildings such as essential facilities and structures containing substantial quantities of hazardous materials, to function during and after design earthquakes. This is achieved by introducing an importance factor of 1.25 for occupancy category III structures and 1.5 for Category IV structures. This factor is intended to reduce the ductility demands and result in less damage. When combined with the more stringent drift limits, the result is improved performance of such facilities.

Shown in Tables 6.3 and 6.4 are the importance factors I_E for seismic, I_{w1} for wind in non-hurricane regions and I_{w2} for wind in hurricane-prone regions.

A value of I greater than unity has the effect of reducing the ductility expected of a structure. However, added strength due to higher design forces by itself is not sufficient to ensure superior seismic performance. Connection details that assure ductility, quality assurance procedures and limitations on building deformation are also important to improve the functionality and safety in critical facilities and those with high-density occupancy. Consequently, the reduction in the damage potential of critical facilities is also addressed by using more conservative drift controls and by providing special design and detailing requirements and materials limitations. The assignment of the importance factor, I_E, is not the sole responsibility of the structural engineer. It is a decision to be made concurrently with the building owners, architect and the building official.

6.3.7 Occupancy Categories

The expected performance of structures is controlled by assignment of each structure to one of four occupancy categories. The ASCE Provisions specify progressively more conservative strength, drift control, system selection and detailing requirements for structures assigned to higher occupancy groups. The purpose is to attain minimum levels of earthquake performance deemed suitable to the individual occupancies. In terms of post-earthquake recovery, certain types of occupancies are vital to public needs. These special occupancies are identified in ASCE 7-05 and given specific recognition. In terms of disaster preparedness, regional communication centers identified as critical emergency services are placed in a higher classification than retail stores, office buildings, and factories.

Specific consideration is given to essential facilities required for post-earthquake recovery. Also included are structures that contain substances deemed to be hazardous to the public. It is at the discretion of the authority having jurisdiction to identify which structures are required for post-earthquake response and recovery.

Although the AISC Provisions explicitly require design for only a single level of ground motion, it is expected that structures designed and constructed in accordance with the provisions will generally be able to meet a number of performance criteria, when subjected to earthquake ground motions of differing severity. Occupancy category I, II, or III structures located where the mapped spectral response acceleration parameter at 1 s period, S_1, is greater than or equal to 0.75 shall be assigned to SDC E. Occupancy category IV structures located where the mapped spectral response acceleration parameter at 1 s period, S_1, is greater than or equal to 0.75 shall be assigned to SDC F. All other structures shall be assigned to a SDC based on their occupancy category and the design spectral response acceleration parameters, S_{DS} and S_{D1}. Each building and structure shall be assigned to the more adverse SDC, irrespective of the fundamental period of vibration of the structure, T.

6.3.7.1 Protected Access for Occupancy Category IV

Where operational access to an occupancy category IV structure is required through an adjacent structure, the adjacent structure shall conform to the requirements for occupancy category IV structures. Where operational access is less than 10 ft from an interior lot line or another structure on the same lot, protection from potential falling debris from adjacent structures shall be provided by the owner of the occupancy category IV structure.

TABLE 6.3
Occupancy Category of Buildings and Importance Factors

Nature of Occupancy	Occupancy Category	Importance Factor I_E	I_{W1}	I_{W2}
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I	1.0	0.87	0.77
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II	1.0	1.0	1.0
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with day care facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities • Jails and detention facilities	III	1.25	1.15	1.15
Buildings and other structures, not included in occupancy category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to • Power-generating stations[a] • Water-treatment facilities • Sewage-treatment facilities • Telecommunication centers				
Buildings and other structures not included in occupancy category IV (including but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released				
Buildings and other structures containing toxic or explosive substances shall be eligible for classification as occupancy category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the toxic or explosive substances does not pose a threat to the public				
Buildings and other structures designated as essential facilities, including, but not limited to	IV	1.5	1.15	1.15

(*continued*)

TABLE 6.3 (continued)
Occupancy Category of Buildings and Importance Factors

Nature of Occupancy	Occupancy Category	Importance Factor I_E	I_{W1}	I_{W2}
• Hospitals and other health care facilities having surgery or emergency treatment facilities				
• Fire, rescue, ambulance, and police stations and emergency vehicle garages				
• Designated earthquake, hurricane, or other emergency shelters				
• Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response				
• Power-generating stations and other public utility facilities required in an emergency				
• Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for the operation of occupancy category IV structures during an emergency				
• Aviation control towers, air traffic control centers, and emergency aircraft hangars				
• Water storage facilities and pump structures required to maintain water pressure for fire suppression				
Buildings and other structures having critical national defense functions				
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction				
Buildings and other structures containing highly toxic substances shall be eligible for classification as occupancy category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.				

Source: From ASCE 7-05, Table 1.1.

Note: I_E, seismic importance factor; I_{W1}, wind importance factor, non-hurricane prone regions and hurricane prone regions with V = 85–100 mph, and Alaska; I_{W2}, wind importance factor, hurricane prone regions with $V >$ 100 mph.

[a] Cogeneration power plants that do not supply power on the national grid shall be designated occupancy category II.

TABLE 6.4
Importance Factors

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

Source: ASCE 7-05, Table 11.5.1.

Note: The importance factor reflects the relative importance assigned to a building based on its occupancy category. In a conceptual sense, it indicates the performance expected of the building during and after an earthquake. In an attempt to achieve the desired performance, the design-based shear is increased 25% for those in occupancy category III, and 50% for those in occupancy category IV.

The value for occupancy category III buildings, which includes buildings with an occupancy greater than 5000 and college buildings with a capacity greater than 500 students, is now 1.25, while in previous codes it has been 1.0. There is new emphasis in attempting to control the amount of ductility demand for these occupancies.

6.3.8 Seismic Design Category

The predicted earthquake performance of a building is based upon system performance, and considerable energy dissipation through repeated cycles of inelastic straining is assumed. The reason for dependence on inelastic excursions is entirely due to economics. The associated cost of providing enough strength to maintain linear elastic response in ordinary buildings is simply too much. However, structures that include facilities critical post-earthquake operations—such as hospitals, fire stations, and communication centers—must not only survive without collapse, but must also remain operational after an earthquake. Therefore, in addition to life safety, damage control is also a design consideration for structures deemed vital to post-earthquake function. The current requirements of achieving this goal are to increase the magnitude of design forces by a factor of 1.25 or 1.5 depending upon the nature of occupancy. For certain structures such as nuclear facilities, yielding cannot be tolerated and as such, the design needs to be based on forces determined by elastic analysis.

ASCE 7 establishes five SDCs that are the keys for establishing design requirements for any building. The SDC A, B, C, D, E, or F is established, using the short period, and 1 s period response parameters, S_{DS} and S_{D1}, and the occupancy category.

SDC A represents structures in regions where anticipated ground motions are minor, even for very long return periods. For such structures, ASCE 7 requires only that a complete lateral-force-resisting system be provided and that all elements of the structure be tied together. A nominal design base shear equal to 1% of the weight of the structure is used to proportion the lateral system.

SDC B includes structures in regions of seismicity where only moderately destructive ground shaking is anticipated. In addition to the requirements for SDC A, structures in SDC B must be designed for forces determined using ASCE 7-05 Seismic Maps.

SDC C includes buildings in regions where moderately destructive ground shaking may occur. The use of some structural systems is limited and certain nonstructural components must be specifically designed for seismic resistance.

SDC D includes structures located in regions expected to experience destructive ground shaking, but not located very near major active faults. In SDC D, severe limits are placed on the use of some

TABLE 6.5
SDC Based on Short-Period Response Acceleration Parameter

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \le S_{DS} < 0.33$	B	B	C
$0.33 \le S_{DS} < 0.50$	C	C	D
$0.50 \le S_{DS}$	D	D	D

Source: ASCE 7-05, Table 11.6.1.

TABLE 6.6
SDC Based on 1 s-Period Response Acceleration Parameter

Value of S_{D1}	Occupancy Category		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \le S_{D1} < 0.133$	B	B	C
$0.133 \le S_{D1} < 0.20$	C	C	D
$0.20 \le S_{D1}$	D	D	D

Source: ASCE 7-05, Table 11.6.2.

structural systems and irregular structures must be subjected to dynamic analysis techniques as part of the design process.

SDC E includes structures in regions located very close to major active faults and SDC F includes occupancy category IV structures in these locations. Very severe limitations on systems, irregularities and design methods are specified for SDC E and F. For the purpose of determining if a structure is located in a region that is very close to a major active fault, ASCE 7 uses a trigger of mapped MCE spectral response acceleration at 1 s periods, S_1 of $0.75g$ or more regardless of the structure's fundamental period. The mapped short-period acceleration S_s is not used for this purpose because short-period response accelerations do not tend to be affected by near-source conditions as strongly as do response accelerations at long periods.

See Tables 6.5 through 6.8 for SDCs. It should be noted that each building shall be assigned to the more severe SDC in accordance with Tables 6.5 through 6.8, irrespective of the fundamental period of vibration, T, of the building.

6.3.9 Design Requirements for SDC A Buildings

SDC A represents structures in regions where anticipated ground motions are minor, even for very long return periods. For such structures, ASCE 7-05 requires only that a complete seismic-force-resisting system be provided and all elements of the structure be tied together. A nominal design force equal to 1% of the weight of the structure is used to proportion the lateral system.

It is not considered necessary to specify seismic-resistant design on the basis of a maximum considered earthquake ground motion for SDC A structures. However, it is desirable to provide some protection against earthquakes and many other types of unanticipated loadings. Thus, the

TABLE 6.7
SDC Based on S_{D1} and S_1

Nature of Occupancy (Typical Examples)	Occupancy Category	Importance Factors	S_{D1}				S_1
			$S_{D1} < 0.067$	$0.067 \le S_{D1}$ <0.133	$0.133 \le S_{D1}$ <0.20	$0.20 \le S_{D1}$	$S_1 \ge 0.75$
Agricultural, temporary facilities and minor storage facilities	I	$I_W = 0.83$ or 0.77 $I_E = 1.0$	A	B	C	D	E
Typical residential and office buildings	II	$I_W = 1.0$ $I_E = 1.0$	A	B	C	D	E
Schools, colleges, fire and police stations, detention facilities, and buildings containing toxic substances	III	$I_W = 1.15$ $I_E = 1.25$	A	B	C	D	E
Hospitals, health care facilities, and designated emergency shelters	IV	$I_W = 1.15$ $I_E = 1.50$	A	C	D	D	F

TABLE 6.8
SDC Based on S_{DS} and S_1

Nature of Occupancy (Typical Examples)	Occupancy Category	Importance Factors	S_{DS}				S_1
			$S_{DS} < 0.167$	$0.167 \le S_{DS}$ <0.33	$0.33 \le S_{DS}$ <0.5	$0.5 \le S_{DS}$	$S_1 \ge 0.75$
Agricultural, temporary facilities and minor storage facilities	I	$I_W = 0.87$ or 0.77 $I_E = 1.00$	A	B	C	D	E
Typical residential and office buildings	II	$I_W = 1.00$ $I_E = 1.00$	A	B	C	D	E
Schools, colleges, fire and police stations, detention facilities, and buildings containing toxic substances	III	$I_W = 1.15$ $I_E = 1.25$	A	B	C	D	E
Hospitals, health care facilities, and designated emergency shelters	IV	$I_W = 1.15$ $I_E = 1.50$	A	C	D	D	F

requirements for SDC A provide a nominal amount of structural integrity that will improve the performance of buildings in the event of a possible but rare earthquake even though it is possible that the ground motions could be large enough to cause serious damage or even collapse. The result of design to SDC A requirements is that fewer buildings would collapse in the vicinity of such an earthquake.

Structural integrity is provided by a combination of requirements. First, a complete load path for lateral forces must be designed for a lateral force based on a 1% acceleration of the mass. The minimum connection forces specified for SDC A also must be satisfied.

It is of interest to note that the 1% value has been used in other countries as a minimum value for structural integrity. For many structures, design for the wind loadings specified in the local building codes normally will control the lateral force design when compared to the minimum integrity force on the structure. However, many low-rise, heavy structures or structures with significant dead loads resulting from heavy equipment may be controlled by the nominal 1% acceleration. Also, minimum connection forces may exceed structural forces due to wind in some structures.

As noted previously, the minimum lateral force for SDC A structures is a structural integrity issue related to the load path. It is intended to specify design forces in excess of wind loads in heavy low-rise construction. The design calculation is simple and easily done to ascertain if it governs or the wind load governs. The ASCE 7-05 provision requires a nominal lateral force, F_x, equal to 1% of the gravity load assigned to a story to assure general structural integrity.

The complete list of seismic design requirements are as follows:

- Analyze the structure for the effect of static lateral forces, F_x, applied independently in each of the two orthogonal directions.
- Apply the static forces, F_x, at all levels simultaneously.
- Determine the force, F_x, at each level by using the following equation:

$$F_x = 0.01W_x$$

  where
  F_x is the design lateral force applied at level x
  W_x is the portion of the dead load, D, located or assigned to level x

- Connect all parts of the structure to form a continuous load path to the lateral-force-resisting system.
- Tie any smaller portion of the structure to the remainder of the structure with connections capable of transmitting at least 5% of the portion's weight.
- Provide a positive connection for resisting a horizontal force acting parallel to each horizontal member such as slab, beam, girder, or truss. The connection shall be adequate to transfer a minimum horizontal force equal to 5% of the dead plus live load vertical reaction.
- Anchor concrete walls at the roof and all floors. Design the anchors for a horizontal force equal to 5% of the wall weight, but not less than 280 lb per linear foot.

Because of the very low seismicity associated with sites with S_{DS} less than 0.25 and S_{D1} less than 0.10, it is considered appropriate for Category A buildings to require only a complete seismic-force-resisting system, good quality of construction materials and adequate ties and anchorage as specified in this section. Category A buildings will be constructed in a large portion of the United States that is generally subject to strong winds but low earthquake risk. Those promulgating construction regulations for these areas may wish to consider many of the low-level seismic requirements as being suitable to reduce the windstorm risk. Since the seismic provisions consider only earthquakes, no other requirements are prescribed for Category A buildings. Only a complete seismic-force-resisting system and wall anchorage are required.

Construction qualifying under Category A may be built with no special detailing requirements for earthquake resistance. Special details for ductility and toughness are not required in Category A.

6.3.9.1 Lateral Forces

SDC A buildings as stated earlier, are not designed for resistance to any specific level of earthquake ground shaking as the probability that they would ever experience shaking of sufficient intensity to cause life-threatening damage is very low so long as the structures are designed with basic levels of structural integrity. Minimum levels of structural integrity are achieved in a structure by assuring

that all elements in the structure are tied together so that the structure can respond to shaking demands in an integral manner and also by providing the structure with a complete seismic-force-resisting system. It is believed that structures having this level of integrity would be able to resist, without collapse, the very infrequent earthquake ground shaking that could affect them. In addition, requirements to provide such integrity provide collateral benefit with regard to the ability of the structure to survive other hazards such as high windstorms, tornadoes, and hurricanes.

The procedure outlined for Category A buildings is intended to be a simple approach to ensuring both that a building has a complete seismic-force-resisting system and that it is capable of sustaining at least a minimum level of lateral force. In this analysis procedure, a series of static lateral forces equal to 1% of the weight at each level of the structure is applied to the structure independently in each of two orthogonal directions. The structural elements of the seismic-force-resisting system then are designed to resist the resulting forces in combination with other loads under the load combinations specified by the building code.

The selection of 1% of the building weight as the design force for SDC A structures is somewhat arbitrary. This level of design lateral force is consistent with prudent requirements for lateral bracing of structures to prevent inadvertent buckling under gravity loads. It is also sufficiently small as to not present an undue burden on the design of structures in zones of very low seismic activity.

The seismic weight W is the total weight of the building and that part of the service load that might reasonably be expected to be attached to the building at the time of an earthquake. It includes permanent and movable partitions and permanent equipment such as mechanical and electrical equipment, piping, and ceilings. The normal human live load is taken to be negligibly small in its contribution to the seismic lateral forces. Buildings designed for storage or warehouse usage should have at least 25% of the design floor live load included in the weight, W. Snow loads up to 30 psf are not considered. Freshly fallen snow would have little effect on the lateral force in an earthquake; however, ice loading would be more or less firmly attached to the roof of the building and would contribute significantly to the inertia force. For this reason, the effective snow load is taken as the full snow load for those regions where the snow load exceeds 30 psf with the proviso that the local authority having jurisdiction may allow the snow load to be reduced up to 80%. The question of how much snow load should be included in W is really a question of how much ice buildup or snow entrapment can be expected for the roof configuration or site topography, and this is a question best left to the discretion of the local authority having jurisdiction.

6.3.10 Geologic Hazards and Geotechnical Investigation

This section, given at the very end of Chapter 11, ASCE 7-05, applies to buildings assigned to SDC C through F. It addresses limitations for siting buildings, and requirements of geotechnical investigation report.

Siting of a structure assigned to SDC E or F is not permitted where there is a known potential for an active fault to case rupture of the ground surface at the structure.

For SDC C through D buildings, a geotechnical investigation report is required. The report shall include an evaluation of the following potential geologic and seismic hazards:

1. Slope instability
2. Liquefaction
3. Differential settlement
4. Surface displacement due to faulting or lateral spreading

The report shall contain recommendations for appropriate foundation designs or other measures to mitigate the effects of the previously mentioned hazards. Where deemed appropriate by the authority having jurisdiction, a site-specific geotechnical report is not required where prior evaluations of nearby sites with similar soil conditions provide sufficient direction relative to the proposed construction.

Additional geotechnical investigation report requirements for SDCs D through F are:

1. The determination of lateral pressures of basement and retaining walls due to earthquake motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes, and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration is permitted to be determined based on a site-specific study taking into account soil amplification effects or, in the absence of such a study, peak ground accelerations shall be assumed equal to $S_s/2.5$.
3. Assessment of potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures and how they shall be considered in the design of the structure.

6.3.10.1 Seismic Design Basis

The first and foremost design requirement is to provide complete lateral and vertical-force-resisting systems capable of providing adequate strength, stiffness and energy-dissipation capacity to withstand design ground motions within the prescribed limits of deformation and strength demand. The design ground motions shall be assumed to occur along any horizontal direction of a building structure. The adequacy of the structural system shall be demonstrated through analysis of a mathematical model for the effects of design ground motions.

6.3.10.1.1 Member Design, Connection Design, and Deformation Limit

Individual members, including those not part of the seismic-force-resisting system, shall be provided with adequate strength to resist the shears, axial forces, and moments. Connections shall develop the strength of the connected members or the forces determined by the analysis. The deformation of the structure shall not exceed prescribed limits when subjected to design seismic forces.

6.3.10.1.2 Continuous Load Path and Interconnection

A continuous load path, or preferably load paths, with adequate strength and stiffness shall be provided to transfer all forces from the point of application to the final point of resistance. All parts of the structure shall be interconnected to provide a continuous path to the seismic-force-resisting system, and the connections shall be capable of transmitting the seismic force, F_p, induced by the parts being connected. Any smaller portion of the structure shall be tied to the remainder of the structure with elements having a design strength capable of transmitting a seismic force of 0.133 times the short-period or 5% of the portion's weight, whichever is greater. Connection design forces need not exceed the maximum forces that the structural system can deliver to the connection.

6.3.10.1.3 Connection to Supports

A positive connection for resisting a horizontal force acting parallel to the member for each beam, girder or truss shall be provided either directly to its supporting elements or to slabs designed to act as diaphragms. Where the connection is through a diaphragm, then the member's supporting element must also be connected to the diaphragm. The connection shall have minimum design strength of 5% of the dead plus live load reaction.

6.3.10.2 Structural System Selection

Select a system that conforms to one of the types shown in ASCE 7-05 Table 12.2.1. If you want to use a system that is not in the table, you may do so, but the catch is you are required to backup the seismic performance of your system with analytical and test data that establish its energy-dissipation capacity and dynamic performance. The seismic coefficients, R, Ω_o, and C_d, used for your system should be the same as for the system being duplicated.

Note that all systems indicated in the table may be used for SDC B buildings without height limits, except for cantilevered column systems that have a height limit of 35 ft.

6.3.10.2.1 Combinations of Framing Systems along Two Orthogonal Axes

Different seismic-force-resisting systems are permitted to be used to resist seismic forces along each of the two orthogonal axes of the structure. Where different systems are used, the respective R, C_d, and Ω_o coefficients shall apply to each system. See Figure 6.57B(a).

6.3.10.2.2 Combinations of Framing Systems in the Same Direction

Where different seismic-force-resisting systems are used in combination to resist seismic forces in the same direction of structural response, other than those combinations considered as dual systems, the more stringent system limitations shall apply.

6.3.10.2.3 R, C_d and Ω_o Values for Vertical Combinations

The value of response modification coefficient, R, used for design at any story shall not exceed the lowest value of R that is used in the same direction at any story above that story (see Figure 6.57B(a)). Likewise, the deflection amplification factor, C_d, and the system overstrength factor, Ω_o, used for the design at any story shall not be less than the largest value of this factor that is used in the same direction at any story above that story.

A two-stage equivalent lateral-force procedure is permitted to be used with structures having a flexible upper portion above a rigid lower portion, provided that the design of the structure complies with the following:

1. The stiffness of the lower portion must be at least 10 times the stiffness of the upper portion.
2. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.
3. The flexible upper portion shall be designed as a separate structure using the appropriate values of R and p.
4. The rigid lower portion shall be designed as a separate structure using the appropriate values of R and p. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the upper portion over R/p of the lower portion. This ratio shall not be less than 1.0.

6.3.10.2.4 R, C_d and Ω_o Values for Horizontal Combinations

Where combination of different structural systems is used to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value of any of the systems utilized in that direction. Resisting elements are permitted to be designed using the least value of R for the different structural systems found in each independent line of resistance if the following three conditions are met: (1) occupancy category I or II building, (2) two stories or less in height, and (3) the use of light-frame construction or flexible diaphragms. The value of R used for the design of diaphragms in such structures shall not be greater than the least value for any of the systems utilized in the direction.

The deflection amplification factor, C_d, and the overstrength factor, Ω_o, in the direction under consideration at any story shall not be less than the largest values of these factors used in the same direction.

6.3.10.2.5 Combination Framing Detailing Requirements

Structural components common to different framing systems used to resist seismic motions in any direction shall be designed using the detailing requirements required by the highest modification coefficient, *R*, of the connected framing systems.

6.3.10.2.6 System-Specific Requirements

The structural framing shall also comply with the following system-specific requirements of this section:

- *Dual system*. For a dual system, the moment frames shall be capable of resisting at least 25% of the design forces. The total seismic force resistance is to be provided by the combination of the moment frames and braced frames in proportion to the rigidities.
- *Cantilever column systems*. The load on individual cantilever column elements calculated in accordance with the applicable load combinations shall not exceed 15% of the design strength of the column to resist axial loads alone. The foundation and other elements used to provide overturning resistance at the base of cantilever column elements shall have the strength to resist the load combinations with prescribed overstrength factor Ω_o.
- *Inverted pendulum-type structures*. Supporting columns of inverted pendulum-type structures shall be designed for bending moment calculated at the base determine using the equivalent lateral force (ELF) procedure and varying uniformly to a moment at the top equal to one-half the calculated bending moment at the base.

6.3.11 Building Irregularities

Prior to the 1988 UBC, building codes published a list of irregularities defining the conditions, but provided no quantitative basis for determining the relative significance of a given irregularity. However, starting in 1988, seismic codes have attempted to quantify irregularities by establishing geometrically or by use of building dimensions, the points at which the specific irregularity becomes an issue as to require extra analysis and design considerations. The code requirements for determining the presence of plan and vertical irregularity, and the required methods to compensate for it, have now become complex, as can be seen in Tables 6.9 and 6.10, and their graphic interpretation in Tables 6.11 and 6.12. Observe that the remedial measures range from a simple requirement of a dynamic distribution of lateral forces (e.g., mass irregularity), to special load combination of gravity and seismic forces (e.g., out-of-plane offset irregularity).

6.3.11.1 Plan (Horizontal) Irregularity

Tables 6.9 and 6.11 indicate under what circumstances a building must be designed as having a plan irregularity. A building may have a symmetrical geometric shape without reentrant corners or wings but still be classified as irregular in plan because of distribution of mass or vertical-seismic-force-resisting elements. Torsional effects in earthquakes can occur even when the centers of mass and resistance coincide. For example, ground motion waves acting at a skew with respect to the building axis can cause torsion. Cracking or yielding in nonsymmetrical elements also can cause torsion. These effects also can magnify the torsion due to eccentricity between the static centers. For this reason, buildings having an eccentric dimension perpendicular to the direction of the seismic force should be classified as irregular. The vertical-resisting components may be arranged so that the centers of mass and resistance are within the limitations given above and still be asymmetrically arranged so the prescribed torsional forces would be unequally distributed to the various components. Torsional irregularities are subdivided into two categories: non-extreme, and extreme irregularity. Extreme torsional irregularities are prohibited for structures located very close to major active faults and should be avoided when possible, in all structures.

TABLE 6.9
Horizontal Structural Irregularities (Plan Irregularities)

Irregularity Type and Description	Reference Section of ASCE 7-05	SDC Application
1a. **Torsional irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4	D through F
	12.8.4.3	C through F
	12.7.3	B through F
	12.12.1	C through F
	Table 12.6-1	D through F
	16.2.2	B through F
1b. **Extreme torsional irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1	E and F
	12.3.3.4	D
	12.7.3	B through D
	12.8.4.3	C and D
	12.12.1	C and D
	Table 12.6-1	D
	16.2.2	B through D
2. **Reentrant corner irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4	D through F
	Table 12.6-1	D through F
3. **Diaphragm discontinuity irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4	D through F
	Table 12.6-1	D through F
4. **Out-of-plane offsets irregularity** is defined to exist where there are discontinuities in a lateral force–resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4	D through F
	12.3.3.3	B through F
	12.7.3	B through F
	Table 12.6-1	D through F
	16.2.2	B through F
5. **Nonparallel systems irregularity** is defined to exist where the vertical lateral-force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic-force-resisting system.	12.5.3	C through F
	12.7.3	B through F
	Table 12.6-1	D through F
	16.2.2	B through F

Source: ASCE 7-05, Table 12.3-1.

There is a second type of distribution of vertical components that, while not being classified as irregular, does not perform well in earthquakes. This occurs in a core-type building with the seismic-force-resisting system concentrated near the center of the building. Better performance has been observed when the vertical components are distributed near the perimeter of the building. In recognition of this, a torsional amplification factor is prescribed in the ASCE Provisions.

A building have a regular configuration can be square, rectangular, or circular. A square or rectangular building with minor reentrant corners would still be considered regular but large reentrant corners creating a crucifix form would be classified as an irregular configuration. The response of the wings of this type of building is generally different from the response of the building as a whole, and this produces higher local forces than would be determined by analysis. Other plan configurations such as H-shapes that have a geometrical symmetry also would be classified as irregular because of the response of the wings.

TABLE 6.10
Vertical Structural Irregularities

Irregularity Type and Description	Reference Section of ASCE 7-05	SDC Application
1a. **Stiffness soft story irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D through F
1b. **Stiffness extreme soft story irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D through F
2. **Weight (mass) irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D through F
3. **Vertical geometric irregularity** is defined to exist where the horizontal dimension of the seismic-force-resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D through F
4. **In-plane discontinuity in vertical–lateral-force-resisting element irregularity** is defined to exist where an in-plane offset of the lateral-force-resisting elements is greater than the length of those elements or there exists a reduction in the stiffness of the resisting element in the story below.	12.3.3.3 12.3.3.4 Table 12.6-1	B through F D through F D through F
5a. **Discontinuity in lateral strength–weak story irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D through F
5b. **Discontinuity in lateral strength–extreme weak story irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D through F B and C D through F

Source: ASCE 7-05, Table 12.3-2.

Significant differences in stiffness between portions of a diaphragm are classified as irregularities since they may cause a change in the distribution of seismic forces to the vertical components and create torsional forces not accounted for in the normal distribution considered for a regular building.

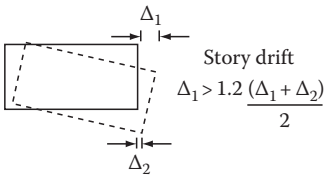
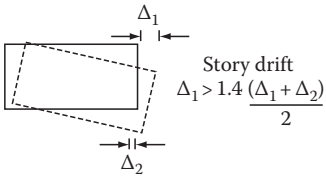
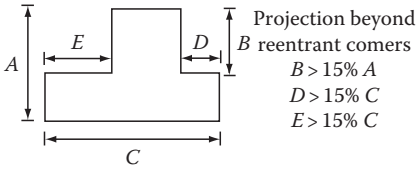
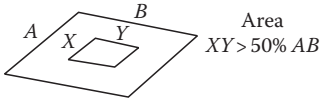
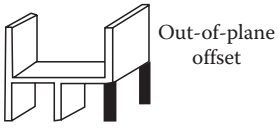
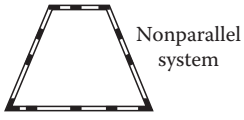
Where there are discontinuities in the path of lateral force resistance, the structure can no longer be considered "regular." The most critical of the discontinuities is the out-of-plane offset of vertical elements of the seismic-force-resisting elements. Such offsets impose vertical and lateral load effects on horizontal elements that are at the least, difficult to provide for adequately.

Where vertical elements of the lateral-force-resisting system are not parallel to or symmetric about major orthogonal axes, the static lateral force procedures cannot be applied and, thus, the structure must be considered "irregular."

6.3.11.2 Vertical Irregularity

Tables 6.10 and 6.12 indicate under what circumstances a structure must be considered to have a vertical irregularity. Vertical irregularities affect structural response and induce loads at the irregularity levels that are significantly different from the distribution assumed in the equivalent lateral force procedure.

TABLE 6.11
Horizontal Irregularities (Plan Irregularities)

Types of Irregularity	Graphic Interpretation	Remedial Measures	SDC Application
1a. Torsional irregularity		1 2 3 4 5	D through F C through F B through F C through F D through F
1b. Extreme torsion irregularity		6 1 3 2 4 5	E and F D B through D C and D C and D D
2. Reentrant corner irregularity		1 5	D through F D through F
3. Diaphragm discontinuity irregularity		1 5	D through F D through F
4. Out-of-plane offsets irregularity		1 7 3 5	D through F B through F B through F D through F
5. Nonparallel systems irregularity		8 3 5	C through F B through F D through F

Remedial measures

1. Increase design forces determined by static procedure by 25% for connection of diaphragms to vertical elements and to collectors, and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for load combinations with overstrength factor Ω_o (ASCE 7-05, Section 12.3.3.4).

Source: ASCE 7-05, Table 12.3-1.

A building would be classified as irregular if the ratio of mass to stiffness in adjoining stories differs significantly. This might occur when a heavy mass, such as a swimming pool, is placed at one level while the floors above or below have typical floor loads. A comparative stiffness ratio between stories is given at a benchmark to exempt structures from being designed as having a vertical irregularity.

TABLE 6.12
Vertical Irregularities

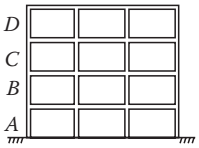
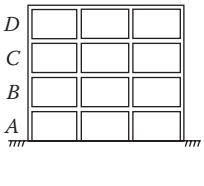
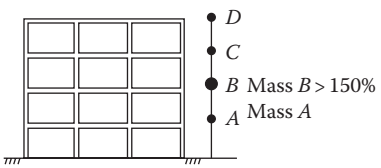
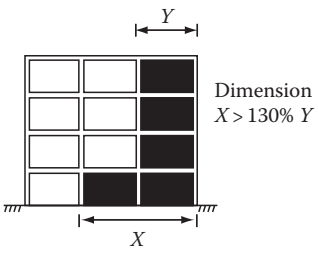
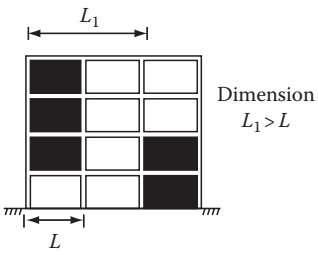
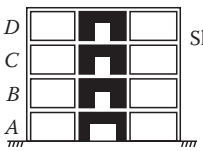
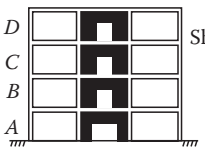
Type of Irregularity	Graphic Interpretation	Remedial Measures	SDC Application
1a. Stiffness irregularity (soft story)		5	D through F
1b. Stiffness irregularity (extreme soft story)		6 5	E and F D through F
2. Weight (mass) irregularity		5	D through F
3. Vertical geometric irregularity		5	D through F
4. In-plane discontinuity in vertical–lateral-force-resisting systems		7 1 5	B through F D through F D through F
5a. Discontinuity in lateral strength (weak story)		6 5 6	E and F D through F D through F
5b. Discontinuity in lateral strength (extreme weak story)		9 5	B and C D through F

TABLE 6.12 (continued)
Vertical Irregularities

Remedial measures

1. Increase design forces determined by static procedure by 25% for connection of diaphragms to vertical elements and to collectors, and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for load combinations with overstrength factor Ω_o (ASCE 7-05, Section 12.3.3.4).
2. Multiply, M_{ta}, the torsional moment due to accidental torsion by a torsional amplification $A_x = \left(\delta_{max.}/1.2\delta_{ave}\right)^2 \leq 3.0$ (ASCE 7-05, Section 12.8.4.3).
3. Perform a 3D dynamic analysis with due consideration for diaphragm stiffness. Use cracked section properties for concrete elements. Include PΔ effects (ASCE 7-05, Section 12.7.3).
4. Compute story drift, Δ, as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration (ASCE 7-05, Section 12.12.1).
5. Use model analysis or more rigorous procedure (ASCE 7-05, Table 12.6.1).
6. Not permitted (NP) (ASCE 7-05, Section 12.3.3.1).
7. Design columns, beams, trusses, or slabs supporting discontinuous walls or frames to resist maximum axial forces determined by using the following load combinations with the overstrength factor Ω_o:
   $(1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S$
   $(0.9 - 0.2S_{DS})D + \Omega_o Q_E + 1.6H$
   (ASCE 7-05, Section 12.3.3.3).
8. Use 100x + 30y, if you are using ELF or modal analysis. Use simultaneous application of load, if you are analyzing the structure using a linear or nonlinear response history procedure (ASCE 7-05, Section 12.5.3).
9. Maximum height limit 30 ft or two stories, unless the weak story is capable of resisting a seismic force = Ω_o times the design force (ASCE 7-05, Section 12.3.3.2).

Source: ASCE 7-05, Table 12.3-2.

Another type of vertical irregularity is created by unsymmetrical geometry with respect to the vertical axis of the building. The building may have a geometry that is symmetrical about the vertical axis and still be classified as irregular because of significant horizontal offset in the vertical elements of the lateral-force-resisting system at one or more levels. An offset is considered to be significant if the ratio of the larger dimension to the small dimension is more than 130%. The building also would be considered irregular if the smaller dimension were below the larger dimension, thereby creating an inverted pyramid effect.

Weak story irregularities occur whenever the strength of a story to resist lateral demands is significantly less than that of the story above. This is because buildings with this configuration tend to develop all of their inelastic behavior at the weak story. This can result in a significant change in the deformation pattern of the building, with most earthquake induced displacement occurring within the weak story. This can result in extensive damage within the weak story and even instability and collapse.

The soft story irregularity is subdivided into two categories: non-extreme and extreme soft story category. Like weak stories, soft stories can lead to instability and collapse. Buildings with extreme soft stories are prohibited on sites located very close to major active faults.

6.3.12 Redundancy Reliability Factor, ρ

Lack of redundancy exists when the failure of a building component results in the failure of the entire structure. A conceptual explanation of a highly redundant and not-so-highly redundant building is quite easy as shown in Figure 6.58a, but quantifying redundancy with numerical values is another matter.

Redundancy provisions were first introduced into building codes and standards via the 1997 UBC. Since their original inception the redundancy provisions created much controversy with respect to the interpretation and implementation. The debate centered mainly on the following issues:

- A sliding redundancy value as was proposed originally, based on the force in only one of the elements of the system, was too precise and not technically justified.
- A better approach to determining redundancy is to base it on whether the loss or removal of an important component within the system would result in more than a 33% reduction in story strength, or would the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b, ASCE 7-05, Table 12.3-1).
- Checking redundancy throughout the entire building height is not necessary. Only those stories resisting more than 35% of the base shear in the direction of interest need to be checked.
- Well-distributed perimeter systems should automatically qualify for a redundancy value of 1.0—specifically, those structures that are regular in plan at all levels.

Now it seems that the issue is settled with the ASCE 7-05 permitting the redundancy factor to be taken as 1.0 for

1. Structures assigned to SDC B or C
2. Drift calculation and $P\Delta$ effects
3. Design of nonstructural components
4. Design of nonbuilding structures that are not similar to buildings

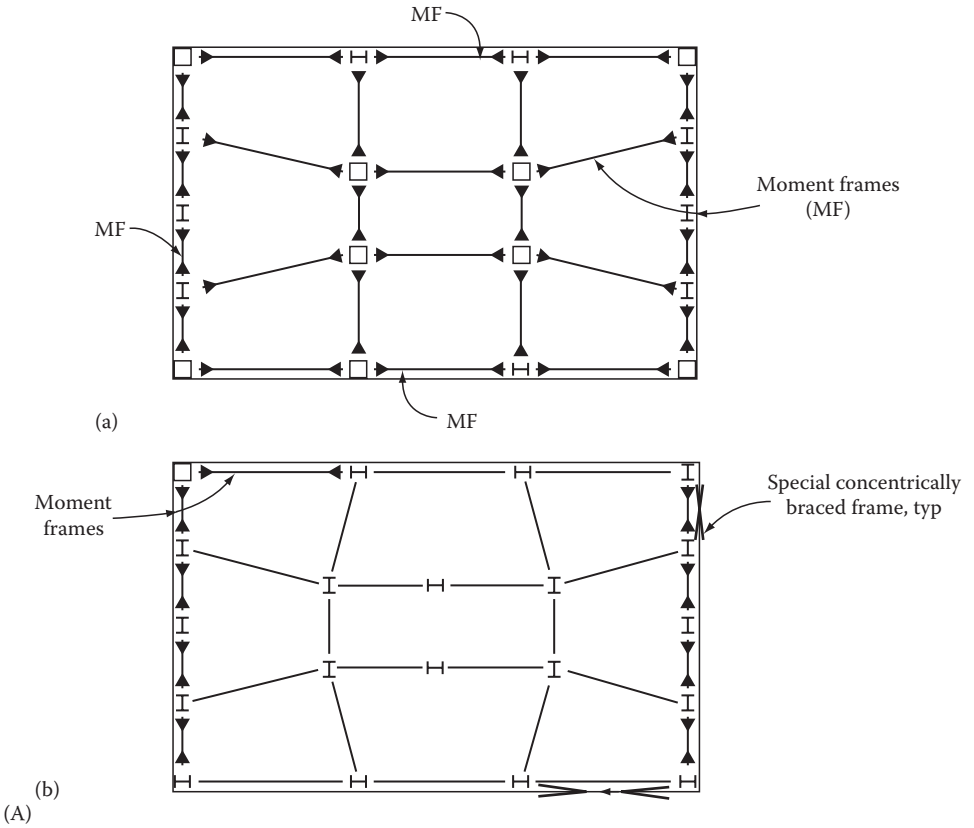


FIGURE 6.58 (A) (a) Highly redundant building and (b) not-so-redundant building.

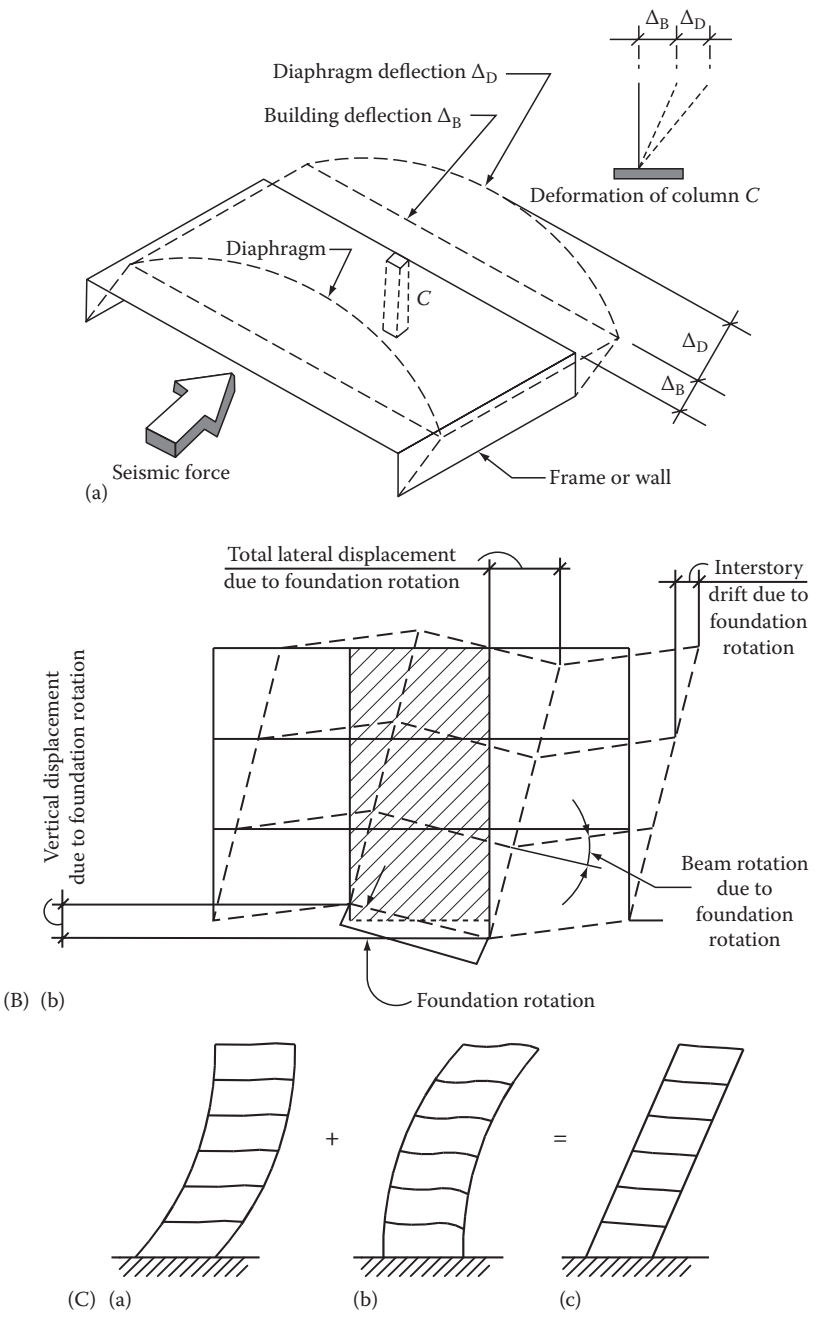


FIGURE 6.58 (continued) (B) Column deformation for use in compatibility considerations. (a) Deformation of column = building deflection Δ_B + diaphragm deflection Δ_D. (b) Foundation flexibility for deformation compatibility considerations. (C) Characteristic shape of fundamental mode: (a) shear distortion; (b) bending deformation; (c) overall combined deformation.

5. Design of collector elements, splices, and their connections for which the load combinations with overstrength factors are used
6. Design of members or connections where the load combinations with overstrength are required for design
7. Diaphragm loads determined using Equation 12.10-1 of ASCE 7-05
8. Structures with damping systems

A logical way to determine the lack of redundancy is to check whether a component's failure results in an unacceptable amount of the loss of story strength or in the development of extreme torsional irregularity.

In the ASCE 7-05, $\rho = 1.0$ or 1.3, depending on whether or not an individual element can be removed from the lateral-force-resisting system without

1. Causing the remaining structure to suffer a reduction of story strength of more than 33%
2. Creating an extreme torsional irregularity

Additionally, the redundancy factor, ρ, is 1 for the following cases:

1. *Moment frames*: The loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (see Table 5.25 for additional ASCE 7-05) comments.
2. *Shear wall or wall pier with a height-to-length ratio of great than 1.0*: The removal of a shear wall or wall pier with a height-to-length ratio of greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity.

The redundancy factor ρ is used to encourage designers to provide a reasonable number and distribution of vertical LFRS elements.

As stated previously, ASCE 7 contains a list of eight circumstances when ρ is permitted to be set equal to 1.0. Item one permits ρ to be 1.0 for all structures assigned to SDCA A, B, or C. Item two permits ρ to be set to 1.0 for drift calculation and determining *P*-delta effects. The balance of the items apply to components and elements rather than the primary LFRS.

ASCE 7 requires that ρ be 1.3 for all structures in SDC D, E, or F, unless the structure is qualified for a ρ of 1.0 using one of two possible methods. Both methods require further evaluation of each story that resists more than 35% of the base shear. In a three-story building, it would be anticipated that the bottom two stories require evaluation, but possibly not the top story.

In the first method, the user is asked to remove vertical resisting elements one at a time, and check to see if (1) the story strength is reduced by more than 33% or (2) if an extreme torsional irregularity is created with the element removed. If either of these conditions exist, a ρ of 1.3 must be used; if it does not, ρ may be taken as 1.0.

Table 6.13 lists evaluation requirements for determining seismic vulnerability of each story resisting more than 35% of base shear, for purposes of establishing $\rho = 1.0$ or 1.3.

6.3.13 Seismic Load Combinations

It may so happen that some elements of properly detailed structures cannot be designed to safely resist ground shaking demands through inelastic behavior. To ensure safety, these elements must be designed with sufficient strength to remain elastic. Calculation of special seismic loads, factored by the Ω_o coefficient, is a method of determining the maximum force these elements are ever likely to experience. ASCE 7-05 however, permits the special seismic loads to be taken as less than the value

TABLE 6.13
Requirements for Each Story Resisting More than 35% of the Base Shear

Lateral-Force-Resisting Element	Requirement
Braced frames	Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Moment frames	Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Cantilever columns	Loss of moment resistance at the base connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).
Other	No requirements

Source: ASCE 7-05, Table 12, 3-3.

computed by applying the Ω_o, when it can be shown that yielding of other elements in the structure will limit the load that can be delivered to the element. A case in point is the axial load induced in a column of a moment-resisting frame by shear forces in the connecting beams. The axial loads due to seismic need never be taken greater than the sum of the shears corresponding to the development of a beam hinge, considering the probable strength and strain-hardening effect. For frames controlled by beam hinge-type mechanisms, this would typically be $2M_p/L$, where M_p is the expected plastic moment capacity of the beam and L is the clear span length of beam. In the context of seismic design, the term capacity means the expected or median anticipated strength of the element, considering potential variation in material yield strength and strain-hardening effects. For this purpose, material strengths should not be reduced by capacity or resistance factors.

6.3.13.1 Vertical Seismic Load, $0.02S_{DS}$

The factor of $0.2S_{DS}$ placed on the dead load accounts for the effects of vertical acceleration and is not intended to represent the total vertical response. The concurrent maximum response of vertical accelerations and horizontal accelerations is unlikely, and, therefore, the direct addition of responses is not considered.

The ρ factor relates to the redundancy inherent in the seismic-force-resisting system and is, in essence, a reliability factor, penalizing designs which are likely to be unreliable due to concentration of the structure's resistance to lateral forces in a relatively few elements. However, it should be noted that there is very little research that speaks directly to the merits of redundancy in buildings for seismic resistance.

6.3.13.2 Overstrength Factor Ω_o

The seismic load effect with overstrength is intended to address those situations where failure of an isolated, individual, brittle element can result in the loss of a complete seismic-force-resisting system or in instability and collapse.

This somewhat arbitrary factor attempts to quantify the maximum force that can be delivered to nonductile elements based on historic observation that the real force that could develop in a structure may be three to four times the design levels. The use of the Ω_o provides an estimate of the maximum forces likely to be experienced by an element.

One case where structural overstrength should specifically be considered is in the design of columns beneath discontinuous braced frames and shear walls. Overstrength in shear walls could cause buckling failure of such columns with resulting structural collapse.

Although the most common cases in which structural overstrength can lead to an undesirable failure mode are identified in the ASCE Provisions, not all such conditions are noted. Therefore, designers should be alert to conditions where the isolated independent failure of any element can lead to a condition of instability or collapse and should use the overstrength factor Ω_o for the design of such elements. Other elements that may warrant such a design approach include transfer structures beneath discontinuous lateral-force-resisting elements and the diaphragms.

6.3.14 Elements Supporting Discontinuous Walls or Frames

The purpose of the special load combinations is to protect the gravity load–carrying system against possible overloads caused by overstrength of the lateral-force-resisting system. Either columns or beams may be subject to such failure; therefore, both should include this design requirement. Beams may be subject to failure due to overloads in either the downward or upward directions of force. Examples include reinforce concrete beams or unbraced flanges of steel beams or trusses. Hence, the provision has been limited simply to downward force, but instead to the larger context of vertical load. The connections between the discontinuous elements and the supporting members shall be adequate to transmit the forces for which the discontinuous elements were required to be designed.

6.3.15 Direction of Loading

Earthquake forces may act in both principal directions of the building simultaneously, but the effects in the two principal directions are unlikely to reach their maxima simultaneously. A reasonable and adequate method for combining them is to design structural elements for 100% of the effects of seismic forces in one principal direction combined with 30% of the effects in the orthogonal direction.

Orthogonal effects are typically negligible for beams, girders, slabs, and other horizontal elements that are essentially one-directional in their behavior, but they may be significant in columns or other vertical members that participate in resisting earthquake forces in both principal directions of the building. For two-way slabs, orthogonal effects at slab-to-column connections can be neglected provided the moment transferred in the minor direction does not exceed 30% of that transferred in the orthogonal direction and there is adequate reinforcement within an equivalent width of one and one-half times the slab thickness either side of the column to transfer all the minor direction moment.

6.3.16 Period Determination

In the denominator of base shear equations, T is the fundamental period of vibration of the building. It is preferable to determine this using the structural properties and deformational characteristics of the building, by using a dynamic analysis. However, a dynamic analysis can calculate the period only after the building has been designed. Therefore, an approximate method is necessary to estimate building period, with minimal information available on the building characteristics. Hence, the ASCE 7-05 provides simple formulas that involve only a general description of the building type and the overall height or number of stories.

Building periods, computed even with the use of sophisticated software, are only as good as the modeling assumptions used in the analysis and, to a great extent, are dependent on stiffness assumptions. The smaller the assumed stiffness, the longer the computed period, which translates directly into a lower design base shear. The computed period is thus open to possible abuse. Therefore, certain limits are imposed on the computed period. For design purposes, it may not be taken any larger than a coefficient C_u times the approximate period calculated. Reasonable mathematical rules should be followed such that the increase in period allowed by the C_u coefficient does not take advantage of when

TABLE 6.14
Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤0.1	1.7

Source: ASCE 7-05, Table 12.8.1.

the structure does not merit it. *Note that for purposes of drift analysis only, the upper bound limitation on the computed fundamental period T of the building does not apply.* It may be noted from Table 6.14 that larger values of C_u are permitted as the design spectral response acceleration parameter, at 1 s, S_{D1}, of a location decreases. This is because buildings in areas with lower lateral-force requirements are thought likely to be more flexible. High values of C_u for lower values of S_{D1} also result in less dramatic changes from prior practice in seismic risk areas. It is generally accepted that the equations for T_a are tailored to fit the types of construction common in areas with high lateral-force requirements. It is unlikely that buildings in lower seismic risk areas would be designed to produce as high a drift level as allowed by ASCE 7-05, due to stability ($P\Delta$) considerations and wind requirements.

In seismic design, the closer the frequency of an earthquake is to the natural frequency of a building, the more energy is introduced into the building structure. Buildings with shorter fundamental periods attract higher forces as evidenced by the code-based or site-specific response spectrum.

Conversely, taller buildings because of long fundamental periods tend to attract lower seismic forces. For wind design, the opposite behavior is observed. Longer fundamental periods are indicative of buildings that are more susceptible to dynamic amplification effects from wind gusts and therefore result in higher design forces.

Most designers are familiar with the use of the fundamental period of the structure, T, in conjunction with calculating the seismic response coefficient, C_s, for base shear determination using the equivalent lateral force procedure. The most straightforward method for determining the building period is to use the empirical formulas for the calculation of the approximate building period, T_a, presented in Chapter 12 of ASCE 7-05.

The equations are based on data from several instrumented buildings subjected to ground motion during seismic events. The formulas are intentionally skewed to represent a conservative estimation of the fundamental building period. Shorter building periods result in higher and more conservative base shears.

If desired, the approximate period, T_a, may be used for the strength design and checking the drift limits of the building. However, this practice typically results in significantly overly conservative results.

Therefore, ASCE 7-05 allows the use of a "properly sustained analysis." for period determination. In today's practice this means use of computer programs to perform an eigenvalue analysis to determine the mode shapes and periods of a building. It is important to note that the periods determined using an eigenvalue analysis are significantly longer than those determine using the approximate equations.

For strength design, ASCE 7-05 caps the maximum building period to the approximate building period T_a, multiplied by the factor, C_u. See ASCE 7-05 Table 12.8-1 (Table 6.14). The cap is intended to prevent the so-called sharp pencil effects resulting from erroneous assumptions used in the "properly

TABLE 6.15
Values of Approximate Period Parameters C_t and X

Structure Type	C_t	X
Moment-resisting frame systems in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by components that are more rigid and will prevent the frames from deflecting where subjected to seismic forces		
Concrete moment-resisting frames	0.016 (0.0466)[a]	0.9
		0.5
All other structural systems	0.02	0.75

Source: Adapted from ASCE 7-05, Table 12.8.2.

substantiated analysis." However, for the determination of seismic drift, ASCE 7-05 removes the cap and allows the engineer to use the building period resulting from analysis without restriction.

The most basic dynamic property of a structure is its fundamental period of vibration. ASCE 7 provides the following formula for the approximate period of vibration:

$$T_a = C_t h_n^x$$

where

h_n is the height of the highest (nth) level above the base, ft
x is the exponent dependent on structure type
= 0.80 for moment-resisting systems of steel
= 0.75 for eccentrically braced steel frames = 0.75 for all other structures
C_t is the coefficient dependent on structure type
= 0.028 for moment-resisting systems of steel
= 0.030 for eccentrically braced steel frames
= 0.020 for all other structures

Tabulated values of approximate period parameters C_t and x are shown in Table 6.15.

Note that ASCE 7 provides optional alternative definitions for the approximate period T_a for steel moment frames.

Damping is another dynamic property of the structure that affects earthquake performance. Observe that in the development of the ASCE 7 design spectra, a modest amount of damping that is applicable to all building types is included.

6.3.17 Inherent and Accidental Torsion

The torsional moment to be considered in the design consists of two parts:

1. M_t, the moment due to eccentricity between centers of mass and resistance for that story. This is computed as the story shear times the eccentricity perpendicular to the direction of applied earthquake forces.
2. M_{ta}, commonly referred to as "accidental torsion," which is computed as the story shear times the "accidental eccentricity," equal to 5% of the dimension of the structure (in the story under consideration) perpendicular to the applied earthquake forces.

Accidental torsion is intended to cover the effects of several factors that have not been explicitly considered in the design. These factors include the rotational component of ground motion about

a vertical axis; unforeseeable differences between computed and actual values of stiffness, yield strengths and dead-load masses; and unforeseeable unfavorable distributions of dead- and live-load masses.

6.3.18 Overturning

The structure must be designed to resist overturning moments statically consistent with the design story shears. In ASCE 7-05 *Provisions*, there is no modification factor *T*, to reduce the moment to account for the effects of higher mode except at the soil-foundation interface. A 25% reduction is allowed in overturning values if the analysis is performed by using ELF procedure, and a 10% reduction if the analysis is by modal response method. However, no reduction is permitted for cantilevered column type structures.

6.3.19 *P*Δ Effects

P-delta effects in a given story are due to the eccentricity of the gravity load above that story. If the story drift due to the lateral forces were Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load *P*, above the story. The ratio of the *P*-delta moment to the lateral force story moment is called stability coefficient, ϕ. If the stability coefficient is less than 0.10 for every story, *P*-delta effects on story shears and moments and member forces may be ignored. If, however, the stability coefficient ϕ exceeds 0.10 for any story, the *P*-delta effects on story drifts, shears, member forces, etc., for the entire structure must be determined by a rational analysis.

The *P*-delta procedure checks the static stability of a structure based on its initial stiffness.

There is justification for using the P-delta amplifier as based on elastic stiffness because

1. Many structures display strength well above the strength implied by code-level forces. This overstrength likely protects structures from stability-related failures.
2. The likelihood of a failure due to instability decreases with increased intensity of expected ground-shaking. This is due to the fact that the stiffness of structures designed for extreme ground motion is significantly greater than the stiffness of the same structures designed for lower intensity shaking or for wind. Since damaging, low-intensity earthquakes are rare, there would be little observable damage.

6.3.20 Drift Determination

In the ASCE 7-05, for the first time, allowable drifts are based on building occupancy category (see Table 6.16). The more significant (or "essential") the occupancy, the more restrictive the allowable story drift. For occupancy category IV buildings (essential facilities), the allowable story drift has been reduced by a factor of two relative to the UBC. The current requirement specifies an allowable story drift of 0.01, while the UBC value was 0.02. Depending on the selected building system, this can have a significant effect on the seismic design.

The design story drift is the difference in the deflections, δ_x, at the top and bottom of the story under consideration. The deflections, δ_x, are determined by multiplying the deflections, δ_{xe} (determined from an elastic analysis), by the deflection amplification factor, C_d. The elastic analysis is made for the seismic-force-resisting system using the prescribed seismic design forces and considering the structure fixed at the base. Stiffness other than those of the seismic-force-resisting system is not included since they may not be reliable at higher inelastic strain levels.

The deflections are determined by combining the effects of joint rotation of members, shear deformations between floors, the axial deformations of the overall lateral resisting elements, and the shear and flexural deformations of shear walls, if any. Centerline dimensions between the frame

TABLE 6.16
Allowable Story Drift, Δ_a[a,b]

Structure	Occupancy Category I or II	Occupancy Category III	Occupancy Category IV
Structures, other than masonry shear wall structures, four stories or less with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Source: ASCE 7-05, Table 12.12-1.

[a] h_{sx} is the story height below level x.

[b] For seismic-force-resisting systems comprised solely of moment frames in SDCs D through F, the allowable story drift shall comply with the requirements of Section 12.12.1.1 of ASCE 7-05.

[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 of ASCE 7-05 is not waived.

[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support that are so constructed that moment transfer between shear walls (coupling) is negligible.

elements often are used for analysis, but clear-span dimensions with consideration of joint panel zone deformation also may be used.

The term "drift" has two connotations: story drift and building drift.

1. "Story drift" is the maximum lateral displacement with a story (i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads).
2. "Building drift" is the lateral deflection of the building relative to the base. This is not used for drift control or stability considerations. However, it is important when considering seismic separation requirements.

There are many reasons for controlling drift; one is to control member inelastic strain, and the other to address stability considerations. The stability of members under elastic and inelastic deformation is a direct function of both axial loading and bending of members and is assured by limiting the drift and thus the resulting secondary moment from *P*-delta effects. Under small lateral deformations, secondary stresses are normally within tolerable limits. However, large deformations with heavy vertical loads can lead to significant secondary moments from *P*-delta effects. The drift limits, therefore, indirectly provide upper bounds for these effects.

Another reason for drift control is to restrict damage to partitions, elevator shafts, stair enclosures, glass, and other fragile nonstructural elements. The design of some nonstructural components that span vertically between floors can be complicated. For example, a glazing system supported by precast concrete spandrels must be designed to accommodate the full story drift, even though the height of the glazing system is only a fraction of the floor-to-floor height. The condition arises because the precast spandrels will behave as rigid bodies relative to the glazing system, and, therefore, all the drift must be accommodated by the joint between the precise spandrel and the glazing unit.

Determination of story drift involves the following steps:

1. Determine the lateral deflections at floor levels by elastic analysis. The lateral deflection at floor level x, obtained from this analysis, is termed δ_{xe}. The subscript "e" stands for elastic analysis.

2. Amplify d_{xe} by the deflection amplification factor, C_d. The resulting quantity, $C_d\delta_{xe}$, is an estimated design earthquake displacement at floor level x. ASCE 7-05 requires this quantity to be divided by the importance factor, I_E, because the forces under which the δ_{xe} displacement is computed are already amplified by I_E. Since drift limits are tighter for buildings in higher occupancy categories, this division by I_E is important. Without it, there would be a double tightening of drift limitations for buildings with seismic importance factors greater than 1. The quantity $C_d\delta_{xe}/I_E$ at floor level x is δ_x, the adjusted design earthquake displacement.
3. Similarly calculate the displacement δ_{x-1} for the story below level x. The story drift Δ_x is given by

$$\Delta_x = \delta_x - \delta_{x-1}$$

The Δ_x values must be kept within limits, as given in Table 6.16 (ASCE Table 12.12.1).

Three items are worth noting:

1. The design story drift must be computed under strength-level design earthquake forces irrespective of whether member design is done using the strength design or the allowable stress design load combinations.
2. The redundancy coefficient, p, is equal to 1.0 for computation of the design story drift.
3. For determining compliance with the story drift limitations, the deflections, δ_x, may be calculated as indicated previously using design forces corresponding to the fundamental period of the structure, T, calculated without the limit, $T < C_u T_x$. The same analytical model of the building used in determining the deflections must be used for determining T. The waiver does not pertain to the calculation of drifts for determining $P\Delta$ effects on member forces, overturning moments, etc. If $P\Delta$ effects are significant, the design story drift must be increased by the resulting incremental factor.

As stated previously, the $P\Delta$ effects in a given story are due to the eccentricity of the gravity load above that story. If the design story drift due to the lateral forces is Δ, the bending moments in the story are augmented by an amount equal to Δ times the gravity load P above the story. The ratio of the $P\Delta$ moment to the lateral-force moment is designated as the stability coefficient θ. If θ is greater than 0.10 for every story, then the $P\Delta$ effects, must be determined by a rational analysis. However, with the availability of computer programs that take into consideration $P\Delta$ effects automatically within the analysis, separate calculations of θ, for determining whether $P\Delta$ is significant, are rarely necessary.

$P\Delta$ effects are much more significant in buildings assigned to low SDCs than in buildings assigned to high SDCs. This is because lateral stiffness of buildings is typically greater for higher SDCs.

The design story drift limits reflect consensus judgment taking into account the goals of drift control outlined above. In terms of life safety and damage control objectives, the drift limits should yield a substantial, though not absolute, measure of safety.

To provide a higher performance standard, the drift limit for structures contained in the four occupancy categories are progressively more stringent.

It should be emphasized that the drift limits, Δ_x, are story drifts and, therefore, are applicable to each story (i.e., they must not exceed in any story even though the drift in other stories may be well below the limit).

Stress or strength limitations imposed by design level forces may provide adequate drift control for low-rise buildings. However, it is expected that the design of moment-resisting frames and tall, narrow shear wall buildings will be governed at least in part by drift considerations. In areas having large design spectral response accelerations, S_{DS} and S_{D1}, it is expected that seismic drift considerations will predominate for buildings of medium height. In areas having low design spectral response accelerations and for very tall buildings in areas with large design spectral response accelerations, wind considerations generally will control. Even then, as stated many times in this book, the detailing of members must comply with the governing seismic provisions.

Due to probable first mode drift contributions, the ELF procedure may be too conservative for drift design of very tall moment-frame buildings. It is suggested for these buildings, where the first mode would be responding in the constant displacement region of a response spectrum (where displacement would be essentially independent of stiffness), that the response spectrum procedure be used for design even when not required, the reason being economy.

Building separations are necessary between two adjoining buildings or parts of the same building, with or without frangible closures, for the purpose of permitting the adjoining buildings or parts to respond independently to earthquake ground motion. Unless all portions of the structure have been designed and constructed to act as a unit, they must be separated by seismic joints. For irregular structures that cannot be expected to act reliably as a unit, seismic joints should be used to separate the building into units whose independent response to earthquake ground motion can be predicted.

The separation should be sufficient to avoid damaging contact and possible destructive hammering between buildings. It is recommended that the separation between buildings be equal to the statistical sum of the lateral deflections, δ_x, of the two units.

Because drift due to lateral loads increases with height, it is evident the width of separation joint also increases similarly. However, if the effects of hammering can be shown not to be detrimental, the separation distances can be reduced.

6.3.21 Deformation Compatibility

The ASCE 7-05 requires that for buildings in SDC D, E, or F, all structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, must be designed and/or detailed to adequately maintain support of gravity loads when subjected to the expected deformations caused by seismic forces. Important features of deformation compatibility requirements are as follows:

1. Expected deformations must be greater of the maximum inelastic response displacement, δ_x, considering $P\Delta$ effects and deformation induced by a story drift of 0.0025 times the story height.
2. When computing expected deformations, stiffening effects of those elements not part of the lateral-force-resisting system must be neglected.
3. Forces induced by expected deformations may be considered factored forces.
4. In computing forces, restraining effect of adjoining rigid structures and nonstructural elements must be considered.
5. For concrete elements that are not part of the lateral-force-resisting system, assigned flexural and shear stiffness properties must not exceed one-half of gross section properties, unless a rational cracked section analysis is performed.
6. Additional deformations that may result from foundation flexibility and diaphragm deflection must be considered.

The deformation compatibility requirements prescribed for SDC D, E, or F buildings provide a means of protecting elements of the structure that are not part of the seismic-force-resisting system. Just because certain elements of the structure are not intended to resist seismic forces and are not detailed as such, does not prevent them from actually providing this resistance resulting in severe damage, hence the compatibility requirements.

Of particular concern are the shears that can be induced in structural components that are not part of the lateral-force-resisting system since sudden shear failures have been catastrophic in past earthquakes.

The ASCE Provisions encourage the use of intermediate or special detailing in beams and columns that are not part of the lateral-force-resisting system. In return for better detailing, such beams and columns are permitted to be designed to resist moments and shears from unamplified

deflections. This reflects observations that well-detailed components can accommodate large drifts by responding inelastically without losing significant vertical load–carrying capacity.

It should be noted that diaphragm and foundation deformations must be included in checking compatibility requirements (see Figure 6.58b and c).

6.3.22 Seismic Response Modification Coefficient, *R*

The response modification factor, *R*, reduces the forces resulting in a large earthquake to more managable design seismic forces. It is a function of the ductility and overstrength of the lateral-force-resisting system and is based on the experience in past earthquakes that have demonstrated that buildings designed to a significantly lower base shear can adequately resist seismic forces without collapse. This experience provides the basis for use of the *R* factor. The reason for adequate performance at a lower base shear is thought to be the result of both extra and reserve strength in the structural system and nonstructural elements and finishes, and stable inelastic behavior of the structural elements. Based on reserve strength and inelastic behavior, ASCE 7-05 allows use of *R* values significantly greater than 1.0 (as much as 8). See Table 12.2-1, ASCE 7-05.

6.3.23 Seismic Force Distribution for the Design of Lateral-Load-Resisting System

It is interesting to note that seismic forces are calculated and distributed throughout the structure in the reverse order used for evaluating wind forces. For wind design, forces at each level are calculated first from the external pressure and suctions and then the total shear (referred to as base shear in seismic terminology) is determined by summing the forces in the horizontal direction. For determining seismic forces, the process is just the reverse. The seismic shear at the base, the base shear, is calculated as a product of the applicable seismic coefficient, C_s, and the total mass M of the structure equal to W/g, where W is the total seismic weight of the building and g is the acceleration due to gravity.

The next step after determining the base shear V, is the vertical distribution of V as story forces F_x at each story.

The formula for F_x will produce a triangular distribution of horizontal story forces if the masses (tributary weights) assigned to the various story levels are all equal. If the weights are not equal, some variation from the triangular distribution will result effectively mimicking the first-mode shape. Therefore, accelerations and, correspondingly, inertia forces ($F = Ma$) typically increase with increasing height above the shear base.

The distribution of F_x is the same for all SDCs and is given by

$$F_x = C_{vx}V$$

and

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

where

C_{vx} is the vertical distribution factor
V is the total base shear
w_i, w_x are the tributary weights assigned to level i or x
h_i, h_x are height from the base of structure to level i or x
k is an exponent related to the structural period
= 1 for structures having a period of 0.5 s or less
= 2 for structures having a period between 0.5 and 2.5 s

As stated previously, the distribution of base shear over the building height is based on the assumption that the characteristic shape of the fundamental mode is a straight line from the shear base to top of the building. Although this is an approximation, the supposition is quite reasonable for typical buildings. The justification for the assumption may be understood if it is recognized that the total distortion of a typical building is the sum of two effects:

1. The shear distortion of the frame
2. The change in length of columns due to overall bending of the building (flexural deformation)

The former tends to produce a deflected shape that is concave to the left, and the latter a shape concave to the right (see Figure 6.58b). The combined effect results in a shape that approaches a straight line.

6.3.24 Seismic Loads due to Vertical Ground Motions

During earthquakes, vertical ground motions generate vertical forces in addition to the dominant horizontal seismic forces due to horizontal ground motions. The vertical forces are invariably much smaller than the horizontal forces.

ASCE 7 incorporates the effect of vertical ground motions in determining the seismic forces. The result of including the vertical seismic forces is an increase in net uplift and an increase in downward forces when considering overturning effects.

As stated previously, the method used to calculate the horizontal story forces involves three parts. The first part is calculating the *base shear* (the horizontal force acting at the base of the building, V). The second part is assigning the appropriate percentages of this force to the various story levels throughout the height of the structure (story forces). The third part is to determine the forces on particular elements as a result of the story forces (element forces). However, there are several multiplying factors required to convert these *element forces* into design seismic forces for the elements at a story.

The story forces are given the symbol F_x (the force at level x). It is evident that the sum of the F_x forces must equal the base shear V. It should be noted that the story forces increase with increasing height above the base of the building. The magnitudes of the story forces depend on the mass distribution over the height. The vertical distribution provisions have an exponent of between one and two on the height term in the vertical distribution. This exponent accounts for the increased top-story forces that can occur in buildings with long periods.

The effect of vertical seismic forces is considered in ASCE 7-05 by using an equation of the form

$$E = \rho Q_E \pm 0.2 S_{DS} D$$

where

ρ = a factor representing redundancy and reliability
Q_E = the horizontal seismic force component
S_{DS} = the design spectral response acceleration at short periods
D = the deal load
E = the effect of horizontal and vertical earthquake induced forces

The first term of the right side of the equation above, represents horizontal forces, while the second term represents vertical forces. The latter reduces dead load for overturning resistance, while increasing downward vertical reactions.

6.3.25 Seismic Force for the Design of Diaphragms

A unique aspect of seismic force evaluation is that there are two different sets of story force distributions for the primary LFRS. One set of story forces is to be used in the design of the vertical

elements in the LFRS, and the other set applies to the design of horizontal diaphragms. A different notation system is used to distinguish the two sets of story forces.

The forces for designing the vertical elements are given the symbol F_x, and the forces applied to the design of horizontal diaphragms are given the symbol F_{px}. Both F_x and F_{px} are horizontal *story forces* applied to level x. Thus, the horizontal forces are assumed to be concentrated at the story levels in much the same manner as the masses tributary to a level are "lumped" or assigned to a particular story height.

The rationale behind two different F_x and F_{px} distributions has to do with the fact that the forces occurring during an earthquake change rapidly and even abruptly with time. Because of these rapidly changing forces and the different modes of vibration, it is likely that maximum force on an individual horizontal diaphragm will not occur at the same instant in time as the maximum force on another horizontal diaphragm. Hence, the larger loading F_{px} is to account for the possible larger *instantaneous* forces that will occur on individual horizontal diaphragms. Observe that F_{px} story force is that used in the design of individual horizontal diaphragms, diaphragm collectors (drag struts) and related connections.

Floor and roof assemblies obligatory for resisting gravity loads also must provide the necessary strength and stiffness to effectuate load path continuity. This linkage between the vertical and horizontal structural elements is necessary to distribute lateral loads due to wind and seismic, to the lateral support subsystems. The subsystems typically consisting of vertical bracing systems such as moment frames, braced frames and shear walls are generally referred to as vertical–lateral-load-resisting systems, VLLRS. However, to keep the verbiage easy-flowing, we will refer to these simply as vertical systems, it being understood that these systems are the designated lateral-load-resisting systems. Their primary function, in addition to carrying vertical loads is, to resist lateral loads due to seismic and wind loads.

In this role, the floor and roof surfaces which are most typically horizontal, act as beams or diaphragms spanning between vertical subsystems. They may effectively engage beam elements at the perimeter (transverse to the direction of the load) as top and bottom "chords" or "flanges," in which case the bending moment can be resolved into a tension and compression couple and considered resisted by the beam elements (with shear resisted by the diaphragm surface). In the absence of such flange elements to take the moment couple, the floor or roof must act as a deep plate taking both bending and shear forces. Either type of diaphragm behavior requires effective transfer of bending and shear forces in the plane of the floor and roof, necessitating careful detailing of connections between the diaphragm and the lateral support subsystem.

The stiffness of the diaphragm has an important effect on the proportionate distribution of lateral loads to the various components of the lateral support system. This effect is indicated in Figure 6.48 which shows how diaphragm and lateral support element relative rigidities influence load distribution. At one extreme is the *rigid diaphragm*, which distributes the loads to the lateral support elements in proportion to just *their* rigidities. At the other extreme is the *flexible diaphragm*, which transmits lateral loads based on tributary area. The *semirigid diaphragm* falls somewhere in between these two extremes and involves both aspects of behavior.

No new concepts are involved in the analysis and design of floor and rood assemblies for gravity loads. The slab, beam, or truss elements are designed using the well-accepted assumption that gravity loads are proportional to the areas tributary to these elements.

Perhaps the easiest way to understand the role of a diaphragm in seismic design is to ask ourselves a question: "How does the inertia load generated in a unit area of a diaphragm get from there to the shear base where it ultimately dissipates into the ground?" Thus achieving a logical load path between points of load application resistance is our first step. Other design steps, in a manner of speaking, are quite simple.

The next step is to determine bending moments and shears due to F_{px}. Considering the most elementary case of a simple span between two walls of equal stiffness, we may idealize the diaphragm as a simple span beam on its side. We note the following formulas apply for our diaphragm.

Unit diaphragm shear is

$$V_u = \frac{wL}{2B}$$

where
V_u is the maximum unit shear
w is the uniform lateral load
L is the span of diaphragm between lateral support elements
B is the width of diaphragm, parallel to direction of lateral load

Using the familiar beam bending formula, the diaphragm bending stress is

$$F_b = \frac{Mc}{I}$$

where
F_b = bending stress
M = maximum bending moment, in our case is equal to wL^2/B
c = $B/2$
I = moment of inertia in the plane of bending

If the diaphragm has flanges, the moment may be assumed to be resisted by a tension and compression couple computed as follows:

$$C = T = \frac{M}{b}$$

6.3.25.1 Distribution of Seismic Forces for Diaphragm Design

Having explained the concept behind the formula for F_{px} for use in diaphragm design, we now turn our attention to the distribution of this force in the plane of the diaphragm. The theory behind this distribution is based on the idea of inertial forces generated in the diaphragm. If it is visualized that the mass tributary to each square foot of diaphragm has a corresponding inertial force generated by an earthquake force, then the loading shown in Figure 6.59a through c becomes clear. If each square foot of area has the same weight (and hence the same mass), the distributed seismic force is in proportion to the length of the diaphragm that is parallel to the direction of seismic force. Hence, the magnitude of the distributed force is large where the dimension of the diaphragm parallel to the force is large and it is small where this dimension is small. The same argument holds true if the distributed unit mass of the diaphragm is not the same for the entire diaphragm are as shown in Figure 6.59c.

6.3.25.2 General Procedure for Diagram Design

In its role as a distributor of seismic loads to vertical lateral supports, the floor or roof surface act as a diaphragm essentially responding as a horizontal beam, spanning between lateral support points. It may engage beam elements at the perimeter transverse to the direction of load as top and bottom *chords* or flanges. In this case the bending moment can be resolved into a tension and compression couple and considered resisted by the beam elements. If, on the other hand, there are no beam elements, the floor or roof must act as a deep plate resisting both bending and shear forces. Either type of diaphragm behavior requires effective transfer of bending and shear forces in the plane of the roof or floor, necessitating careful detailing of connections between the diaphragm and the lateral support system.

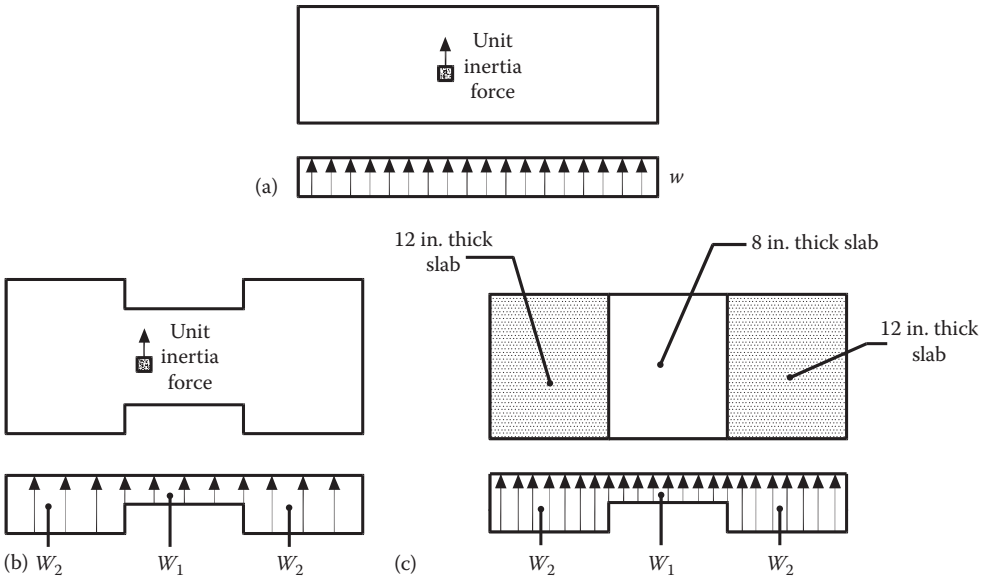


FIGURE 6.59 Distribution of seismic forces for diaphragm design: (a) diaphragm of uniform geometry and weight; (b) diaphragm of varying geometry; (c) diaphragm of different slab thicknesses.

Collectors are elements of the floor or roof structures that serve to transmit lateral forces from their location of origin to the vertical seismic-force-resisting elements (e.g., walls or moment frames) of the building. Typically collectors transfer earthquake forces in axial tension or compression. When a collector is a part of the gravity-force-resisting system, it is designed for seismic axial forces along with the bending moment and shear force from the applicable gravity loads acting simultaneously with seismic forces.

When subjected to lateral forces corresponding to a design earthquake, most buildings are expected to undergo inelastic, nonlinear behavior. Typically, the structural elements of a building that are intended to perform in the nonlinear range are the vertical elements of the seismic-force-resisting system, such as structural walls or moment frames. For the intended seismic response to occur, other parts of the seismic-force-resisting elements, should have the strength to remain essentially elastic during an earthquake. It is for this reason ACI 318-08, Section 21.11 mandates design of collectors and their connections in buildings assigned to SDC D and higher, for seismic forces amplified by a factor, Ω_o. The amplification factor ranges between 2 and 3 depending upon the type of seismic system.

The intent of the Ω_o amplification factor, as stated previously is to allow for the likely overstrength of the vertical seismic-force-resisting elements so that major yielding does not occur in collectors and their connections prior to yielding and inelastic response at the vertical elements of the building's seismic-force-resisting system.

6.3.25.3 Diaphragm Design Summary: Buildings Assigned to SDC C and Higher

Step 1

- Evaluate the diaphragm inertial force F_{px} at the floor and roof levels by the formula

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \text{ASCE 7-05 Eq. (12.10.1)}$$

where

F_{px} = diaphragm design force
F_i = design force applied to level i
w_i = weight tributary to level i
w_{px} = weight tributary to the diaphragm at level x

F_{px} need not exceed $0.4S_{DS}Iw_{px}$ but shall not be less than $0.2S_{DS}Iw_{px}$.

- Observe that additional shear forces resulting from transfer of vertical seismic elements or changes in their relative stiffness must be added to F_{px}. These additional forces shall be multiplied by the redundancy factor, ρ, equal to that used in the design of the structure.

It should be noted that F_{px} computed from the equation above, is typically larger than story force F_x determined by

$$F_x = C_{vx}V \qquad \text{ASCE 7-05 Eq. (12.8-11)}$$

where

$$C_{vx} = \text{vertical distribution factor} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{ASCE 7-05 Eq. (12.8-12)}$$

V is the total design lateral force often referred to as base shear (kip)
w_i and w_x are the portion of the total gravity load of the structure (w) located or assigned to level i or x
h_i and h_x are the height (ft) from the base to level i or x
k is an exponent related to the structure period as follows:
For structures having a period of 0.5 s or less, $k = 1$
For structures having a period of 2.5 s or more, $k = 2$
For structures having a period between 0.5 and 2.5 s, k shall be 2 or shall be determined by linear interpolation between 1 and 2

The formula for F_{px} allows for a higher-mode participation that can result in larger forces at individual diaphragm levels than predicted by the equation for F_x.

Step 2

- Perform a 3D lateral load analysis of the building by applying F_{px} at floor and roof levels. Include torsion but ignore its effect if it reduces shear in the vertical lateral-load-resisting elements.

Step 3

- Determine the net shear in the vertical elements due to F_{px} equal to the difference in shears resisted by the vertical elements immediately above and below the level of the diaphragm being designed. Conceptually the shear forces may be considered as reactions to the inertial forces of the diaphragm at that level.

Step 4

- Determine a set of equivalent loads at the diaphragm level that is in equilibrium with the shear forces determined in step 3. Use both force and moment equilibrium conditions. The equivalent loads may be determined as a combination of primary action due to F_{px} and a secondary action due to torsional effects.

Step 5

- Using the equivalent loads, determine the shear and bending moment at critical sections of the diaphragm.
- Compute the shear per unit length to check the shear capacity of the diaphragm. Provide collectors, also referred to as drag beams, to carry the shear that is in excess of force transferred directly into the vertical elements.

Step 6

- Assuming that concrete topping over metal deck is designed as diaphragm, calculate the ultimate shear capacity of the diaphragm as follows:

$$V_u = \phi(V_c + V_s)$$
$$= \phi A_{cv}(2\sqrt{f'_c} + \rho_n f_y)$$

Note that the strength reduction factor for shear, ϕ, in diaphragm design must not exceed the value used for the shear design of vertical elements of lateral-force-resisting systems.

Step 7

- Check perimeter beams (or equivalent widths of slab assumed to act as beams), and their connections for diaphragm chord forces.
- Extend chords at reentrant corners, if any, to develop the forces calculated at critical sections.

The reader is referred to the publication "Reinforced Concrete Design of Tall Buildings" by the author, for a numerical example of diaphragm design.

6.3.26 Catalog of Seismic Design Requirements

The seismic design requirements given in the ASCE 7-05 are cascading, meaning that what applies to a lower SDC building also applies to buildings in higher SDCs. For example, if analysis using bidirectional lateral loads is required for an SDC B building, they are also a requirement for SDC C, D, E, and F buildings. In other words, what applies to SDC A applies to SDC B through F, what applies to SDC C applies to SDC D through F, and so on.

6.3.26.1 Buildings in SDC A

1. Determine a pseudo-seismic lateral force at each level by using the following equation:

$$F_x = 0.01 w_x$$

where
F_x is the design lateral force applied at story x
w_x is the portion of the total dead load of the structure, D, located at or assigned to level x

This minimum base shear equal to 1% of the seismic weight of the structure applies to all buildings irrespective of SDC. The term w_x applies only to the dead load of the floor. It is a revision to ASCE 7-02 provision in which w_x included a list of other loads such as 25%

of the floor live load in a storage structure, partition load, weight of permanent equipment, and 20% of the flat roof snow load. The load F_x does not represent a seismic-related calculation, but rather meant to provide a certain level of strength relative to the mass of the structure to ensure structural integrity.

2. Apply the static lateral force F_x independently in two orthogonal directions. The lateral forces in each direction shall be applied at all levels simultaneously. Orthogonal combination procedure of applying 100% of the forces in one direction plus 30% in the perpendicular direction is not required. Similarly, simultaneous applications of orthogonal ground motions are not warranted.
3. Interconnect all parts of the structure to provide a continuous load path. Tie any smaller portion of the structure to the remainder of the structure with connections having a lateral design strength not less than 5% of the portion's weight.
4. Provide connections at supports for each beam, girder, or truss for resisting a horizontal force acting parallel to the member. The connections shall have a minimum design strength of 5% of the dead plus live load reaction.
5. Anchor concrete walls to the roof and all floors to provide lateral support for the wall. The connection shall be capable of resisting a horizontal force equal to 5% of the wall weight but not less than 280 lb/linear foot.
6. The following combinations of factored loads using strength design are applicable. Only the more common dead, live, wind, and earthquake loads are given here.
   1. $1.4D$
   2. $1.2D + 1.6L + 0.5L_r$
   3. $1.2D + 1.6L_r + (L \text{ or } 0.8W)$
   4. $1.2D + 1.6W + L + 0.5L_r$
   5. $1.2D + 1.0E + L$
   6. $0.9D + 1.6W$
   7. $0.9D + 1.0E$

   In the above equations, the term E refers to the effect of earthquake loads as determined by applying a horizontal load of F_x equal to 1% of the building dead load at each level. Vertical effects, E_v, of the seismic loads, design coefficients, and seismic factors such as response modification coefficient R, over strength factors Ω_o, and deflection amplification factor, C_d, are not of concern in the design of structures assigned to SDC A.
7. Just about any structural system that has a continuous load path is permitted for buildings assigned to SDC A. You are not restricted to the list give ASCE 7-05, Table 12.2.1. The structural members and connections need only to be designed for the forces determined from the analysis. The nonductile detailing requirements given in the applicable standards are presumed to be sufficient to provide the necessary strength and ductility for buildings assigned to SDC A.
8. For SDC A buildings, ASCE 7-05 provisions do not require structural design of diaphragms, chords, and collectors. This is not so for buildings in SDC B through F.
9. There is no requirement to increase the design loads due to horizontal and vertical structural irregularities. Extreme irregularities are not prohibited. This is not the case for buildings assigned to SDC B through F.
10. There is no requirement to multiply the torsional moment by a torsional amplification factor.
11. There is no requirement to compute the story drift, Δ, along the building edges.
12. Elements supporting discontinuous walls or fames need not be designed for load combinations along with overstrength factor Ω_o.
13. There is no height limit for buildings exhibiting extreme weak story irregularity.

6.3.26.2 SDC B Buildings

It is well worth remembering that SDC B buildings will be constructed in the largest portion of the United States. Earthquake-resistant requirements for SDC B are increased appreciably over Category A requirements, but they still are quite simple compared to requirements in areas of high seismicity or for buildings assigned to SDC C and higher. The Category B requirements specifically recognize the need to design diaphragms, provide collectors and reinforcing around openings. These requirements may seem elementary and obvious but, because they are not specifically covered in many codes, some engineers totally neglect them.

SDC B includes occupancy category I, II, and III structures in regions of moderate seismicity. Structures in this category must be designed for the calculated forces in addition to the requirements of SDC A. The design requirements are as follows:

1. Instead of lateral force, $F_x = 0.01W_x$ at each level, we now use a rational procedure to determine the total design seismic force and its distribution over the height of the building. Typically, the ELF procedure or a dynamic procedure is used for this purpose.
2. The building structure shall include complete lateral and vertical-force-resisting systems capable of providing adequate strength stiffness and *energy dissipating capacity* to withstand the design ground motions without exceeding prescribed limits of deformation and strength demand.
3. The directions of application of seismic forces used in the design shall be those which will produce the most critical effects. However, application of seismic forces independently in two orthogonal directions (as for SDC A buildings) is deemed sufficient. Similarly orthogonal effects are permitted to be neglected.
4. The connections shall develop the strength of the connected members or the forces determined from seismic analysis. Adequate strength shall be provided in individual members to resist the shears, moment, and axial forces determined from seismic analysis.

   It is always good to remember that analysis of a structure for a given design ground motion alone does not make a structure earthquake resistant; additional details are necessary to provide adequate earthquake resistance in structures. Experienced seismic designers normally fill these requirements, but because some requirements are not formally specified, they often are overlooked by inexperienced engineers.

   Probably the most important single requirement of an earthquake-resistant structure is that it be tied together to act as a unit. This is important not only in earthquake-resistant design, but also is an indispensable strategy in resisting high winds, floods, explosion, progressive failure, and even such hazards as foundation settlement. Hence, the requirement that all parts of the building (or unit if there are separation joints) be so tied together. Any part of the structure must be tied to the rest to resist a force of $0.133S_{DS}$ (but not less than 0.05) times the weight of the smaller. In addition, beams must be tied to their supports or columns, and columns to footings for a minimum of 5% of the dead and live load reaction.

   The connections shall be capable of transmitting the seismic force, F_p, induced by the parts being connected. Any small portion of the structure shall be tied to the main structure with connections capable of delivering a seismic force equal to $0.33S_{DS}$, times the weight of the smaller portion, or 5% of the portion weight, whichever is greater.
5. Extreme weak story irregularity is permitted for SDC B buildings not over two stories or 30 ft in height. The height limit does not apply where the extreme weak story is capable of resisting a total seismic force equal to Ω_o times the design force.
6. There is no requirement to increase the diaphragm forces due to horizontal or vertical irregularities.

7. Torsional moment due to accidental torsion need not be amplified.
8. Performing a 3D dynamic analysis is a requirement for structures exhibiting torsional irregularity, extreme torsional irregularity, out-of-plane irregularity, or nonparallel-systems irregularity.
9. There is no requirement to compute the story drift, Δ, along the building edges.
10. Extreme torsional irregularity, extreme soft story irregularity, or extreme weak story irregularity are not prohibited.
11. Designing the elements supporting discontinuous walls or frames for load combinations using overstrength factor Ω_o is a requirement.
12. Performing a 3D dynamic analysis with due consideration for diaphragm stiffness, use of cracked section properties for concrete elements, and inclusion of $P\Delta$ effects is a requirement.
13. There is no need to multiply, M_{ta}, the torsional moment due to accidental torsion by a torsionally amplification

$$A_x = (\delta_{max}/1.2\delta_{ave})^2 \leq 3.0$$

6.3.26.3 SDC C Buildings

The requirements for Category C are more restrictive than those for Categories A and B. Also, a nominal interconnection between pile caps and caissons is required.

SDC C includes occupancy category I, II, and III structures in regions of severe seismicity. The design requirements are as follows:

1. All requirements SDC A and B also apply to SDC C.
2. A geotechnical investigation shall be conducted and a report shall be submitted that includes an evaluation of the following potential geologic and seismic hazards:
   a. Slope instability
   b. Liquefaction
   c. Differential settlement
   d. Surface displacement due to faulting or lateral spreading

   The report shall contain recommendations for appropriate foundation designs or other measures to mitigate the effects of the previously mentioned hazards. Where deemed appropriate by the authority having jurisdiction, a site-specific geotechnical report is not required when prior evaluations of nearby sites with similar soil conditions provide sufficient direction relative to the proposed construction.
3. Foundations:

   Pole-type structures: Where construction employing posts or poles as columns embedded in earth or embedded concrete footings is used to resist lateral loads, depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria as established in the foundation investigation report.

   Foundation ties: Individual pile caps, drilled piers, or caissons shall be interconnected by ties. All ties shall have a design strength in tension or compression at least equal to a force equal to 10% of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraint will be provided by reinforced concrete beams within slabs-on-grade or reinforced concrete slabs-on-grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

   Pile anchorage requirements: Where required for resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete filled steel pipe H piles to the pile cap shall be made by means other than concrete bond to the base steel section.

Exception: Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

4. Increase design forces determined by static procedure by 25% for connection of diaphragms to vertical elements and to collectors, and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for load combinations with overstrength factor Ω_o (ASCE 7-05, Section 12.3.3.4).
5. Multiply, M_{ta}, the torsional moment due to accidental torsion by a torsional amplification factor A_x using the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{ave}} \right)^2 \leq 3.0$$

6. Compute story drift, Δ, as the largest difference of the deflections along any of the building edges.
7. Use 100% of the load in one direction and 30% of the load in the perpendicular direction, if you are using ELF or modal analysis. Use simultaneous application of load in both directions, if you are analyzing the structure using a linear or nonlinear response history procedure.

6.3.26.4 SDC D Buildings

Category D requirements compare roughly to present design practice in seismic areas of California for buildings other than schools and hospitals. Interaction effects between structural and nonstructural element must be investigated. Foundation interaction requirements are increased.

SDC D includes occupancy category I, II, III, and IV structures in regions of high seismicity, but not located close to a major fault, as well as occupancy category IV structures in regions of somewhat less severe seismicity. The use of some structural systems is restricted in this design category and dynamic analysis must be used for design of irregular structures. The design requirements are as follows:

1. All requirements of SDC A, B, and C also apply to SDC D.
2. Additional geotechnical investigation report requirements shall include the following:
   a. The determination of lateral pressures on basement and retaining walls due to earthquake motions.
   b. The potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes, and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration is permitted to be determined based on a site-specific study taking into account soil amplification effects or, in the absence of such a study, peak ground accelerations shall be assumed equal to $S_s/2.5$.
   c. Assessment of potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls, and flotation of buried structures.
   d. Discussion of mitigation measures such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures, and how they shall be considered in the design of the structure.
3. A special moment frame that is used but not required by ASCE 7-05, Table 12.2.1, shall not be discontinued and supported by a more rigid system with a lower response coefficient *R*, unless the building height is limited to two stories, and a 25% increase in loads is taken in the design of diaphragm elements.

4. Vertical irregularity Type 5b of ASCE 7-05 Table 12.3-2 (Table 6.10 of this book) shall not be permitted.
5. For structures having a horizontal structural irregularity of Type 1a, 1b, 2, 3, or 4 in ASCE 7-05, Table 12.3-1 or a vertical structural irregularity of Type 4 in Table 12.3-2, the design forces shall be increased 25% for connections of diaphragms to vertical elements and to collectors and for connections of collectors to the vertical elements. Collectors and their connections also shall be designed for these increased forces unless they are designed for the load combinations with overstrength factor.
6. Horizontal structural components shall be designed for a net upward force of 0.2 times the dead load in additional to the other applicable load combinations.
7. Redundancy factor, ρ, shall equal 1.3 unless one of the two following conditions is met, whereby ρ is permitted to be taken as 1.0:
   a. Each story resisting more than 35% of the base shear in the direction of interest shall comply with ASCE 7-05, Table 12.3-3.
   b. Structures that are regular in plan at all levels provided that the seismic-force-resisting systems consist of at least two bays of seismic-force-resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35% of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height for light-framed construction.
8. Foundations:
   Anchorage of the pile into the pile cap should be conservatively designed to allow energy dissipating mechanisms, such as rocking to occur in the soil without structural failure of the pile. Precast prestressed concrete piles are exempt from the concrete special moment frame column confinement requirements since these requirements were never intended for slender, precast prestressed concrete elements and will result in unbuildable piles. These piles have been proven through cyclic testing to have adequate performance with substantially less confinement reinforcing than required by ACI 318. Therefore, a transverse steel ratio reduced from that required in frame columns is permitted in concrete piles. It should be noted that confinement provided by the soil improves the behavior of concrete piles.

Design and construction of concrete foundation components shall confirm to the requirements of ACI 318, Section 21.8, except as modified here in

Pole-type structures: Where construction employing posts or poles as columns embedded in earth or embedded concrete footings in the earth is used to resist lateral loads, depth of embedment required for posts or poles to resist seismic forces shall be determined by means of the design criteria as established in the foundation investigation report.

Foundation ties: Individual pile caps, drilled piers, or caissons shall be interconnected by ties. In addition, individual spread footings founded on soil defined in Chapter 20 as Site Class E or F shall be interconnected by ties. All ties shall have a design strength in tension or compression at least equal to a force equal to 10% of S_{DS} times the larger pile cap or column factored dead plus factored live load unless it is demonstrated that equivalent restraining will be provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils, very dense granular soils, or other approved means.

General pile design requirement: Piling shall be designed and constructed to withstand deformations from earthquake ground motions and structure response. Deformations shall include both free-field soils strains (without the structure) and deformations induced by lateral pile resistance to structure seismic forces, all as modified by soil–pile interaction.

Batter piles: Batter piles and their connections shall be capable of resisting forces and moments from the load combinations with overstrength factor of Section 12.4.3.2 or 12.14.3.2.2. Where vertical and batter piles act jointly to resist foundation forces as a group, these factors shall be distributed to the individual piles in accordance with the relative horizontal and vertical rigidities and the geometric distribution of the piles within the group.

Batter pile systems that are partially embedded have historically performed poorly under strong ground motions. Difficulties in examining fully embedded batter piles have led to uncertainties as to the extent of damage for this type of foundation. Batter piles are considered as limited ductile systems and should be designed using the special seismic load combinations.

6.3.26.5 SDC E Buildings

SEC E includes occupancy category I, II, and III structures located close to major active fault that is defined as a region with $S_1 \geq 0.75g$. Severe restrictions are placed on the use of certain structural systems, irregular structures and analysis methods. The design requirements are as follows:

1. All requirements of SDC A, B, C and D also apply to SDC E.
2. Sitting of a structure is prohibited where there is a known potential for an active fault that may cause rupture of the ground surface at the structure.

6.3.26.6 SDC F Buildings

SDC F includes occupancy category structures IV structures located close to major active faults (regions where $S_1 \geq 0.75g$). As in SDC E structures, severe restrictions are placed on the use of certain structural systems, irregular structures and analysis methods. The design requirements are as follows:

1. All requirements of SDC A, B, C, D, and E also apply to SDC F.
2. Single-story steel ordinary moment frames and intermediate moment frames are permitted to a height of 65 ft. where the dead load supported by and tributary to roof does not exceed 20 psf. In addition, the dead load of the exterior walls tributary to the moment frame shall not exceed 20 psf.
3. Steel intermediate moment frames are permitted in light-frame construction.

6.3.27 Analysis Procedures

A flow chart of permitted analysis procedures, developed from ASCE 7-05, Table 12.6-1 (Table 6.17 of this book), is shown in Figure 6.63.

The modal superposition method is a general procedure for linear analysis of the dynamic response of structures. In various forms, modal analysis has been widely used in the earthquake-resistant design of special structures such as very tall buildings, offshore drilling platforms, dams and nuclear power plants for a number of years; however its use has become more common for ordinary structures as well because of the advent of high-speed, desktop computers and the availability of relatively inexpensive structural analysis software capable of performing 3D modal analyses. When modal analysis is specified by the ASCE provisions, a 3D analysis generally is required except in the case of highly regular structures or structures with flexible diaphragms.

The ELF and the response spectrum procedures are both based on the approximation that the effects of yielding can be adequately accounted for by the linear analysis of the seismic-force-resisting system for the design spectrum, which is the elastic acceleration response spectrum reduced by the response modification factor, R. The effects of the horizontal component of ground motion perpendicular to the direction under consideration in the analysis, the vertical component of ground motion, and torsional motions of the structure are all considered in the same simplified

TABLE 6.17
Permitted Analytical Procedures

SDC	Structural Characteristics	ELF Analysis Section 12.8	Modal Response Spectrum Analysis Section 12.9	Seismic Response History Procedures Chapter 16
B and C	Occupancy category I or II buildings of light-framed construction not exceeding three stories in height	P	P	P
	Other occupancy category I or II buildings not exceeding two stories in height	P	P	P
	All other structures	P	P	P
D through F	Occupancy category I or II buildings of light-framed construction not exceeding three stories in height	P	P	P
	Other occupancy category I or II buildings not exceeding two stories in height	P	P	P
	Regular structures with $T < 3.5T_s$ and all structures of light-frame construction	P	P	P
	Irregular structures with $T < 3.5T_s$ and having only horizontal irregularities Type 2, 3, 4, or 5 of Table 12.2-1 of ASCE 7-05 or vertical irregularities Type 4, 5a, or 5b of Table 12.3-1 of ASCE 7-05	P	P	P
	All other structures	NP	P	P

Source: ASCE 7-05, Table 12.6-1.
Note: P, permitted; NP, not permitted.

approaches in the two procedures. The main difference between the two procedures lies in the distribution of the seismic lateral forces over the height of the building. In the modal analysis procedure, the distribution is based on properties of the natural vibration modes, which are determined from the mass and stiffness distribution. In the ELF procedure, the distribution is based on simplified formulas that are appropriate for regular structures. Otherwise, the two procedures are subject to the same limitations.

The simplifications inherent in the ELF procedure result in approximations that are likely to be inadequate if the lateral motions in two orthogonal directions and the torsional motion are strongly coupled. Such would be the case if the buildings were irregular in its plan configuration or if it had a regular plan, but its lower natural frequencies were nearly equal. The modal analysis method includes a general 3-D model that is more appropriate for the analysis of such structures.

The methods of modal analysis can be generalized further to model the effect of diaphragm flexibility, soil–structure interaction, etc. In the most general form, the idealization would take the form of a large number of mass points, each with six DOFs (three translational and three rotational) connected by generalized stiffness elements.

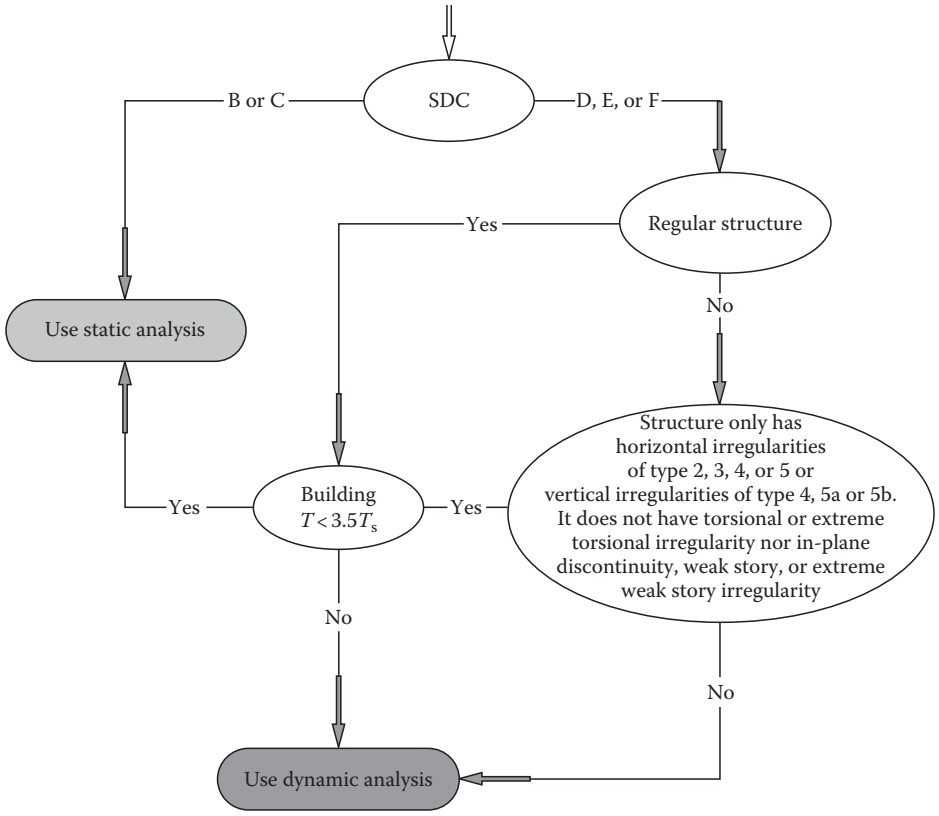


FIGURE 6.60 Permitted seismic analysis procedures (Source from ASCE 7-05, Table 12.6-1).

The ELF procedure and the response spectrum procedure are all likely to err systematically on the unsafe side if story strengths are distributed irregularly over height. This feature is likely to lead to the concentration of ductility demands in a few stories of the building. The nonlinear static (or the so-called pushover) procedure is a method to more accurately account for irregular strength distribution. However, it also has limitations and is not particularly applicable to tall structures or structures with relatively long fundamental periods of vibration.

The actual strength properties of the various components of a structure can be explicitly considered only by a nonlinear analysis of dynamic response by the direct integration of the coupled equations of motion. This method has been used extensively in earthquake research studies of inelastic structural response. If the two lateral motions and the torsional motion are expected to be essentially uncoupled, it would be sufficient to include only one DOF per floor, for motion in the direction along which the structure is being analyzed; otherwise at least three DOFs per floor, two translational and one torsional, should be included. It should be recognized that the results of a nonlinear response history analysis of such mathematical structural models are only as good as the models chosen to represent the structure vibrating at amplitudes of motion large enough to cause significant yielding during strong ground motions. Furthermore, reliable results can be achieved only by calculating the response to several ground motion-recorded accelerograms and/or simulated motions, and examining the statistics of response.

It is possible with presently available computer programs to perform a 2D and/or a 3D inelastic analyses of reasonably simple structures. The intent of such analyses could be to estimate the sequence in which components become inelastic and to indicate those components requiring strength adjustments so as to remain within the required ductility limits. It should be emphasized

that with the present state of the art in analysis, there is not one method that can be applied to all types of structures. Further, it should be kept in mind that the reliability of the analytical results is sensitive to

1. The number and appropriateness of the input motion records
2. The practical limitations of mathematical modeling including interacting effects of inelastic elements
3. The nonlinear solution algorithms
4. The assumed hysteretic behavior of members

Because of these sensitivities and limitations, the maximum base shear produced in an inelastic analysis is mandated not be less than 80% of that required by ELF.

The least rigorous analytical procedure that may be used in determining the design seismic forces and deformations in structures depends on the SDC of the building and the structural characteristics (in particular, regularity). Except for structures assigned to SDC A, the ELF procedure is the minimum level of analysis. The modal analysis procedure adequately addresses vertical irregularities of stiffness, mass, or geometry, and may be used for the analysis of

1. Structures with irregular mass and stiffness properties in which case the simple equations for the vertical distribution of lateral forces may lead to erroneous results
2. Structures (regular or irregular) in which the lateral motions in two orthogonal directions and the torsional motion are strong coupled
3. Structures with the irregular distribution of story strengths leading to the possible concentration of ductility demand in a few stories of the building

Many of the standard procedures available today (2011) for the analysis of forces and deformations in structures subjected to earthquake ground motion are listed below in order of increasing rigor and expected accuracy:

1. ELF procedure
2. Response spectrum (modal analysis) procedure
3. Linear response history procedure
4. Nonlinear static procedure, involving the incremental application of a pattern of lateral forces and the adjustment of the structural model to account for progressive yielding under load applications (pushover analysis)
5. Nonlinear response history procedure, involving the step-by-step integration of the coupled equations of motion

Observe that each procedure becomes more rigorous if effects of soil–structure interaction are considered.

7 Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-10

PREVIEW

Similar to the 2005 AISC Seismic Provisions, which is referenced in the 2007 IBC, the 2010 edition of AISC 341 is expected to be referenced in the next publication of IBC. Therefore, the seismic design of all steel and composite buildings in the United States will be governed by this document, allowing engineers to develop their designs in a consistent fashion, no matter what the jurisdiction. This should result in improved steel and composite construction throughout the United States and other countries around the world that adopt this standard. Before discussing the AISC Seismic Provisions, it is perhaps instructive to recall the events that occurred right after the Northridge earthquake that have resulted in its extensive revisions.

The 1994 Northridge earthquake in Southern California resulted in an unprecedented level of interest in the seismic performance of steel frame structures. This concern led to major research and development programs that resulted in significant modifications to the U.S. seismic design provisions for steel structures. In addition to updating the requirements for systems used in current design practice, in 2005, the AISC Seismic Provisions incorporated two new systems: steel plate shear walls and buckling-restrained braced frames. It should be noted that AISC Seismic Provisions have been developed for use in conjunction with the AISC Steel Construction Manual, now in its 14th edition.

Until quite recently, major failures in steel components and buildings were rare. Five steel buildings collapsed or were fatally damaged in Mexico City during the 1985 Michoacán earthquake. This damage was the result of a large torsion irregularity, a resonance condition between the soft soil and the building, and, perhaps, poor fabrication of the built-up square columns. Other typical damage included buckled braces, failure of a few connections, and damage to infills and attached cladding. Loss of masonry cladding from entire sides of a building was observed.

Prior to the 1994 Northridge, California, earthquake, the steel moment frame was considered to be an invincible structural element to resist earthquake because of its excellent ductility. However, during this earthquake, over 200 buildings experienced fractured, beam–column or column–base plate connections. The reasons for this poor performance are complex. One significant factor appears to be lack of quality control of the entire welding process, in combination with the use of weld filler that has almost no notch toughness. Other factors contributed to this poor behavior, such as relatively thick plates of the column and beam flanges, the stiffness and strength of the panel zones, tri-axial stress effects, high confinement of the joints, poor welding cool down, and conditions allowing hydrogen embrittlement. The increased beam depths used in designs as compared to those tested, also played an important role.

Current design procedures recognize that structures cannot economically be designed to elastically resist the effects of earthquakes. Therefore, inelastic response is permitted to occur if an earthquake as large as anticipated by the design procedures occurs. The role of the designer is to ensure that this inelasticity can develop in a stable manner without the risk of structural collapse.

In the context of structural engineering, a “ductile” material is one that is capable of undergoing large inelastic deformations without losing its strength. “Ductility” maybe defined as “the ability of

a material to deform plastically without fracture." "Brittleness," on the other hand, is the opposite of ductility. It refers to the "quality of a material that leads to crack propagation without plastic deformations." In that perspective, structural steel is the most ductile of the widely used engineering materials.

However, there are many situations in which an explicit approach to the design of ductile steel structures is necessary because the inherent material ductility alone is not sufficient to provide the desired ultimate performance. For example, in nearly all buildings designed today, survival in large earthquakes depends directly on the ability of their faming system to dissipate energy hysteretically while undergoing large inelastic (i.e., plastic) deformations. To achieve this ductile response, one must recognize and avoid conditions that may lead to brittle failures of connections and adopt appropriate design strategies to allow for stable and reliable hysteretic energy-dissipation mechanisms. As stated previously, numerous steel structures suffered damage during the Richter magnitude of 6.8 Northridge (Los Angeles) earthquake of January 17, 1994, and the Richter magnitude 7.2 Kobe (Japan) earthquakes that occurred, coincidentally, exactly 1 year later (January 17, 1995). Both earthquakes occurred in highly developed urban areas in nations known for their leadership in earthquake engineering, and both confirmed that material ductility alone is not a guarantee of ductile structural behavior when steel components and connections can fail in a brittle manner. In particular, these earthquakes raised important concerns regarding the ductile behavior of some standard beam-to-column moment connections. In the years since, many new detailing regulations have been introduced to ensure ductile response during earthquakes. However, the road to achieve this goal has been long and bumpy at best.

The design of steel structures for ductile response requires the following:

1. Material ductility
2. Cross section and member ductility
3. Configuration of connections to prevent brittle failure
4. Prescribing a hierarchy of yielding on the structure to ensure a desirable failure mode

Therefore, to gain a good understanding of the philosophy of seismic design, we must necessarily study the plastic behavior of structural systems and the advantages of imposing a well-defined hierarchy of yielding.

In seismic design, the engineer first acknowledges that inelastic action is inevitable during severe earthquakes. The designer then dictates where inelastic response should occur. Such zones of possible inelastic action are selected to be regions where large inelastic deformations can develop without significant loss of strength; these regions are detailed to suppress premature undesirable failures modes, such as local buckling or member instability. Then, one eliminates the likelihood of inelastic action or failure elsewhere in the structure by making the capacities of adjoining structural members greater than that needed to reach the maximum capacity of the inelastic zone.

The proverbial example to explain this concept is the capacity designed chain (Figure 7.1). In this chain, one link is designed to absorb a large amount of plastic energy in a stable manner prior to failure (e.g., link 4). Therefore, the other links (e.g., 1, 2, 3, 5, 6, and 7) can be designed without concern for plastic deformation, provided their capacities exceed the maximum capacity of the plastic link, thus avoiding the need for special detailing in all but one link.

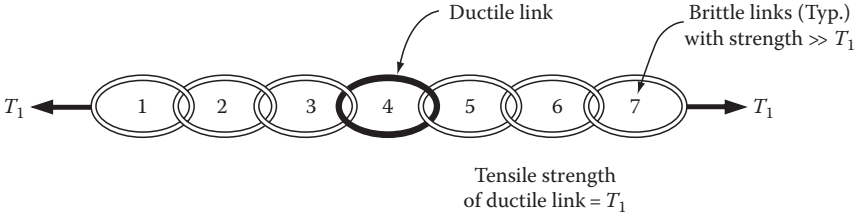


FIGURE 7.1 Chain with ductile and brittle links.

In capacity design, the strength of the seismic load resisting system (SLRS) is proportioned such that inelastic behavior occurs in specific designated locations that are adequately detailed to accommodate this behavior. When these elements yield, they limit the force that can be transmitted to other elements, effectively shielding them from overstress and allowing them to resist design earthquake forces while remaining elastic. This practice permits the elements that are not expected to yield or experience inelastic behavior to be designed and specified without rigorous detailing practices intended to provide ductile behavior.

Many of the provisions contained in AISC 341 are intended to realize a capacity design of the SLRSs. As an example, special moment frames (SMFs) must be proportioned such that inelastic behavior is accommodated through plastic hinging within the spans of the beams in moment frames, rather than the columns. As another example, special concentrically braced frames (SCBFs) must be designed such that the connections and columns are stronger than the braces so that inelastic behavior is accommodated through yielding and/or buckling of the braces.

The purpose of this chapter is to anatomize the document, *AISC seismic provisions for structural steel buildings (ANSI/AISC 341-10).*

7.1 AISC 341-10 SEISMIC PROVISIONS, OVERVIEW

7.1.1 General Requirements

In general, no special limitations are placed on which base materials are deemed to be acceptable for seismic applications. However, the specified minimum yield stress, F_y, is limited to 50 ksi except for columns where inelastic behavior is not expected and for lower ductility systems (OMF and OCBF) that allow the use of 55-ksi material. In addition, it is required that any member of the seismic system that has thick elements (2 in. or thicker for plate materials and 1½ in. or thicker for rolled shapes), have a minimum level of Charpy V-Notch (CVN) toughness to help ensure ductile behavior of these members. Perhaps the most important part of this section of AISC 341 is the requirement to consider the expected yield strength and expected tensile strength in the determination of the required strength. This is necessary because in seismic design one of the key goals is to force the primary inelastic action in the frame to certain key elements that are specifically designed and detailed for this purpose. It is therefore important to have the best estimate possible of the actual yield and tensile strengths (as opposed to the ASTM specified minimum values) of all members in the system. For base materials, the provision specifies a term, R_y, that when multiplied by the nominal yield strength, F_y, results in the expected yield strength of the material. A second term, R_t, multiplied by the minimum nominal tensile strength, F_u, results in the expected tensile strength of the material. Other sections in the provisions define when the R_y and R_t terms are to be used in determining the required strength of the members.

Different steel members conforming to the same specification and grade of steel can have significantly different yield and tensile strengths. This is because each heat of steel that is produced has a slightly different chemistry and also because the yield strength of a steel member also depends on the member's geometry, the thickness of its webs and flanges, and member's thermal history, and the amount of cold working that occurs as the member is formed and then fabricated. The minimum specified values that are associated with standard specifications such as ASTM A992 and A572 are, as the name implies, minimum acceptable values. Most structural steel conforming to one of these specifications will have strengths that exceed these minimums. Although the possible variation of strength varies from one grade of steel to another, depending in part on the limitations contained in the standard specification, coefficients of variation on the order of 20% of the minimum specified value are not uncommon. This means that more than half of the structural steel used on projects is likely to have a strength that is 20% higher than the minimum, and some will have a strength that is nearly 40% higher than the minimum.

While additional strength is not of concern in design for most types of load conditions, it can be a problem in seismic design. This is because in seismic design, particularly for structural systems designed as special or intermediate, the design is based on an assumption that yielding and other inelastic behavior will occur in specific portions of the structure. For example, in braced-frame structures, it is desirable that inelastic behavior is accommodated in the form of yielding of tensile braces rather than failure of bolted connections. Therefore, AISC 341 requires that connections in such structures must be designed so that the connection is able to develop the yield strength of the braces. In order to do this, the designer must have an understanding of how strong the member is likely to be.

AISC 341 as stated previously, uses the concept of expected strength to provide designers with an estimate of the probable effects of steel strength variation on the strength of individual members. The specification defines a coefficient, R_y, that is a measure of how much stronger, on average, typical steel section are than the minimum-specified value. The quantity R_yF_y is termed the expected strength and is used to estimate the maximum force that a member is likely to be able to develop before it yields, so that other members and connections that are not supposed to yield can be designed with a strength that exceeds this force. In some provisions, the expected strength includes an additional factor that is intended to account for the effects of strain hardening of the steel, which further elevates the effective yield strength.

7.1.2 Member and Connection Design

Section D, of AISC 341-10 General Member and Connection Design Requirements, addresses the design of connections, joints, and fasteners in the SLRS. The section begins with a general statement that all connections should be configured so that a ductile limit state controls the strength of the connection. All bolts are to be pre-tensioned, high strength, with faying surfaces prepared for Class A or better slip-critical joints. But, it is permitted to design the bolts in bearing. This apparent contradiction between design approach and surface preparation is presented because it is felt that avoidance of bolt slip in smaller earthquakes would be desirable, but that in the design event, the bolts would go into bearing. Another important requirement is that bolts and welds are not allowed to share the same force in a joint. This is more stringent than for typical steel design because of the expectation that the joints will be subjected to yield level forces and that the difference in stiffness between welds and bolts may not allow them to completely share the load. Standard holes are to be used, except short-slotted holes are allowed when placed perpendicular to the line of force, again to limit the chance for excessive deformation due to bolt slip. For brace diagonals, oversized (OVS) holes may also be used, if they are in one ply of slip-critical joints.

Section D also addresses the requirements for welds in the SLRS. All such welds must be made with filler metals that have a minimum CVN toughness of 20 ft-lb at minus 20°F as demonstrated by AWS classification or manufacturer certification. Observe this change, as first incorporated in the 2005 AISC is different from the previous temperature of 0°F. To ensure proper performance at operating temperatures, additional toughness requirements are placed on the demand critical complete-joint-penetration (CJP) groove welds in various systems (e.g., welds of beam flanges to columns, column splices, and welds of beam webs to columns flanges). The additional requirement is that a CVN toughness of 40 ft-lb at 70°F be provided for a wide range of test conditions. The term "protected zone" defined in the Seismic Provisions alerts the engineer that discontinuities in the members of the SLRS must be avoided to limit the chance for premature, brittle fracture of the members. Specific detailing requirements for continuity plates are also provided in this section.

The majority of the remainder of this section focuses on column design. Column demands are limited to ensure that the potential for column failure is minimized, due to the potentially catastrophic effects of such a failure. Similar limitations are also placed on column splices and column bases. In addition, the splices in columns that are not part of the SLRS also have special requirements. This is the only reference to members that are not part of the SLRS in the document and

is provided because studies indicated that continuity of these columns significantly improved the seismic performance of steel frames in severe seismic events. The final paragraphs of Section D address the design of steel H-piles used as part of building foundations.

7.1.3 Moment Frames

Sections E1 through E4 of the provisions address the requirements for the design of moment resisting frame buildings. Special Moment Frames, SMFs presented in Section E3, are intended to have the most ductile response. Consequently they have been assigned the highest *R* factor equal to 8. Because of the damage caused in the Northridge earthquake, SMF connections must be demonstrated to be capable of performing through a tested interstory drift of 0.04 rad (approximately 6 in. in a 13 ft floor-to-floor height) based on a standard cyclic testing protocol. Demonstration of this capacity can be accomplished by one of the three following means:

1. Using a connection prequalified for use in an SMF in accordance with ANSI/AISC 358.
2. Using a connection prequalified for use as an SMF in accordance with Section J of the Provisions. This section establishes minimum requirements for prequalification of SMF, IMF, and link-to-column connections in eccentrically braced frames (EBFs). A Connection Prequalification Review Panel (CPRP) is to be established that will review all test results and other data to assure that the connection can meet the required interstory drift level.
3. Providing qualifying test results in accordance with the Provisions. The provision addresses how such tests are to be conducted and demonstrated to be adequate for the proposed design. This section also specifies that the test subassemblage match the prototype as closely as practical and defines the essential test variables that must be considered such that the connection design, detailing, construction features, and material properties are consistent with that proposed for the building. The test results can be taken from testes reported in the literature, or from tests performed specifically for the project under consideration.

In addition to having deformation capacity demonstrated by testing, the shear connection of SMFs must be designed for the gravity shear force plus the shear generated by the formation of plastic hinges at each end of the beam.

The design of the panel zone capacity is intended to be consistent with that provided in the qualifying connection tests. In addition, the panel zone must have an expected strength that is adequate to provide an approximately "balanced" yielding condition between the beams and the panel zone, since it is believed that this will result in good performance. Another important consideration for SMF design is the so-called strong column–week beam provision. This provision is provided to prevent weak story conditions from occurring in this system, by requiring that the design confirm that the moment capacity of the columns exceed that of the beam framing into the SMF connections. Exceptions are provided for lightly loaded columns and conditions where individual connections that do not meet the strong column–week beam concept contribute a relatively minor amount to the capacity of the moment frame at the level under consideration.

7.1.4 Stability of Beams and Columns

Section E3-4b of the Provisions addresses the out-of-plane stability of the beams, columns, and connections in SMF systems. Provision of this stability is critical to such systems that are expected to undergo significant inelastic response in the design earthquake. Both strength and stiffness requirements are provided. For beams, the provisions require that bracing be provided near all expected plastic hinge locations. The design force to be considered is 6% of the expected beam flange capacity.

The final requirements for SMF systems is that the column splices be designed to develop the full flexural capacity of the smaller column, and that the shear connection be strong enough to develop a plastic hinge at one end of the column. This stringent requirements on column splices resulted from extensive analytical studies demonstrating that large moments, on the order of the yield capacity of the columns, can be developed over the height of the columns in severe earthquakes.

7.1.5 Intermediate Moment Frames

The requirements for IMF systems are presented in Section E2. Similar to SMF, these systems must have their moment connections qualified by connection testing. The qualifying interstory drift limit for these connections is reduced to 0.02 rad (half as much as for SMF) to reflect the more limited ductility demands expected to be placed on these systems. It should be noted that ASCE 7-05/10 Seismic Design Requirements severely limit the use of these systems in the higher seismic design categories (SDCs). Other than the requirement for connection qualification by testing and more restrictive lateral bracing requirements, the design of these systems is generally performed to be in accordance with the AISC Specification for Structural Steel Buildings.

OMF systems are permitted to be designed without connection testing. The connection must have a strength of 1.1 times the expected strength of the connected members, in an effort to force the inelastic action into the members and away from the connections. This section provides a number of connection detailing requirements to promote ductile performance of the connections. Specific requirements are provided for continuity plates, weld backing and run-of tabs, weld access holes, etc. OMFs are typically used in light metal buildings and small building applications in the higher seismic design applications.

7.1.6 Special Truss Moment Frames

The design requirements for STMF systems are presented in Section E4. These provisions define a special segment of the truss that is intended to be the location of the inelastic behavior in the system. All other members in the frame are designed to be able to develop the capacity of the special segment. This design concept is parallel to EBF design, where all other members are designed to be strong enough to force yielding into the link beams. Both Vierendeel and X-braced special segment panels are allowed. The requirements also provide lateral bracing stipulations similar to those required for SMF systems to ensure out-of-plane stability. As stated previously in Chapter 1 of this book, STMFs incorporate horizontal truss elements rigidly attached to columns and resist lateral displacement by traditional frame action. The limits on the horizontal truss span and depth are set by the tested range.

The special segment is the yield mechanism of the system and the frame elements outside of the special segment are designed and detailed to remain nominally elastic in order to support this yield mechanism. The horizontal truss elements are designed and constructed with special segment that is usually placed near mid-span of the truss element in order to mitigate the gravity shear effects on the special segment. The special segment can be designed and detailed utilizing either X-braced panel segments or Vierendeel panel segments, as shown in Figure 1.14. The properly designed and detailed special segment withstands large inelastic deformations and dissipates energy through a combination of flexural yielding of the chord and flexural or axial yielding of the web members, depending on which type of special segment is used.

7.1.6.1 Special Concentric Braced Frames

SCBF design requirements are presented in Section F2. The design concept for SCBF systems is that prior to connection failure, the diagonal braces should buckle and dissipate energy. Special provisions are included to improve the ductility of the system. For example, the orientation of bracing in all frame lines must be such that there is approximately the same number of braces in compression

and tension. In addition, there are strict limits on the width–thickness ratios and stitching requirements for built-up brace members. Bracing connections in SCBF must be designed to develop the full tensile capacity of the members or the maximum force that can be delivered to the brace by the rest of the system. Full flexural strength must also be provided in the bracing connections, unless the connection includes a gusset plate that will yield in such a manner to allow the ductile post-buckling behavior of the braces. Special limitations are provided for V-type and inverted V-type bracing to reflect the potentially undesirable behavior of these bracing configurations. K-braced frame configurations are not permitted in SCBFs. Column splices in SCBF are required to develop a shear capacity of approximately 50% of the member capacity to reflect the substantial demands on these elements when subjected to severe earthquake ground motions.

Like OMFs, OCBF systems (Section F1) have severely limited applications in high SDCs due to their limited ductility. The provisions also place limitation on the use of V-type and inverted V-type bracing. Connections in OCBFs are designed including the amplified seismic load. The previous requirement to design the members in OCBFs for the amplified seismic load was removed when the *R* factor was reduced in ASCE 7-05.

7.1.7 Eccentrically Braced Frames

EBF systems are addressed in Section F3. The basic intent of EBF design is to seek for a system where the diagonal braces, columns, and beams outside the link beams remain essentially elastic under the forces that can be generated by the fully yielded and strain-hardened link beams. Because of their importance to the performance of the system, a good deal of this section is devoted to the design of the link beam elements. There are strict limits placed on width–thickness ratios for these elements to ensure proper inelastic performance. The link beams can be designed to yield in shear or flexure, or a combination of both. Laboratory testing has demonstrated that properly designed shear yielding links can undergo a link rotation angle of 0.08 rad. Such links are provided with closely spaced web stiffeners to delay web buckling. Significant strain hardening (on the order of 50% of the nominal shear yielding capacity of the link section) develops in such properly braced links. This strain hardening must be considered in the design of the rest of the frame members. Moment yielding links are designed to undergo a link rotation angle of 0.02 rad, which is consistent with SMF systems. Interpolation is allowed for links with a length that results in a combination of shear and flexural yielding. Web stiffening requirements are also modified for flexural yielding links. Because of the high local deformation demands, link-to-column connections must be demonstrated by testing similar to SMFs. An exception is provided if there is substantial reinforcement of the connection that would preclude inelastic behavior in the connection welds. As with SMF and STMF systems, there is significant lateral stability bracing requirements for EBF systems. Lateral bracing is required at both ends of all link members and along the remainder of the beam to ensure that stability is provided. As noted earlier, the design of other members in the system and all the connections between the members, are required to have a capacity that is sufficient to develop the fully strain hardened link beams. Column capacities are not required to develop the simultaneous yielding and stain hardening of all links in the system.

7.1.8 Buckling-Restrained Braced Frames

It is interesting to note that the 2006 edition of International Building Code (IBC) was the first code to include the BRBF system. Although the system had been available for a decade or more, procuring building department approval often was difficult in the past because of its absence from adopted codes. BRBFs can now be used readily in the more demanding SDCs (D, E, and F).

The brace member itself makes this system unique. It consists of two main parts: the core and its casing. The casing, typically a grout-filled HSS, is axially decoupled from the core and restrains

it from buckling when loaded in compression. Ductile compression yielding replaces non-ductile buckling as an energy dissipating mechanism.

Because the effective length of the brace is essentially zero and buckling is not a consideration, the brace has similar tensile and compressive strengths. Conventional braces tend to be much stronger in tension than compression because they must be sized to prevent buckling. The frame elements surrounding the brace are designed to remain elastic when subjected to the maximum forces that the brace can generate. Because the steel cores of these braces do not contain the excess material normally required to prevent buckling, the frame beams, columns and connections are designed for much smaller forces.

Designing, detailing, and erecting the structural steel for a building that utilizes the BRBF system are similar to other steel buildings, except the project team has an extra player: the brace supplier.

Unlike the more common braces used in regions of high seismicity, buckling-restrained braces are proprietary members and are manufactured by the supplier. They are not fabricated with the rest of the steel by the fabricator. The brace supplier designs, details, and fabricates the braces based on strength and stiffness requirements provided by the Engineer of Record. The brace detailer and the steel detailer, who details all the steel but the braces, work together while preparing shop drawings so that the members fit together properly in the field.

Early in the design process, a structural model is used to determine approximate brace forces and their required stiffness. This information, along with the basic frame geometries, is provided to the supplier. The supplier's brace designer requires this preliminary design information to size the casings and compute stiffness modification and overstrength factors. The casing sizes are useful for coordinating wall thicknesses, door locations, etc., with the architect. The stiffness modification factor indicates approximately how many times stiffer the real braces will be, compared to the modeled braces.

The modeled braces typically have centerline-to-centerline lengths and constant cross sections. The real braces will be shorter and hence stiffer due to the geometry of connections. The overstrength factor accounts for higher than specified yield strengths and strain hardening. The overstrength factor multiplied by the axial strength results in the adjusted brace strength. Depending on the braces sizes and frame geometries, the stiffness modification and overstrength factors may differ for each brace requiring design iterations.

When determining the required stiffness it is important to remember that the code allows the importance factor, I, and redundancy factor, ρ, to be equal to unity in determining the seismic forces. Also, if the computed fundamental period of the structure is higher than the approximate fundamental period, T_a, a higher period may be used for determining the drift level forces. However, the ultimate, axial brace forces, P_u, provided to the brace supplier should include I and ρ as required by the code.

AISC Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-10) requires that the brace designs be based on test results from qualifying cyclic tests. It describes the procedures for testing, including test reporting requirements and acceptance criteria. The provisions specify, among other things, the materials and manufacturing procedures necessary to ensure that project braces are produced in a manner consistent with the test-qualified braces. Observe that the Seismic Provisions define the steel core and the elements that connect the steel core to the frame as protected zones. Welds, bolts, screws and shot-in attachments are prohibited within these zones.

Section F4 of AISC 341-10 addresses the BRBF. The key feature of this system is that it relies on a brace element that is restrained from overall member buckling, thereby significantly increasing the energy dissipation of the system over that of a traditional concentric braced frame (CBF) system.

The requirements for testing of the brace elements are specified Section J. As with EBF systems, the provisions intend that the connections and other members in the BRBF system remain essentially elastic at the full capacity of the bracing elements. Connection design requirements recognize the fact that the braces are likely to be stronger in compression than tension. It should also be noted that because of the better energy dissipation characteristics of the bracing elements in a BRBF, the bracing configuration limitations are not as strict as those imposed on SCBF frames.

7.1.9 Special Plate Shear Walls

Section F5 presents design requirements for Special Plate Shear Walls, SPSW, in which the anticipated performance is controlled by the web members. Since the design of SPSW systems is based on the use of relatively thin plates, tension field action (as in a deep plate girder) develops in the web members under lateral loading. The other elements of the shear walls such as the boundary members are designed to remain essentially elastic for the capacity of the webs. Limitations on configurations, width–thickness ratios, etc., are provided to be consistent with the successful test results.

SPSWs, sometimes referred to as steel plate shear walls, consist of columns, beams, and a steel plate element that infills the boundaries created by the columns and beams. The behavior of special steel plate shear walls is analogous to a vertically cantilevered steel plate girder with the columns acting as the plate girder flanges, the beams acting as the intermediate stiffeners, and the steel plate acting as the web of the plate girder. Figure 7.2 shows an elevation of a SPSW. SPSWs can be used as independent planar walls, coupled walls linking two planar walls with a coupling beam (which allows openings to be placed through the shear wall), or interconnected walls, creating a three-dimensional system, which is beneficial for taller structures. SPSWs can also be used as part of a dual system, utilizing moment frames designed to carry a portion of a prescribed seismic forces. The steel plate acts as the primary shear resistance of the shear wall and may be stiffened or unstiffened. When the web plate is designed as stiffened, it is designed to prevent buckling of the plate prior to shear yielding, and when it is designed as unstiffened, it is expected to buckle in order to utilize tension-field action. Test results have indicated that SPSWs provide a stiff and ductile SLRS, capable of dissipating significant amounts of energy.

7.1.10 Composite Structural Steel and Reinforced Concrete Systems

Section G and H of AISC Seismic Provisions address the design of composite moment frames, and composite braced shear wall systems of structural steel and reinforced concrete.

The available research demonstrates that properly detailed composite members and connections can perform reliably when subjected to seismic ground motions. However, there is at present limited experience with composite building systems subjected to extreme seismic loads, and many of the recommendations are necessarily of a conservative and/or qualitative nature. The design and construction of composite elements and systems continues to evolve in practice. With further experience and research, it is expected that these Provisions will be better quantified, refined and expanded.

It is generally anticipated that the overall behavior of composite systems will be similar to that for the counterpart structural steel systems or reinforced concrete systems and that inelastic

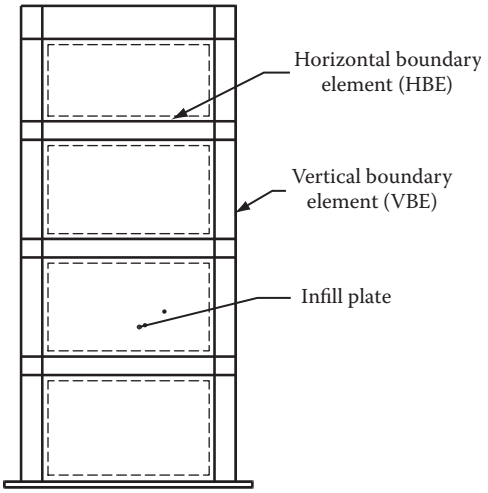


FIGURE 7.2 Special steel plate shear wall.

deformations will occur in conventional ways, such as flexural yielding of beams in fully restrained (FR) moment frames or axial yielding and/or buckling of braces in braced frames. However, differential stiffness between steel and concrete elements is more significant in the calculation of internal forces and deformations of composite systems than for structural steel or reinforced concrete only systems. When composite systems have both ductile and nonductile elements, the relative stiffness of each should be properly modeled. The ductile elements may be allowed to deform inelastically while the nonductile elements are designed to remain nominally elastic.

The use of composite connections often simplifies some of the special challenges associated with traditional steel and concrete construction. For example, compared to structural steel, composite connections often avoid or minimize the use of field welding, and compared to reinforced concrete, there are fewer instances where anchorage and development of primary beam reinforcement are a problem. In most composite structures built to date, engineers have designed connections using basic mechanics, equilibrium, existing standards for steel and concrete construction, test data and good judgments. The provisions are intended to help standardize and improve design practice by establishing basic behavioral assumptions for developing design models.

7.2 AISC 341-10, DETAILED DISCUSSION

AISC 341-10, commonly referred to as AISC Seismic, is used in conjunction with ANSI/AISC 360-05/10 for the design of steel and composite seismic force resisting systems (SFRSs).

Seismic forces for steel buildings in SDC B and C (but not composite systems) can be determined using an $R = 3$ system whenever the selected SFRS is designed in conformance with the requirements in SEI/ASCE 7-05/10 SFRSs in such structures must be proportioned with adequate strength and stiffness to resist design seismic forces but need not conform to any special detailing criteria or configuration limitations beyond that required by AISC 360. The code permits the use of this design approach in SDCs B and C.

This design approach is often advantageous in that it can allow the engineer greater flexibility in configuring the structure and usually results in a lower-cost structure. There are several reasons the cost of such structures can be substantially less than those designed in conformance with AISC 341. First, structures designed in this manner can often use lighter sections. This is accomplished by avoiding the special compactness criteria of AISC 341, avoiding the need to design columns to resist axial forces amplified by the Ω_0 coefficient, and avoiding the need to proportion moment frames with strong column–weak beam configurations. In addition, the connection costs of structures designed in this manner can be significantly less because connections need only be proportioned to withstand the design seismic forces, rather than to develop the expected strength of connected members.

It should be recognized, though, that structures designed in this manner may not perform as well as structures designed in conformance with AISC 341, if they actually experience a strong earthquake. $R = 3$ systems are generally not desirable for use in Occupancy Category IV facilities, where post-disaster operability is desirable. It is also important to recognize that some SFRS, covered by AISC 341 have R factors lower than 3, notably cantilevered column systems and ordinary composite moment frames. If one of these structural systems is used, all of the applicable detailing requirements specified by SEI/ASCE 7 and AISC 341 should be complied with, even though a low value of R is used.

The topic of *AISC Seismic Provisions for Structural Steel Buildings (ANSI/ASCE 341-10)* is the design and construction of structural steel and composite building systems for high-seismic applications. It is a separate consensus standard intended to provide guidance to engineers engaged in the design of building structures in area of high-seismicity. On the other hand, the *AISC Specifications for Structural Steel Buildings (AISC 360-05/10)* is intended to cover common design criteria. It is not meant to cover special problems encountered in the inelastic design of either structural steel or composite buildings consisting of a combination of steel and reinforced concrete components.

7.2.1 Moment Frame Systems

The principal advantage of moment frame structures is that they do not have structural walls or vertically oriented diagonal braces. They therefore provide architectural freedom in design, permitting open bays and unobstructed view lines. The tradeoff for these benefits is that moment frames can be more costly to construct than braced frame or shear wall structures. The added cost results from the use of heavier sections in the moment-resisting frames, requiring increased steel usage and more labor intensive connections than is common in braced structures.

Once a steel moment frame solution is selected for a project, designers may be able to choose from several types, including SMFs, IMFs, ordinary moment frames (OMFs), and moment frames not specifically designed for seismic resistance.

Moment frames not specifically detailed for seismic resistance have no special detailing criteria and need only comply with the strength and drift limits of ASCE 7 and the design requirements of AISC 360. These frames are not permitted as seismic force-resisting systems in SDCs D, E, or F. OMFs are permitted in light, single-story structures and low-rise residential structures in all SDCs and are permitted without restriction in SDCs A, B, and C. IMFs are permitted for structures up to 35 ft in height in SDC D, and for light, single-story structures in SDCs E and F. IMFs are permitted without restriction in SDCs A, B, and C, and are permitted in structures 35 ft or less in height in SDC D. SMFs are permitted without restriction in all SDCs and are required in SDCs D, E, and F for structures exceeding 160 ft in height. For structures that meet certain regularity criteria, the requirement to incorporate steel SMFs is triggered at a height of 240 ft.

In Dual System, SMFs capable of providing at least 25% of the required lateral strength are used in combination with shear walls or braced frames. From frames not detailed for seismic resistance to OMFs, IMFs, and steel SMFs, the seismic provisions of ASCE 7 require successively less strength. However, added level of detailing is required for the better performance of the system.

The primary factors affecting SMF member size selection are the need to control design drifts below permissible levels and the need to proportion structures to comply with the strong-column/weak-beam criteria. The use of deep section columns (W24s, W36s, and built-up box sections) is an economical choice that permits both drift control and strong-column/weak-beam requirements. However, those with lighter weights may be susceptible to undesirable local and lateral-torsional buckling. It should be noted that most connections prequalified for use in steel SMFs have limits on beam span to depth ratio that prevent use of excessively short bays.

Even in regions of very high seismic risk, like California and Alaska, severe earthquakes are rare events, affecting typical building sites at average intervals of hundreds of years. Given this, it is economically impractical to design structures to resist such severe but rare earthquakes without damage. Instead, the building codes have adopted a design philosophy intended to provide safety by avoiding earthquake-induced collapse in severe events, while permitting extensive structural and nonstructural damage.

Inelastic behavior in steel SMF structures is intended to be accommodated through the formation of plastic hinges at beam–column joints and column bases. Plastic hinges form through flexural yielding of beams and shear yielding of panel zones.

Beam behavior: It is expected that beams will undergo large inelastic rotation at targeted plastic hinge locations, which might be at the ends of beams or at deliberately weakened portions of the beams with reduced beam section designs. Failure modes can include excessive local buckling and lateral-torsional buckling. Each mode by itself, or by combination of both, leads to a continuous decrease in strength and stiffness.

Beam-to-column connections: The connections must be capable of transferring the moment and shear forces that can be developed in the beam. As a result of material overstrength, and strain hardening effects, these moment and shear forces can be substantially larger than the design forces

obtained from analysis, using code-specified loads. Depending on the type of connection used, this might trigger any of the following failure modes:

- Fracture in or around welds
- Fracture in highly strained base material
- Fractures at weld access holes
- Net section fracture at bolt holes, shearing and tensile failure of bolts, bolt bearing and block shear failures

Joint panel zone behavior: The joint panel zone resists significant shear forces from the beams framing into a column. Acting as part of the column, it can also be subjected to significant compressive stresses. Potential failure modes include shear buckling and, if doubler plates are used to reinforce the panel zone, fracture at welds. Failure modes associated with the direct transfer of forces from the beam flange to the column can include column flange bending, web crippling and web buckling.

Column behavior: The intention is to keep inelastic deformations out of columns to minimize potential formation of single-story mechanisms (Figure 7.3). Regardless, many columns designed in accordance with the strong-column/weak-beam requirements might experience significant inelastic rotations in a major seismic event. Therefore, excessive local buckling and lateral-torsional buckling are potential failure modes, in addition to basic flexural buckling of columns.

Column splices: Failure modes at column splices are similar to those for beam-to-column connections. Failure of column splices will not only reduce bending and tension resistance, it will also reduce the ability of the column to transfer shear forces. Since non-frame gravity columns can experience substantial lateral deformations, column splices in these columns also can be subject to such failures.

Column bases: Failure modes depend on the connection between the column and the foundation. They include anchorage stretching or pull-out, fracture in base plates or in column-to-base plate connections, and/or excessive local and lateral torsional buckling if inelastic deformations are concentrated in the region above the base connection.

P-delta effects: Amplification of internal forces and lateral displacements, known as the P-delta effect, occurs when a structure is subjected to gravity loads and lateral sidesway. This effect reduces frame lateral resistance and stiffness.

Sidesway collapse: Frame collapse can occur when the effective story shear due to inertial forces and P-delta effects exceeds the story shear resistance.

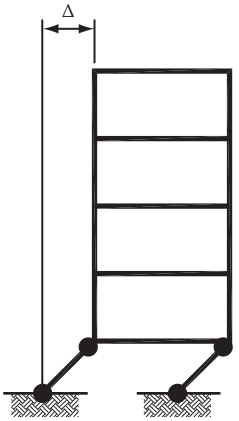


FIGURE 7.3 Single-story sway mechanism.

Lessons learned from recent earthquakes: SMFs are one of the few select systems that U.S. building codes permit without restriction in buildings exceeding 160 ft in height. They are allowed even in the most critical occupancies and in areas mapped as having the highest ground motions. Beams, columns, and beam–column connections in steel SMFs are proportioned and detailed to resist flexural, axial, and shearing actions that result as a building sways through multiple inelastic displacement cycles during strong earthquake ground shaking. Special proportioning and detailing requirements are therefore essential in resisting strong earthquake shaking with substantial inelastic behavior. These moment-resisting frames are called SMFs because of these additional requirements, which improve the inelastic response characteristics of these frames in comparison with less stringently detailed IMFs and OMFs. In the early 1970s, engineers began to use the connection type known today as the welded unreinforced flange bolted web incorporating field-welded, complete joint penetration groove welds to join beam flanges to columns, with shop-welded, field-bolted shear plates joining beam webs to columns.

Following the 1994 Northridge earthquake in Los Angeles area, engineers were surprised to discover that a number of modern special moment-resisting frame structures had experienced brittle fracturing of their welded beam-to-column connections. Similar damage occurred one year later in the 1995 Kobe earthquake in Japan. The causes of this unanticipated behavior were investigated and recommendations developed for more robust moment-resisting frame construction. This resulted in the basis for the current design provisions for moment-resisting frames contained in AISC 341, AISC 358, and AWS D1.8.

In the aftermath of the 1994 Northridge, California earthquake, damage to steel SMF connections (in the Los Angeles area) spawned concern about the reliability of established design and construction procedures. A number of buildings experienced damage in beam-to-column connections that underwent only moderate inelastic demands. Failures included fractures of the bottom beam flange-to-column flange CJP groove welds, cracks in beams flanges, and cracks through the column section. The fractures were a result of the basic connection geometry, lack of control of base materials properties, the use of weld filler metals with inherent low toughness, uncontrolled deposition rates, inadequate quality control, and other factors.

7.2.1.1 SMF Design

Steel SMFs, permitted to be designed using a value of $R = 8$, are expected to sustain multiple cycles of significant inelastic response when subjected to design-level ground motion. However, many steel SMF structures have substantial overstrength. This overstrength results from oversizing of columns to meet strong-column/weak-beam criteria, use of oversize sections to provide sufficient stiffness for drift control, and variability in the strength of the steel material itself. As a result, although the R value of 8 specified by the code would imply initiation of inelastic behavior shaking with an intensity 1/8 that of the design earthquake, many steel SMF structures are likely to remain elastic at an intensity of 1/3 that of the design earthquake. Therefore, in a manner of speaking, the effective R value for an SMF may be as little as 3.

The proportioning and detailing requirements of AISC 341 are intended to provide ductile inelastic response. The primary goal is to achieve a strong-column/weak-beam condition that distributes inelastic response over several stories to prevent early formation of single story mechanism.

In order to avoid development of P-delta instability, it is desirable to achieve a relatively uniform distribution of lateral drift over the structure's height. To achieve this, it is important to avoid early formation of single-story mechanisms in which inelastic response is dominated by formation of plastic hinges at the tops and bottoms of columns within a single story. When such single story mechanisms form, most of the inelastic portion of a structure's drift will occur within these stories, resulting in very large P-delta effects at those locations. In order to avoid this, building codes require designs intended to promote formation of multi-story sideways mechanisms dominated by hinging of beams, (Figure 7.4), as opposed to column hinging, like the idealized sidesway mechanism of Figure 7.3. These requirements are termed strong-column/weak-beam design.

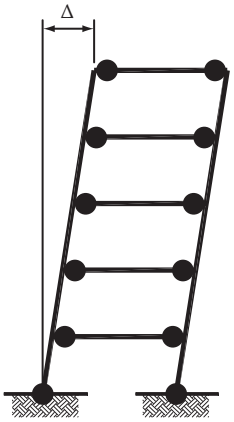


FIGURE 7.4 Multi-story sidesway mechanism dominated by hinging of beams.

AISC 341 requires that the sum of column flexural strengths at each joint exceed the sum of beam flexural strengths. When determining available column flexural strength, it is important to consider the axial loads that will be simultaneously present in the column along with flexural demands. The provisions also require supplemental lateral bracing of beam–column connections, unless it can be shown that the columns will remain elastic. When the column–beam moment ratio is 2 or greater, AISC 341 permits an assumption that columns will remain elastic.

7.2.1.2 AISC Prequalified Connections

Since the 1994 Northridge earthquake, AISC 341 has required that steel SMF moment connections be demonstrated through qualification testing to be capable of developing at least 0.04 rad of inter-story drift without excessive strength loss, when subjected to the cyclic loading protocol specified in AISC 341.

AISC maintains a Connection Prequalification Review Panel that develops an American National Standards Institute (ANSI)-approved standard, AISC-358 Prequalified Connections for Special and Intermediate Moment Resisting Frames for Seismic Applications. The standard presents materials, design, detailing, fabrication, and inspection requirements for a series of prequalified moment connection details.

In some cases, the prequalification available in AISC 358-10, and evaluation service reports may not be adequate to cover the design conditions for a particular project. One reason this may occur is that the sizes of frame elements selected for a steel SMF may fall outside the limits contained within the prequalifications. Another reason this may occur is that presently there are no prequalifications associated with connections to the minor axis of wide-flange columns. If no prequalified connections meet the requirements of a particular design condition, AISC 341 requires project-specific testing. At least two specimens must be tested and must pass the criteria specified in AISC 341.

7.2.1.3 Ductile Behavior

As a highly ductile system, it is expected that steel SMFs will undergo significant inelastic behavior in numerous members when subjected to severe seismic shaking. The primary source of this inelastic behavior is intended to occur in the form of plastic hinging in the beams, adjacent to the beam–column connections. In a properly configured system, this hinging should occur over multiple stories to spread the total displacement demand and limit the local deformations and member strains to a level that the members can withstand. In addition to the hinging of beams, inelastic behavior can be expected to occur in beam–column joint panel zones and at column bases.

A number of features are incorporated into steel SMF design to achieve the intended ductility level. One primary feature is the level of compactness required of beam and column members.

In addition, steel SMF members also must be laterally braced for stability. AISC 341 prescribes a maximum spacing distance for lateral bracing or steel SMF beams and specified stiffness and strength criteria for this bracing to avoid lateral-torsional buckling.

The design of steel SMF column bases should be strong enough so that inelastic deformation is limited to a region that can exhibit significant ductility, such as the column member just above the base connection. Another approach is to design and detail anchor bolts to yield as a means of limiting demand on other elements of the connection. In some cases, engineers may wish to design columns assuming the bases are "pinned." In those cases, it is important to detail the bases to accommodate the large anticipated rotation without failing the anchorage and attachment to foundations.

7.2.1.4 Seismically Compact Sections

Reliable inelastic deformation requires that width–thickness ratios of compression elements be limited to a range that provides a cross section resistant to local buckling into the inelastic range. AISC 360 uses the term "compact" for steel cross sections that are expected to be able to achieve the full plastic section capacity. In AISC 341, a higher level of compactness (termed "seismically compact") is required of both steel special moment frame beams and columns. Seismically compact sections are expected to be able to achieve a level of deformation ductility of at least 4. To be seismically compact, AISC 341 requires member flanges to be continuously connected to the web(s) and the width–thickness ratios of the compression elements must be less than or equal to those that are resistant to local buckling when stressed into the inelastic range.

7.2.1.5 Demand Critical Welds

AISC 341 defines demand critical welds as those that require increased quality and toughness requirements based upon inelastic strain demand and the consequence of failure. Unless otherwise designated by AISC 358, or as determined in either prequalification or qualification testing, welds designated as demand critical in steel SMFs are identified specifically in AISC 341 as CJP groove welds of beam flanges, shear plates, and beam webs to columns. Other complete joint penetration groove welds considered demand critical include those at a column splice, those joining a column and base plate, and those in built-up members joining a web plate to a flange in the plastic hinge region.

7.2.1.6 Protected Zones

AISC 341 requires designation of the region at each end of a steel SMF beam subject to inelastic straining as a protected zone. Protected zones must meet the requirements of AISC 341. AISC 358 designates the location and extent of protected zones for prequalified connections.

7.2.1.7 Panel Zone of Beam-to-Column Connections

Panel zones experience large shear forces due to the transfer of moments from beams to columns. As these shear forces increase, a panel zone starts to yield at its center. Subsequently, yielding propagates toward the panel zone corners. Shear yielding in panel zones is a very ductile mode of deformation. Panel zones can undergo many cycles of large inelastic distortions without deterioration in strength, while exhibiting cyclic hardening. It is desirable to have panel zones participate in dissipating energy through inelastic deformations and by doing so decrease the energy dissipation demands on plastic hinge regions in beams. Global shear yielding of the panel zone occurs when the average shear stress in the panel zone is about $0.6F_y$, where F_y is the specified minimum yield stress. However, the actual shear strength of the panel zone might be considerably larger than the shear force that causes global shear yielding, because of strain hardening and the additional resistance provided by the column flanges bounding the panel zone. AISC 360 permits utilization of this additional shear resistance in design when the flexibility of the panel zone is considered in analysis.

7.2.2 Moment Frame Systems

Moment-frame systems discussed in AISC 341-10 may be organized into six categories as follows:

1. Ordinary moment frames
2. Intermediate moment frames
3. Special moment frames
4. Special moment truss frames
5. Ordinary cantilever column systems (not discussed here)
6. Special cantilever column systems (not discussed here)

7.2.2.1 Ordinary Moment Frames

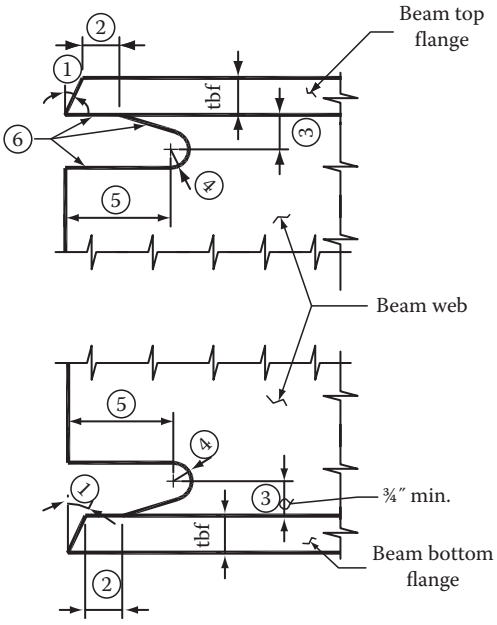
1. Basis of design
   OMF shall be designed to withstand minimal inelastic drift.
2. Members
   Structural steel beams in OMF are permitted to be composite with a reinforced concrete slab to resist gravity loads. Trusses shall be permitted as beams in OMF. The required strength of truss members shall be based on the maximum forces that can be delivered to the truss by the system, including overstrength and strain hardening. Truss connections shall be FR connections.
3. Demand critical welds
   CJP welds of beam flanges to columns shall be demand critical welds. Single-sided partial-joint-penetration groove welds and single-sided fillet welds should be avoided for resisting tensile forces in connections.
4. Requirements for moment connections
   Moment connections that are part of the SFRS shall meet at least one of the following requirements:
   a. Moment connections shall be designed for a required flexural strength that is equal to $1.1R_yM_p$ of the beam. The required shear strength V_u of the connection shall be determined using the following quantify for the earthquake load effect E.

$$E = 2[1.1R_yM_p]/L_{cf}$$

      where
      R_y is the ratio of expected yield stress to the specified minimum yield stress, F_y
      $M_p = F_yZ$ (kip-in.)
      L_{cf} is the clear length of beam (in.)

   b. Moment connections shall be designed for a required flexural strength and a required shear strength equal to the maximum moment and corresponding shear that can be transferred to the connection by the system, including the effects of material overstrength and strain hardening. Factors that may limit the maximum moment and corresponding shear that can be transferred to the connection include the following:
      i. The strength of the columns
      ii. The resistance of the foundations to uplift
      For options (i) and (ii) above, the continuity plates should be provided. The bending moment used to check for continuity plates should be the same bending moment used to design the beam-to-column connection. In other words, either $1.1R_yM_p$ or the maximum moment that can be transferred to the connection by the system.

c. Moment connections between wide flange beams and the flange of wide flange columns shall meet the following requirements:
   i. All welds at the beam-to-column connection shall meet the requirements of ANSI/AISC 358.
   ii. Beam flanges shall be connected to column flanges using CJP groove welds.
   iii. The shape of weld access holes shall be in accordance with the requirements of AWS D1.8 (see Figure 7.5).
   iv. Continuity plates shall be at least as wide as the flanges of the wider connected beam.



Dimension from Provisions Figure 11-1					
Access Hole Type	①	②	③	④	⑤
	Degrees	in.	in.	in.	in.
A	30	½	½	½	1¼
B		½	¾		1½
C		¾	1		2½
D		1	1¼		3½
E		1¼	1½		4½
F		1½	1¾		5½
G		1¾	2		6½
H		2	2¼		7½
I		2¼	2½		8½
J		2½	2¾		9½
K		3	3		11
L		3¼	3½		12½
M		3¾	4		14
N		4	4¼		15

FIGURE 7.5 Weld access hole requirements.

Exception: the welded joints of the continuity plates to the column flanges are permitted to be CJP groove welds, two-sided partial-joint-penetration groove welds, or two-sided fillet welds. The required strength of these joints shall not be less than the available strength of the contact area of the plate with the column flange.

v. The beam web shall be connected to the column flange using either a CJP groove weld extending between weld access holes, or using a bolted single plate shear connection designed for required shear strength.

7.2.2.2 Intermediate Moment Frames

1. Basis of design
   IMF shall be designed to provide limited inelastic drift capacity through flexural yielding of the IMF beams, columns and shear yielding of the column panel zones. Design of connections of beams to columns including panel zones and continuity plates, shall be based on connection tests. The required shear strength of the panel zone should be based on the beam end moments.
2. Stability bracing of beams
   Beams shall be braced to meet the requirements for moderately ductile members. In addition, member braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection.
3. Width–thickness limitations
   Beam and column members shall satisfy the requirements for moderately ductile members.
4. Beam flanges
   Abrupt changes in beam flange area shall not be permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width shall not be permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection.
5. Protected zones
   The region at each end of the beam subject to inelastic straining shall be designated as a protected zone. The plastic hinging zone at the ends of IMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.
6. Beams
   Frame beams in IMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.
7. Connections
   a. Demand critical welds
      CJP groove welds of beam flanges and beam webs to columns shall be demand critical welds.
   b. Beam-to-column connection requirements
      Beam-to-column connections used in the SFRS shall satisfy the following requirements:
      i. The connection shall be capable of sustaining an interstory drift angle of at least 0.02 rad.
      ii. The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.02 rad.

8. Conformance demonstration
   Beam-to-column connections used in the SFRS shall satisfy the requirements of one of the following:
   a. Use of IMF connections designed in accordance with ANSI/AISC 358-10.
   b. Use of a connection prequalified for IMF.
   c. Provision of qualifying cyclic test results. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
      i. Tests reported in the research literature or documented tests performed for other projects that represent the project conditions
      ii. Tests that are conducted specifically for the project are representative of project member sizes, material strengths, connection configurations, and matching connection processes
9. Required shear strength
   The required shear strength of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1R_yM_p]/L_h$$

   where
   R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y
   $M_p = F_yZ$ is the nominal plastic flexural strength (kip-in.)
   L_h is the distance between beam plastic hinge locations (in.)

10. Continuity plates
    Continuity plates shall be consistent with the prequalified connection designated in ANSI/AISC 358.
11. Column splices
    Where welds are used to make the splice, they shall be CJP groove welds.
    When bolted column splices are used, they shall have a required flexural strength that is at least equal to $R_yF_yZ_x$ of the smaller column. The required shear strength of column web splices shall be at least equal to ● $M_{pc/H}$ as appropriate, where ● M_{pc} is the sum of the nominal plastic flexural strengths of the column above and below the splice.
    Exception: The required strength of the column splice need not exceed that determined by a nonlinear analysis considering appropriate stress concentration factors or fracture mechanics stress intensity factors.

7.2.2.3 Special Moment Frames

1. Basis of design
   SMF shall be designed to provide significant inelastic drift capacity through flexural yielding of the SMF beams and limited yielding of column panel zones. Columns shall be designed to be generally stronger than the fully yielded and strain-hardened beams or girders. Flexural yielding of columns at the base shall be permitted. Design of connections of beams to columns, including panel zones and continuity plates, shall be based on connection tests.
2. Moment ratio
   The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M_{pc}^*}{\sum M_{pb}^*} > 1.0$$

where ΣM_{pc}^{*} is the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. ΣM_{pc}^{*} is determined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. It is permitted to determine ΣM_{pc}^{*} as follows:

$$\Sigma M_{pc}^{*} = \Sigma Z_c (F_{yc} - P_{uc}/A_g)$$

When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

ΣM_{pb}^{*} is the sum of the moments in the beams at the intersection of the beam and column centerlines. ΣM_{pb}^{*} is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to determine ΣM_{pb}^{*} as follows:

$$\Sigma M_{pb}^{*} = \Sigma (1.1 R_y F_{yb} Z_b + M_{uv})$$

Alternatively, it is permitted to determine ΣM_{pb}^{*} consistent with a prequalified connection design as designated in ANSI/AISC 358. When connections with reduced beam sections are used, it is permitted determine ΣM_{pc}^{*} as follows:

$$\Sigma M_{pb}^{*} = \Sigma (1.1 R_y F_{yb} Z_{RBS} + M_{uv})$$

A_g is the gross area of column (in.^{2})
F_{yc} is the specified minimum yield stress of column (ksi)
M_{uv} is the additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRED load combinations (kip-in.)
Z_b is the plastic section modulus of the beam (in.^{3})
Z_c is the plastic section modulus of the column (in.^{3})
Z_{RBS} is the minimum plastic section modulus at the reduced beam section (in.^{3})

Exception: These requirements shall not apply if the conditions in (a) or (b) are satisfied.

a. Columns with $P_{rc} < 0.3P_c$ for all load combinations other than those determined using the amplified seismic load that satisfy either of the following:
   i. Columns used in a one-story building or the top story of a multistory building.
   ii. Columns where (1) the sum of the available shear strengths of all exempted columns in the story is less than 20% of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction; and (2) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33% of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10% of the plan dimension perpendicular to the line of columns. For design

$$P_c = F_{yc} A_g \text{ (kip)}$$

   $P_{rc} = P_{uc}$ is the required compressive strength using LRFD load combinations (kip)

b. Columns in any story that has a ratio of available shear strength to required shear strength that is 50% greater than the story above

3. Stability bracing of beams
   Beams shall be braced to meet the requirements for highly ductile members. In addition, member braces shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358.
4. Stability bracing at beam-to-column connections
   a. Braced connections
      When the webs of the beams and columns are co-planar, and a column is shown to remain elastic outside of the panel zone, column flanges at beam-to-column connections shall require stability bracing only at the level of the top flanges of the beams. $\Sigma M_{pc}^{*}/\Sigma M_{pb}^{*}$ is greater than 2.0.
      When a column cannot be shown to remain elastic outside of the panel zone, the following requirements shall apply:
      i. The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Stability bracing shall be permitted to be either direct or indirect. Direct lateral support (bracing) of the column flange is achieved through use of member braces or other members, deck and slab, attached to the column flange at or near the desired bracing point to resist lateral buckling. Indirect lateral support refers to bracing that is achieved through the stiffness of members and connections that are not directly attached to the column flanges, but rather act through the column web or stiffener plates.
      ii. Each column flange member brace shall be designed for a required strength that is equal to 2% of the available beam flange strength $F_y b_f t_{bf}$.
   b. Unbraced connections
      A column containing a beam-to-column connections with no member bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent member braces as the column height for buckling transverse to the seismic frame, except that
   c. The required column strength shall be determined from the appropriate load combinations except that the earthquake load E shall be permitted to be taken as the lesser of
      i. The amplified seismic load.
      ii. 125% of the frame available strength based upon either the beam flexural strength or panel zone shear strength.
      The slenderness L/r for the column shall not exceed 60.
      The column required flexural strength transverse to the seismic frame shall include that moment caused by the application of the beam flange force in addition to the second-order moment due to the resulting column flange lateral displacement.
5. Members
   a. Width–thickness limitations
      Beam and column members shall meet the requirements for highly ductile members, unless otherwise qualified by tests.
   b. Beam flanges
      Abrupt changes in beam flange area shall not be permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width shall not be permitted.
   c. Protected zones
      The region at each end of the beam subject to inelastic straining shall be designated as a protected zone. The extent of the protected zone shall be as designated in ANSI/AISC 358. The plastic hinging zones at end of SMF beams should be treated as protected zones. The plastic hinging zones should be established as part of a prequalification or qualification program for the connection. In general, for unreinforced connections, the protected zone will extend from the face of the column to one half of the beam depth beyond the plastic hinge point.

d. Beams
Frame beams in SMF are permitted to be composite with a reinforced concrete slab to resist gravity loads.

6. Demand critical welds
CJP groove welds or beam flanges and beam webs to columns shall be demand critical welds.
7. Beam-to-column connection requirements
Beam-to-column connections used in the SFRS shall satisfy the following two requirements:
   a. The connection shall be capable of sustaining an interstory drift angle of at least 0.4 rad.
   b. The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 rad.
8. Conformance demonstration
Beam-to-column connections used in the SFRS shall satisfy the following requirements:
   a. Use of SMF connections designed in accordance with ANSI/AISC 358.
   b. Use of a connection prequalified for SMF.
   c. Provision of qualifying cyclic test results. Results of at least two cyclic connection tests shall be provided and shall be based on one of the following:
      i. Tests reported in the research literature or documented tests performed for other projects that represent the project conditions
      ii. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes
9. Required shear strength
The required shear strength of the connection shall be determined using the following quantify for the earthquake load E:

$$E = 2[1.1R_yM_p]/L_h$$

where
R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y
M_p is the nominal plastic flexural strength (kip-in.)
L_h is the distance between plastic hinge locations (in.)

10. Panel zone
   a. Required shear strength
   As a minimum, the required shear strength of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength shall be $\varphi_v R_v$
   where

$$\varphi_v = 1.0$$

   and the nominal shear strength, R_v, according to the limit state of shear yielding.
   b. Panel zone thickness
   The individual thickness, t, of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90$$

   where
   t is the thickness of column web or double plate (in.)
   d_z is the panel zone depth between continuity plates (in.)
   w_z is the panel zone width between column flanges (in.)

Alternatively, when local buckling of the column web and doubler plate is prevented by using plug welds joining them, a minimum of four plug welds shall be provided.

c. Panel zone doubler plates
Doubler plates shall be welded to the column flanges to develop the available shear strength of the full doubler plate thickness, using either a CJP groove-welded or fillet-welded joint. When doubler plates are placed against the column web, they shall be welded across the top and bottom edges to develop the proportion of the total force that is transmitted to the double plate. When doubler plates are placed away from the column web, they shall be place symmetrically in pairs and welded to continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

11. Column splices
Where groove welds are used to make the splice, they shall be CJP groove welds.
When bolted column splices are used, they shall have a required flexural strength that is at least equal to $R_y F_y Z_x$ of the smaller column. The required shear strength of column web splices shall be at least equal to $\Sigma M_{pc}/H$, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.
Exception: The required strength of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by a nonlinear analysis.

7.2.2.4 Special Truss Moment Frames

1. Basis of design
STMF shall be designed to withstand significant inelastic deformation within a special segment of the truss when subjected to the forces from the motions of the design earthquake. STMF shall be limited to span lengths between columns not to exceed 65 ft and overall depth not to exceed 6 ft. The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment.
2. System requirements
   a. Special segment
   Each horizontal truss that is part of the SFRS shall have a special segment that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

   Panels within a special segment shall either be all Vierendeel panels or all X-braced panels neither a combination there of nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Diagonal members within the special segment shall be made of rolled flat bars of identical sections. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a required strength equal to 0.25 times the nominal tensile strength of the diagonal member. Bolted connections shall not be used for diagonal members within the special segment.

   Splicing of chord members shall not be permitted within the special segment, or within one-half the panel length from the ends of the special segment.

   The required axial strength of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03 F_y A_g$.

b. Stability bracing of trusses
Each flange of the chord members shall be laterally braced at the ends of the special segment. The required strength of the lateral brace shall be

$$P_u = 0.06 R_y F_y A_f$$

where A_f is the gross area of the flange of the special segment chord member (in.^{2})

c. Stability bracing of truss-to-column connections
The columns shall be laterally braced at the levels of top and bottom chords of the trusses connected to the columns. The lateral braces shall have required strength of

$$P_u = 0.02 R_y P_{nc}$$

where P_{nc} is the nominal compressive strength of the chord member at the ends, kip

i. Stiffness of stability bracing
The required brace stiffness shall be determined using

$$P_r = R_y P_{nc}$$

d. Members
i. Special segment members
The available shear strength of the special segment shall be calculated as the sum of the available shear strength of the chord members through flexure, and of the shear strength corresponding to the available tensile strength and 0.3 times the available compressive strength of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25% of the required vertical shear strength.
The required axial strength of the chord members shall not exceed 0.45 times φP_n

$$\Phi = 0.90$$

where

$$P_n = F_y A_g$$

ii. Expected vertical shear strength of special segment
The expected vertical shear strength of the special segment, V_{ne} at mid-length, shall be given as

$$V_{ne} = \frac{3.60 R_y M_{nc}}{L_s} + 0.036 EI \frac{\left(L - L_s\right)}{L_s^3} + R_y \left(P_{nt} + 0.3 P_{nc}\right) \sin \alpha$$

where

M_{nc} is the nominal flexural strength of a chord member of the special segment (kip-in.)

E is the modulus of elasticity of a chord member of the special segment (kin/in.^{2})

I is the moment of inertia of a chord member of the special segment (in.^{4})

L is the span length of the truss (in.)

L_s is the length of the special segment (in.)

P_{nt} is the nominal tensile strength of a diagonal member of the special segment (kip)

P_{nc} is the nominal compressive strength of a diagonal member of the special segment (kip)

α is the angle of diagonal members with the horizontal

iii. Width–thickness limitations
Chord members and diagonal web members within the special segment shall meet the requirements for highly ductile members. The width–thickness ratio of flat bar diagonal members shall not exceed 2.5.

iv. Built-up chord members
Spacing of stitching for built-up chord members in the special segment shall not exceed $0.04Er_y/F_y$, where r_y is the radius of gyration of individual components about their weak axis.

v. Protected zones
The region at each end of a chord member within the special segment shall be designated as protected zone. The protected zone shall extend over a length equal to two times the depth of the chord member from the connection with the web members. Vertical and diagonal web members from end to end of the special segments shall be protected zones.

vi. Connections
Connections of diagonal web members in the special segment
The end connection of diagonal web members in the special segment shall have a required strength that is at least equal to the expected yield strength of the web member, $R_yF_yA_g$.

vii. Column splices
Where welds are used to make the splice, they shall be CJP groove welds.
When bolted column splices are used, they shall have a required flexural strength that is at least equal to $R_yF_yZ_x$ of the smaller column. The required shear strength of column web splices shall be at least equal to $\Sigma M_{pc}/H$, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Exception: The required strength of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by a nonlinear analysis.

7.2.3 Braced-Frame and Shear-Wall Systems

Braced frame and shear walls are organized into five categories as follows:

1. Ordinary concentrically braced frames (OCBF)
2. Special concentrically braced frames
3. Eccentrically braced frames
4. Buckling-restrained braced frames
5. Special plate shear walls

7.2.3.1 Ordinary Concentrically Braced Frames

1. Basis of design
   Braced frames that consist of concentrically connected members are referred to as CBFs. Minor eccentricities less than the beam depth are permitted if they are accounted from in the member design by determination of eccentric moments using the amplified seismic load. OCBF are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake.
2. V-braced and inverted V-braced frames
   Beams in V-type and inverted V-type OCBF shall be continuous at brace connections away from the beam–column connection and shall meet the following requirements:
   a. The required strength shall be determined assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E, on the member shall be determined as follows:
      i. The forces in braces in tension shall be assumed to be the least of the following:
         The expected strength of the brace, $R_y F_y A_g$
         The load effect based upon the amplified seismic load
         The maximum force that can be developed by the system
      ii. The forces in braces in compression shall be assumed to be equal to $0.3P_n$
   b. As a minimum, one set of lateral braces is required at the point of intersection of the braces, unless the member has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.
3. K-braced frames
   a. K-type braced frames are not permitted for OCBF
   b. Width–thickness limitations
      Braces shall meet the requirements for moderately ductile members
   c. Slenderness
      Braces in V or inverted V configurations shall have $Kl/r \leq 4\sqrt{E/F_y}$
4. Diagonal brace connections
   The required strength of diagonal brace connections is the load effect based upon the amplified seismic load.

   Exception: The required strength of the brace connection need not exceed the following:
   a. The maximum force that can be developed by the system.
   b. In tension, the expected yield strength of the brace, determined as $R_y F_y A_g$.
   c. In compression, 1.1 times the expected brace strength in compression. The expected brace strength in compression is permitted to be taken as the lesser of $R_y F_{cr} A_g$ and $F_E A_g$.
   d. When OVS holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the load combinations stipulated by the applicable building code, not including the amplified seismic load. Observe that in seismic design, slip is a strength limit state.

7.2.3.2 Special Concentrically Braced Frames

1. Scope
   SCBFs consist of concentrically connected members. Minor eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of system ductility: buckling and tension yielding of braces.
2. Basis of design
   SCBF shall be designed so that inelastic deformation under the design earthquake will occur primarily as brace buckling and yielding in tension.

3. Analysis

Columns, beams, and connections in SCBF shall be designed to resist seismic forces from the load combinations where the seismic load, E, on each element is the larger force determined from the following two analyses:

a. An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension
b. An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength

Braces shall be determined to be in compression or tension neglecting the effects of gravity loads. Analyses shall consider both directions of frame loading.

The expected brace strength in tension is $R_y F_y A_g$.

The expected brace strength in compression is permitted to be taken as the lesser of $R_u F_{cr} A_g$, and $F_E A_g$ where F_{cr} and F_E are determined from the Specification Chapter E and the brace length is taken from brace end to brace end.

The expected post-buckling brace strength shall be taken as a maximum of 0.3 times the expected brace strength in compression. Braces with a slenderness ratio of 200 (the maximum permitted by Section F2.5 AISC 341-10), buckle elastically for permissible materials; the value of $0.3F_{cr}$ for such braces is 2.1 ksi. This value may be used in Section F2.3(ii) AISC 341-10, for braces of any slenderness and a liberal estimate of the required strength of framing members will be obtained.

Exceptions:

1. It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.
2. The required strength of columns need not exceed the least of the following:
   a. The forces determined using load combinations including the amplified seismic load, applied to a building frame model in which all compression braces have been removed
   b. The forces corresponding to the resistance of the foundation to overturning uplift
   c. Forces determined from nonlinear analysis

4. Lateral force distribution

Along any line of braces, braces shall be deployed in alternate directions such that, for either direction of force parallel to the braces, at least 30% but no more than 70% of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the required strength resulting from the appropriate load combinations including the amplified seismic load. For the purposes of this provision, a line of braces is defined as a single lien or parallel lines with a plan offset of 10% or less of the building dimension perpendicular to the line of braces.

5. V-braced and inverted V-braced frames

Beams that are intersected by braces away from beam-to-column connections shall comply with the following requirements:

a. Beams shall be continuous between columns.
b. Beams shall be braced to meet the requirements for moderately ductile members.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) braced frames, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points. Observe that one method of demonstrating sufficient out-of-plane strength and stiffness of the beam is to apply the bracing force to each flange as to form a torsional couple; this loading should be in conjunction with the flexural forces.

6. K-braced frames
   K-type braced frames are not permitted for SCBF.
7. Tension-only braced frames
   These are not permitted in SCBF. Observe that tension-only braced frames are those in which the brace compression resistance is neglected in the design and the braces are designed for tension forces only.
8. Width–thickness limitations
   Columns and braces shall meet the requirements for highly ductile members. Beams shall meet the requirements for moderately ductile members.
9. Diagonal braces
   Braces shall comply with the following requirements:
   a. Slenderness: Braces shall have a slenderness ratio $Kl/r \leq 200$
   b. Built-up braces: The spacing of stitches shall be such that the slenderness ratio l/r of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member

      The sum of the available shear strengths of the stitches shall equal or exceed the available tensile strength of each element. The spacing of stitches shall be uniform. Not less than two stitches shall be used in a built-up member. Stitches shall not be located within the middle one-fourth of the clear brace length.

      Exception: Where the buckling of braces about their critical buckling axis does not cause shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio l/r of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member
   c. The brace effective net area shall not be less than the brace gross area. Where reinforcement on braces is used the following requirements shall apply:
      i. The specified minimum yield strength of the reinforcement shall be at least the specified minimum yield strength of the brace
      ii. The connections of the reinforcement to the brace shall have sufficient strength to develop the expected reinforcement strength on each side of a reduced section
10. Protected zones
    The protected zone of SCBF shall satisfy the following:
    a. For braces, the center one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling
    b. Elements that connect braces to beams and columns
11. Required strength of brace connections
    The required strength in tension, compression and flexure of brace connections (including beam-to-column connections is part of the braced-frame system) shall be determined as required below. These required strengths are permitted to be considered independently without interaction.
    a. Required tensile strength: The required tensile strength is the lesser of the following:
       i. The expected yield strength, in tension of the brace, determined as $R_y F_y A_g$
       ii. The maximum load effect, indicated by analysis, that can be transferred to the brace by the system

       When OVS holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the load combinations, including the amplified seismic load.
    b. Required compressive strength: Brace connections shall be designed for a required compressive strength based on buckling limit states that is at least equal to 1.1 times the expected brace strength in compression.

c. Accommodation of brace buckling: Brace connections shall be designed to withstand the flexural forces or rotations imposed by brace buckling. Connections satisfying either of the following provision are deemed to satisfy this requirement:
   i. Required flexural strength: Brace connections designed to withstand the flexural forces imposed by brace buckling shall have an available flexural strength of at least $1.1R_yM_p$ of the brace above the critical buckling axis.
   ii. Rotation capacity: Brace connections designed to withstand the rotations imposed by brace buckling shall have sufficient rotation capacity to accommodate the required rotation at the design story drift. Inelastic rotation of the connection is permitted. Observe that accommodation of inelastic rotation is typically accomplished by means of a single gusset plate with the brace termination before the line of restraint.

12. Beam-to-column connections
Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:
   a. The connection shall have sufficient rotation capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height
   b. The connection shall be designed to resist a moment equal to the lesser of the following:
      i. A moment corresponding to $1.1R_yF_yZ$ of the beam
      ii. A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column

   This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.
13. Column splices
Column splices in SCBF shall be designed to develop at least 50% of the lesser available flexural strength of the connected members.
The required shear strength shall be $\Sigma M_{pc}/H_c$
where
ΣM_{pc} is the sum of the nominal plastic flexural strengths, $F_{yc}Z_c$, of the columns above and below the splice (kip-in.)
H_c is the clear height of the column between beam connections, including a structural slab, if present (in.)

14. Demand critical welds
Groove welds at column splices shall be demand critical welds.

7.2.3.3 Eccentrically Braced Frames

1. Scope
In EBFs minor eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of system ductility; buckling and tension yielding of braces.
2. Basis of design
EBF shall be designed so that inelastic deformations under the design earthquake will occur primarily in the links.
3. Analysis
The required strength of diagonal braces and their connections, beams outside links, and columns shall be based on the load combinations that include earthquake effects. The earthquake effect E for columns, diagonal braces, connections and beams outside links shall be taken as the forces developed in the member assuming the forces at the end of the link connected to the member correspond to the adjusted link shear strength. The adjusted

link shear strength shall be taken as R_y times the link nominal shear strength multiplied by 1.25 for I-shaped links and 1.4 for box links.

Exceptions:

1. The earthquake effect, E, is permitted to be taken as 0.88 times the force determined above for the design of the following members:
   a. Beams outside links
   b. Columns in frames of three or more stories of bracing
2. It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.
3. The required strength of columns need not exceed the lesser of the following:
   a. Forces corresponding to the resistance of the foundation to overturning uplift
   b. Forces as determined from nonlinear analysis

   The inelastic link rotation angle shall be determined from the inelastic story drift. Alternatively, the inelastic link rotation angle may be determined from nonlinear analysis.

   The seismic load effect E used in the design of EBF members, such as the required axial strength, should be calculated from the analysis above.

4. Link rotation angle

   The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift, Δ. The link rotation angle shall not exceed the following values:

   a. 0.08 rad for links of length $1.6M_p/V_p$ or less
   b. 0.02 rad for links of length $2.6M_p/V_p$ or greater

   The value determined by linear interpolation between the above values for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

   Bracing of link

   Bracing shall be provided at both the top and bottom link flanges at the ends of the link for I-shaped sections. Bracing shall have an available strength and stiffness as required for expected plastic hinge locations.

5. Links

   Link beams subject to shear and flexure due to eccentricity between the intersections of brace centerlines and the beam centerline (or between the intersection of the brace and beam centerlines and the column centerline for links attached to columns) shall be provided. The link beam shall be considered to extend from brace connection to brace connection for center links and from brace connection to column face for link-to-column connections.

   a. Limitations

      Links shall be I-shaped cross sections (rolled wide-flange sections or built-up sections), or built-up box sections. HSS sections shall not be used as links.

      Links shall meet the requirements of highly ductile members.

      Exception: Flanges of links with I-shaped sections with link lengths, $e \leq 1.6M_p/V_p$, are permitted to meet the requirements for moderately ductile members.

      The web or webs of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

      For links made of built-up cross sections, CJP groove welds shall be used to connect the web (or webs) to the flanges.

      Links of built-up box sections shall have a moment of inertia, I_y, about an axis in the plane of the EBF limited to $I_y > 0.67I_x$, where I_x is the moment of inertia about an axis perpendicular to the plane of the EBF.

b. Shear strength

The link design shear strength, $\varphi_v V_n$, shall be the lower value obtained according to the limit states of shear yielding in the web and flexural yielding in the gross section. For both limit states:

$$\varphi_v = 0.9$$

For shear yielding:

$$V_n = V_p$$

where

$V_p = 0.6F_yA_{lw}$ for $P_r/P_c \le 0.15$

$V_p = 0.6F_yA_{lw}\sqrt{1-(P_r/P_c)^2}$ for $P_r/P_c > 0.15$

$A_{lw} = (d - 2t_f)t_w$ for I-shaped link sections

$= 2(d - 2t_f)t_w$ for box sections

$P_r = P_u$

P_u is the required axial strength (kip)

$P_c = P_y$

P_y is the nominal axial yield strength = F_yA_g

For flexural yielding:

$$V_n = 2M_p/e$$

where

$M_p = F_yZ$ for $P_r/P_c \le 0.15$

$M_p = F_yZ[1 - P\lambda_y/P_c/0.85]$ for $P_r/P_c > 0.15$

e is the link length, in. (mm)

c. Link length

If $P_r/P_c > 0.15$, the length of the link shall be limited as follows:

When $\rho' \le 0.5$

$$e'' \ 1.6\frac{M_p}{V_p}$$

When $\rho' > 0.5$

$$e \le 1.6M_p/V_p(1.15 - 0.3\rho')$$

where

$$P' = \frac{P_r/P_c}{V_r/V_c}$$

$V_r = V_u$ (kip)

V_u is the required shear strength (kip)

$V_c = V_y$

V_y is the nominal shear yield strength (kip)

$= 0.6F_yA_{lw}$

d. Link stiffeners for I-shaped cross sections

Full-depth web stiffness shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ or 3/8 in., whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

i. Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation of 0.08 rad or $(52t_w - d/5)$ for link rotation angles of 0.02 rad or less. Linear interpolation shall be used for values between 0.08 and 0.02 rad.

ii. Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.

iii. Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (i) and (ii) in this list.

Intermediate web stiffeners are not required in links of length greater than $5M_p/V_p$.

Intermediate web stiffeners shall be full depth. For links that are less than 25 in. (635 mm) in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 3/8 in., whichever is larger, and the width shall not be less than $(b_f/2) - t_w$. For links that are 25 in. in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The required strength of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ where A_{st} is the horizontal cross-sectional area of the link stiffener. The required strength of fillet welds connecting the stiffener to the link flange is $A_{st}F_y/4$.

5. Link stiffeners for box sections

Full-depth web stiffeners shall be provided on one side of each link web at the diagonal brace connection. These stiffeners may be welded to the outside or inside face of the link webs. These stiffeners shall have a width not less than $b/2$, where b is the inside width of the box. These stiffeners shall each have a thickness not less than $0.75t_w$ or ½ in., whichever is less.

Box links shall be provided with intermediate web stiffeners as follows:

a. For links of length $1.6M_p/V_p$ or less and with web depth-to-thickness ratio, h/t_w greater than or equal to $0.64\sqrt{E/F_y}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20t_w - (d - 2t_f)/8$.

b. For links of length $1.6M_p/V_p$ or less and with web depth-to-thickness ratio, h/t_w less than $0.64\sqrt{E/F_y}$, no intermediate web stiffeners are required.

c. For links of length greater than $1.6M_p/V_p$, no intermediate web stiffeners are required.

Intermediate web stiffeners shall be full depth, and may be welded to the outside or inside face on the link webs.

The required strength of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ where A_{st} is the horizontal cross-sectional area of the link stiffener. Stiffeners of box links need not be welded to link flanges.

6. Diagonal braces

Brace members shall satisfy width–thickness limitation for moderately ductile members.

7. Beam outside link

Where the beam outside of the link is a different section from the link, the beam shall satisfy the width–thickness limitations for moderately ductile members. Note that the diagonal brace and beam segment outside of the link are intended to remain essentially elastic under the forced generated by the fully yielded and strain hardened link. Both the diagonal brace and beam segment outside of the link are typically subject to a combination

of large axial force and bending moment and, therefore, should be treated as beam–columns in design. Where the beam outside the link is the same member as the link, its strength may be determined using expected material properties.

8. Columns
   Column members shall satisfy width–thickness limitations for highly ductile members.

   Protected zones
   Links in EBF's shall satisfy the requirements of protected zone.
9. Link-to-column connections
   1. Requirements
      Link-to-column connections shall be FR moment connections and shall satisfy the following requirements:
      a. The connection shall be capable of sustaining the link rotation angle.
      b. The shear resistance of the connection, measured at the required link rotation angle, shall be at least equal to the nominal shear strength of the link, V_n.
      c. The flexural resistance of the connection, measured at the required link rotation angle, shall be at least equal to the moment corresponding to the nominal shear strength of the link, V_n.
   2. Conformance demonstration
      Link-to-column connections shall satisfy the above requirements by one of the following:
      a. Use a connection prequalified for EBF
         Observe there are no prequalified link-to-column connections.
      b. Provide qualifying cyclic test results. Results of a least two cyclic connection tests shall be provided and are permitted to be based on one of the following:
         i. Tests reported in research literature or documented tests performed for other projects that are representative of project conditions
         ii. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection material properties

      Exception: Cyclic testing of the reinforced connection is not required if the following conditions are met:
      1. Reinforcement at the beam-to-column connection at the link-end precludes yielding of the beam over the reinforced length.
      2. The available strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon adjusted link shear strength.
      3. The link length (taken as the beam segment from the end of the reinforcement to the brace connection) does not exceed $1.6M_p/V_p$.
      4. Full-depth stiffeners are placed at the link-to-reinforcement interface.
10. Diagonal brace connections
    When OVS holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the load combinations including the amplified seismic load.

    Connections of braces designed to resist a portion of the link end moment shall be designed as FR.
11. Beam-to-column connections
    Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:
    a. The connection shall have sufficient capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height

b. The connection shall be designed to resist a moment simultaneous with the required strength of the brace connection, equal to the lesser of the following:
   i. A moment corresponding to $1.1R_yF_yZ$ of the beam
   ii. A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the column
   This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.

12. Column splices
Column splices in EBF shall be of the lesser available flexural strength of the connection members.
The required shear strength shall be $\Sigma M_{pc}/H_c$,
where
ΣM_{pc} is the sum of the nominal plastic flexural strengths $F_{yc}Z_c$ of the columns above and below the splice (kip-in.)
H_c is the clear height of the column between beam connections, including a structural slab, if present (in.)

13. Demand critical welds
The following welds shall be designed as demand critical welds:
a. Welds attaching the link flanges and the link web to the column where links connect to columns
b. Welds connecting the webs to the flanges in built-up link beams
c. Groove welds at column splices

7.2.3.4 Buckling-Restrained Braced Frames

1. Scope
In BRBF, minor eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design and do not change the expected source of system ductility: compression and tension yielding of braces.

2. Basis of design
Braces shall be designed, tested, and detailed to accommodate expected deformations. Expected deformations are those corresponding to a story drift of at least 2% of the story height or two times the design story drift, whichever is larger, in addition to brace deformations resulting from deformation of the frame due to gravity loading.
BRBF shall be designed so that inelastic deformations under the design earthquake will occur primarily as brace yielding in tension and compression.

3. Brace strength
The adjusted brace strength shall be established on the basis of testing as described in this section.
Brace connections and adjoining members shall be designed to resist forces calculated based on the adjusted brace strength.
The adjusted brace strength in compression shall be $\beta\omega R_yP_{ysc}$,
where
β is the compression strength adjustment factor
ω is the strain hardening adjustment factor
P_{ysc} is the axial yield strength of steel core (ksi)

The adjusted brace strength in tension shall be ωR_yP_{ysc}
Exception: The factor R_y need not be applied if P_{ysc} is established using yield stress determined from a coupon test.
The compression strength adjustment factor, β, shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen

measured from the qualification tests for the expected deformations. The large value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0.

The strain hardening adjustment factor, ω, shall be calculated as the ratio of the maximum tension force measured from the qualification tests (for the expected deformations) to P_{ysc} of the test specimen. The larger value of ω from the two required qualification tests shall be used. Where the tested steel core material does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

Analysis

Buckling-restrained braces shall not be considered as resisting gravity forces.

Columns, beams, and connections in BRBF shall be designed to resist seismic forces from the load combinations where the seismic load E is determined from an analysis in which all braces are assumed to resist forces corresponding to their adjusted strength in compression or in tension.

Braces shall be determined to be in compression or tension neglecting the effects of gravity loads. Analyses shall consider both directions of frame loading.

The adjusted brace strength in tension shall be using the following exceptions:

1. It is permitted to neglect flexural forces resulting from seismic drift in this determination. Moment resulting from a load applied to the column between points of lateral support must be considered.
2. The required strength of columns need not exceed the lesser of the following:
   a. The forces corresponding to the resistance of the foundation to overturning uplift
   b. Forces as determined from nonlinear analysis

   The brace deformation shall be determined from the inelastic story drift and shall include the effects of beam vertical flexibility. Alternatively, the brace deformation may be determined from nonlinear analysis.

4. V-braced and inverted V-braced frames

   V-type and inverted V-type braced frames shall meet the following requirements:

   a. The required strength of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the vertical and horizontal earthquake effect, E, on the beam shall be determined from the adjusted brace strengths in tension and compression.
   b. Beams shall be continuous between columns. Beams shall be braced to meet the requirements for moderately ductile members. As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) braces, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

      The beam has sufficient out-of-plane strength and stiffness if the beam bent in the horizontal plane meets the required brace strength and required brace stiffness for column nodal bracing, P_u, may be taken as the required compressive strength of the brace.

   For purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam.

5. K-braced frames

   K-type braced frames are not permitted for BRBF.

6. Bracing connections at beam-to-column connection

   Bracing connections at beam-to-column connection shall be designed to withstand an out-of-plane earthquake force E equal to 6% of the adjusted diagonal brace compression strength.

7. Diagonal braces
   a. Assembly
      Braces shall be composed of two elements: (1) a structural steel core and (2) a system that restrains the steel core from buckling.
      i. Steel core
         Plates used in the steel core that are 2 in. thick or greater shall satisfy the minimum notch toughness requirements.
         Splices in the steel core are not permitted.
      ii. Buckling-restraining system
         The buckling-restraining system shall consist of the casing for steel core. In stability calculations, beams, columns and gussets connecting the core shall be considered parts of this system.
         The buckling-restraining system shall limit local and overall buckling of the steel core for the expected deformations. It is to be noted that conformance to this provision is demonstrated by means of testing.
   b. Available strength
      The steel core shall be designed to resist the entire axial force in the brace.
      The brace design axial strength, φP_{ysc} in tension and compression shall be determined as follows:

$$P_{ysc} = F_{ysc} A_{sc}$$

$$\Phi = 0.90$$

      where
      F_{ysc} is the specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test (ksi)
      A_{sc} is the net area of steel core (in.^{2})

      Note: Load effects calculated based on adjusted brace strength should not be amplified by the overstrength factor, Ω_0.
   c. Conformance demonstration
      The design of braces shall be based upon results from qualifying cyclic tests. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace subassemblage that includes brace connection rotational demands, and the other shall be either a uniaxial or a subassemblage test. Both test types shall be based upon one of the following:
      i. Tests reported in research or documented tests performed for other projects
      ii. Tests that are conducted specifically for the project

   Interpolation or extrapolation or test results for different member sizes shall be justified by national analysis that demonstrates stress distributions and magnitudes of internal stains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes.
8. Width–thickness limitations of beams and columns
   Beam and column members shall meet the requirements for highly ductile members.
9. Protected zones
   The protected zone shall include the steel core of braces and elements that connect the steel core to beams and columns.

10. Diagonal brace connections
    a. Required strength
    The required strength of brace connections in tension and compression (including beam-to-column connections if part of the braced-frame system) shall be 1.1 times the adjusted brace strength in compression.
    When OVS holes are used, the required strength for the limit state of bolt slip need not exceed a load effect based upon using the load combinations, including the amplified seismic load.
    b. Gusset-plate requirements
    The design of connections shall include considerations of local and overall buckling. Lateral bracing consistent with that used in the tests upon which the design is based is required. This provision may be met by designing the gusset plate for a transverse force consistent with transverse bracing forces testing, by adding a stiffener to it to resist this force, or by providing a brace to the gusset plate. Where the supporting tests did not include transverse bracing, no such bracing is required. Any attachment of bracing to the steel core must be included in the qualification testing.
11. Column splices
    Column splices in BRBF shall be designed to develop at least 50% of the lesser available flexural strength of the connected members, determined based on the limit state of yielding. The required shear strength shall be $\Sigma M_{pc}/H_c$.
    where
    ΣM_{pc} is the sum of the nominal plate flexural strengths, $Z_c F_{yc}$, of the columns above and below the splice (kip-in.)
    H_c is the clear height of the column between beam connections, including a structural slab, if present, in.
12. Demand critical welds
    Groove welds of column splices be demand critical welds.
13. Beam-to-column connections
    Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection shall conform to one of the following:
    a. The connection shall have sufficient capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height or
    b. The connection shall be designed to resist a moment equal to the lesser of the following:
        i. A moment corresponding to $1.1R_yF_yZ$ of the beam
        ii. A moment corresponding to $\Sigma 1.1R_yF_yZ$ of the columns
    This moment shall be considered in combination with the required strength of the brace connection and beam connection, including the amplified diaphragm collector forces.

7.2.4 Special Plate Shear Walls

1. Basis of design
   SPSW are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the design earthquake. SPSW shall be designed so that inelastic deformations under the design earthquake will occur as web plate yielding and as plastic-hinge formation in the ends of horizontal boundary elements (HBEs).
2. Analysis
   The webs of SPSW shall not be considered as resisting gravity forces. HBEs, vertical boundary elements (VBEs) and connections in SPSW shall be designed to resist seismic

forces from the load combinations where the seismic load E is determined from the following analysis:

An analysis in which all webs are assumed to resist forces corresponding to their expected strength in tension at an angle α as explained presently, and HBE are resisting flexural forces corresponding to flexural forces at each end equal to $1.1R_yM_p$. Webs shall be determined to be in tension neglecting the effects of gravity loads.

The expected web yield stress shall be taken as R_yF_y.

3. Stiffness of boundary elements
   The VBEs shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c, not less than $0.0031t_wh^4/L$. The HBEs shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, not less than $0.0031L^4/h$ times the difference in web plate thickness above and below
   where
   t_w is the thickness of web (in.)
   h is the distance between HBE centerlines (in.)
   I_b is the moment of inertia of a HBE taken perpendicular to the direction of the web plate line (in.^{4})
   I_c is the moment of inertia of a VBE taken perpendicular to the direction of the web plate line (in.^{4})
   L is the distance between VBE centerlines (in.)

4. HBE-to-VBE connection moment ratio
   The beam–column moment ratio provisions shall be met for all HBE/VBE intersections without consideration of the effects of the webs.
5. Bracing
   HBE shall be braced to meet the requirements for moderately ductile members.
6. Openings in webs
   Openings in webs shall be bounded on all sides by intermediate boundary elements extending the full width and height of the panel respectively, unless otherwise justified by testing and analysis.
7. Webs
   The panel design shear strength φV_n according to the limit state of shear yielding shall be determined as follows:

$$V_n = 0.42F_yt_wL_{cf}\ \sin 2\alpha$$

$$\Phi = 0.90$$

where
t_w is the thickness of the web (in.)
L_{cf} is the clear instance between column flanges, in. (mm)
α is the angle of web yielding in radians, as measured relative to the vertical

The angle of inclination, α, is permitted to be taken as 40°, or may be calculated as follows:

$$\tan^4\alpha = \frac{1+\dfrac{t_wL}{2A_c}}{1+t_wh\left(\dfrac{1}{A_b}+\dfrac{h^3}{360I_cL}\right)}$$

h is the distance between HBE centerlines (in.)
A_b is the cross-sectional area of a HBE (in.^{2})
A_c is the cross-sectional area of a VBE (in.^{2})
I_c is the moment of inertia of a VBE taken perpendicular to the direction of the web plate line (in.^{4})
L is the distance between VBE centerlines (in.)

8. Width–thickness limitations
HBE, VBE, and intermediate boundary elements shall meet the requirements for highly ductile members.
Protected zone
The protected zone of SPSW includes the following:
   1. The webs of SPSW
   2. Elements that connect webs to HBEs and VBEs
   3. The plastic hinging zones at each end of HBEs, over a region ranging from the face of the column to one beam depth beyond the face of the column
9. Connections of to boundary elements
The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle, α.
10. HBE-to-VBE connections
The required shear strength, of a HBE-to-VBE connection shall be determined using a earthquake load effect, E, together with the shear resulting from the expected yield strength in tension of the webs yielding at an angle, α.
11. Column splices
Column splices in SPSW shall be designed to develop at least 50% of the lesser available flexural strength of the connected members, determined based on the limit state of yielding. Groove welds of column splices shall be demand critical welds.
12. Regular layout of circular perforations
A perforated plate is permitted to be used as the web of a SPSW. Perforated webs shall have a regular pattern of holes of uniform diameter spaced evenly over the entire web-plate area in an array pattern so that holes align diagonally at a uniform angle to vertical. Edges of openings shall have a surface roughness of 500 micro-inches (13 microns) or less.
   a. Strength
   The panel design shear strength, φV_n according to the limit state of shear yielding, shall be determined as follows for perforated webs:

$$V_n = 0.42 F_y t_w L_{cf} (1 - 0.7 D/S_{diag})$$

$$\Phi = 0.90$$

   where
   D is the diameter of the holes (in.)
   S_{diag} is the shortest center-to-center distance between the holes (in.)

   b. Spacing
   The spacing, S_{diag}, shall be at least $1.67D$.
   The distance between the first holes and web connections to the HBEs and VBEs shall be at least D, but not exceeding $(D + 0.7S_{diag})$.

c. Stiffness
The stiffness of such regularly perforated infill plates shall be calculated using an effective web-plate thickness, t_{eff}, is given by

$$t_{eff} = \frac{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)}{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)\left(1 - \frac{N_r D \sin\alpha}{H_c}\right)} t_w$$

where
H_c is the clear column (and web-plate) height between beam flanges (in.)
t_w is the web-plate thickness (in.)
N_r is the number of horizontal rows of perforations
α is the angle of the shortest center-to-center lines in the opening array to vertical, degrees

d. Effective expected tension stress
The effective expected tension stress to be used in place of the effective tension stress for analysis is $R_y F_y(1-0.7D/S_{diag})$.

13. Reinforced corner cut-out
Quarter-circular cut-outs are permitted at the corners of the webs provided that the webs are connected to a reinforcement arching plate following the edge of the cut-outs. The plates shall be designed to allow development of the full strength of the solid web and maintain its resistance when subjected to deformations corresponding to the design story drift. This is deemed to be achieved if the following conditions are met.

a. Design for tension
The arching plate shall have the available strength to resist the axial tension force resulting from web-plate tension in the absence of other forces.

$$P_u = R_y F_y t_w R^2/4e$$

where
R is the radius of the cut-out (in.)
$e = R(1 - \sqrt{2}/2)$ (in.)

HBEs and VBEs shall be designed to resist the tension axial forces acting at the end of the arching reinforcement.

b. Design for beam-to-column connection forces
The arching plate shall have the available strength to resist the combined effects of axial force sand moment resulting from connection deformation in the absence of other forces. These forces are

$$P_u = 15EI_y\Delta/4e^2h$$

$$M_{uy} = P_u R/2$$

where
E is the modulus of elasticity (ksi)
I_y is the moment of inertia of the plate (in.^{4})
h is the story height (in.)
Δ is the design story drift (in.)

7.2.5 Composite Systems

As many as 10 different composite systems consisting of moment frames, braced frames and shear walls are identified in the AISC 341-10. Of these, four systems pertain to moment frames and three each to braced frames and shear walls. These are

1. Composite ordinary moment frames, C-OMF
2. Composite intermediate moment frames, C-IMF
3. Composite special moment frames, C-SMF
4. Composite partially restrained moment frames, C-PRMF
5. Composite ordinary braced frames, C-OBF
6. Composite special concentrically braced frames, C-SCBF
7. Composite eccentrically braced frames, C-EBF
8. Composite ordinary reinforced concrete shear walls with steel elements, C-ORCW
9. Composite special reinforced concrete shear walls with steel elements, C-SRCW
10. Composite steel plate shear walls, C-SPW

It is generally anticipated that the overall behavior of the composite systems will be similar to that of their counterparts in steel or reinforced systems.

The ductile response is expected to occur in conventional locations such as flexural yielding of beams. As in steel systems the ductile elements can deform inelastically while the non-ductile elements remain nominally elastic. ACI 318 forms an important reference document.

7.2.5.1 Composite Ordinary Moment Frames

1. Basis of design
   C-OMF is expected to withstand minimal inelastic drift. C-OMFs comparable to reinforced concrete OMFs are only permitted in SDCs B or below in ASCE 7. This is in contract to steel OMFs, which are permitted in higher SDCs. The design requirements are commensurate with providing minimal ductility in the members and connections.
2. Connections
   FR connections shall be designed for the applicable load combinations. Beam-to-column connection design strengths shall be determined according to the Specifications and Section D2.7.

7.2.5.2 Composite Intermediate Moment Frames

1. Scope
   This system consists of moment frames with FR connections that consist of either composite or reinforced concrete columns and structural steel, concrete-encased composite or composite beams.
2. Basis of design
   C-IMF shall be designed to provide limited inelastic drift capacity through flexural yielding of the C-IMF beams, column panel zones and columns. C-IMFs, comparable to reinforced concrete IMFs, are only permitted in SDCs C or below in SEI/ASCE 7. This is in contract to steel IMFs, which are permitted in higher SDCs. The design requirements are commensurate with providing limited ductility in the members and connections.
3. Stability bracing of beams
   Beams shall be braced to meet the requirements for moderately ductile members.

   In addition, member braces shall be placed near concentrated loads, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the C-IMF.

   The required strength of stability bracing shall be provided adjacent to plastic hinges.

4. Detailing requirements
   Steel and composite members shall meet the requirements for moderately ductile members.
5. Beam flanges
   Abrupt changes in the beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is not permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges.
6. Protected zones
   The region at each end of the beam subject to inelastic straining shall be designed as a protected zone. The plastic hinge zones at the ends of C-IMF beams should be treated as protected zones. In general, the protected zone will extend from the face of the composite column to one-half of the beam depth beyond the plastic hinge point.
7. Connections
   Connections shall be FR.
8. Demand critical welds
   CJP groove welds of beam flanges, diaphragm plates, shear plates, and beam webs to columns shall be demand critical welds.
9. Beam-to-column connection requirements
   Beam-to-composite column connections used in the SFRS shall satisfy the following requirements:
   a. The connection shall be capable of sustaining an interstory drift angle of at least 0.02 rad.
   b. The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.02 rad, where M_p is defined as the nominal flexural strength of the steel, concrete-encased or composite beams.
10. Required shear strength
    The required shear strength of the connection, V_u, shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1M_{p,exp}]/l_h$$

    where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-encased or composite beams. For concrete-encased or composite beams, $M_{p,exp}$ shall be calculated using the plastic stress distribution or the strain compatibility method. Appropriate R_y factors shall be used for different elements of the cross section while establishing section force equilibrium and calculating the flexural strength. L_h shall be equal to the distance between beam plastic hinge locations, in. For steel beams, $M_{p,exp}$ may be taken as R_yM_p of the beam.
11. Continuity plates and diaphragms
    Connection diaphragm plates are permitted for filled composite columns both external to the column and internal to the column.

    Where diaphragm plates are used, the thickness of the plates shall be at least the thickness of the beam flange.

    The diaphragm plates shall be welded around the full perimeter of the column using either CJP welds or two sided fillet welds. The required strength of these joints shall not be less than the available strength of the contact area of the plate with the column sides.

    Internal diaphragms shall have circular openings sufficient for placing the concrete.
12. Column splices
    Where groove welds are used to make the splice, they shall be CJP groove welds. When column splices are not made with groove welds, they shall have a required flexural strength that is at least equal to the moment capacity ΣM^*_{pcc} of the smaller composite column. The required shear strength of column web splices shall be at least equal to $\Sigma M^*_{pcc}/H$ where

ΣM^*_{pcc} is the sum of the nominal flexural strengths of the composite columns above and below the splice. ΣM^*_{pcc} is defined as the nominal flexural strength of the composite or reinforced concrete column. For composite columns, the nominal flexural strength shall take into consideration of the required axial strength, P_{rc}. For reinforced concrete columns, the nominal flexural strength shall be calculated based on the provisions of ACI 318 with consideration of the required axial strength, P_{rc}.

7.2.5.3 Composite Special Moment Frames

1. Scope
   This system consists of moment frames with FR connections that consist of either composite or reinforced concrete columns and either structural steel or concrete-encased composite or composite beams.
2. Basis of design
   C-SMF shall be designed to provide significant inelastic drift capacity through flexural yielding of the C-SMF beams and limited yielding of the column panel zones. Except where otherwise permitted in this section, columns shall be designed to remain essentially elastic when subjected to forces associated with fully yielded and strain hardened beams or girders. Flexural yielding columns at the base shall be permitted.
3. Moment ratio
   The following relationship shall be satisfied at beam-to-column connections:

$$\Sigma M^*_{pcc} / \Sigma M^*_{p,exp} > 1.0$$

   where

   ΣM^*_{pcc} is the sum of the nominal flexural strengths of the composite columns above and below the joint at the intersection of the beam and column centerlines. ΣM^*_{pcc} is determined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used

   $\Sigma M^*_{p,exp}$ is the sum of the expected flexural strengths of the steel beams or concrete-encased composite beams at the intersection of the beam and column centerlines. $\Sigma M^*_{p,exp}$ is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\Sigma M^*_{p,exp} = \Sigma(1.1 M_{p,exp} + M_{uv})$

   M_{uv} is the moment due to shear amplification from the location of the plastic hinge to the column centerline

4. Stability bracing of beams
   Beams shall be braced to meet the requirements for highly ductile members.
   In addition, stability bracing shall be placed near concentrated forces, changes in cross section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the C-SMF.

   The required strength of stability bracing provided adjacent to plastic hinge shall have the required strength.
5. Stability bracing at beam-to-column connections
   Composite columns with unbraced connections shall satisfy the requirements of Section E3.4c(2).
6. Detailing requirements
   Steel and composite members shall meet the requirements for highly ductile members.

Exception: Reinforced concrete-encased beams shall satisfy the requirements for moderately ductile members if the reinforced concrete cover is at least 2 in. and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement shall meet the requirements of ACI 318.
Concrete-encased composite beams that are part of C-SMF shall also meet the following requirements. The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed the following requirement

$$Y_{PNA} = \frac{Y_{con} + d_b}{1 + \left(\frac{1,700F_y}{E}\right)}$$

where
Y_{con} is the distance from the top of the steel beam to the top of the concrete (in.)
d_b is the depth of the steel beam, in. (mm)
F_y is the specified minimum yield stress of the steel beams, ksi (MPa)
E is the elastic modulus of the steel beam, ksi (MPa)

7. Beam flanges
Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is not permitted unless testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges to the required interstory drift angle.
8. Protected zones
The region at each end of the beam subject to inelastic straining shall be designed as a protected zone. The plastic hinge zones at the ends of C-SMF beams should be treated as protected zones. In general, the protected zones will extend beyond the plastic hinge point.
9. Demand critical welds
CJP groove welds of beam flanges, diaphragm plates, shear plates, and beam webs to columns shall be demand critical welds.
10. Beam-to-composite column connection requirements
Beam-to-composite column connections used in the seismic load force resisting system (SLRS) shall satisfy the following two requirements:
    a. The connection shall be capable of sustaining an interstory drift angle of at least 0.04 rad.
    b. The measured flexural resistance of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 rad.
    The two requirements above shall be satisfied as follows:
    a. When beams are interrupted at the connection, the connections shall be qualified using test results. Results of at least two cyclic connection tests shall be provided and shall be based on one of the following:
        i. Tests reported in research literature or documented tests performed for other projects that represent the project conditions
        ii. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes
    b. When beams are uninterrupted or continuous through the composite or reinforced concrete column, beam flange welded joints are not used, and the connection is not otherwise susceptible to premature fracture, the performance requirements shall be demonstrated by qualifying test results in accordance with (a), or other substantiating data

Connections that accommodate the required interstory drift angle within the connection elements and provide the measured flexural resistance and shear strengths are permitted.

In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

11. Required shear strength
The required shear strength of the connection, V_u, shall be determined using the following quantity for the earthquake load E.

$$E = 2[1.1M_{p,exp}]/L_h$$

where $M_{p,exp}$ is the expected flexural strength of the steel, concrete-encased or composite beams. For concrete-encased or composite beams, L_h shall be the distance between plastic hinge locations, in.

7.2.5.4 Composite Partially Restrained Moment Frames

This system is applicable to frames that consist of structural steel columns and composite beams that are connected with partially restrained (PR) moment connections. C-PRMFs shall be designed so that under earthquake loading yielding occurs in the ductile components of the composite PR beam-to-column moment connections. Limited yielding is permitted at other locations, such as column base connections. Connection flexibility and composite beam action shall be accounted for in determining the dynamic characteristics, strength and drift of C-PRMF.

Composite beams shall be unencased, fully composite. For purposes of analysis, the stiffness of beams shall be determined with an effective moment of inertia of the composite section.

The required strength of the beam-to-column PR moment connections shall be determined considering the effects of connection flexibility and second-order moments. In addition, composite connections shall have a nominal strength that is at least equal to 50% of M_p, where M_p is the nominal plastic flexural strength of the connected structural steel beam ignoring composite action. Connections shall have a total interstory drift angle of 0.04 rad that is substantiated by cyclic testing.

In the design of PR composite connections, the bending and shear loads are considered separately. The bending resistance is assigned to a couple acting between the mild steel reinforcement in the slab and a steel angle attached to the bottom flange of the steel beam.

The connections are designed with a yield stress that is less than that of the connected members to prevent local limit states, such as local buckling of the flange in compression, web crippling of the beam, panel zone yielding in the column and bolt or weld failures.

Because the moments of inertia for composite beams in the negative and positive regions are different, the use of a weighted average is recommended.

7.2.5.5 Composite Ordinary Braced Frames

1. Scope
Columns for C-OBF shall be structural steel, encased composite, filled composite, or reinforced concrete members. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities are permitted if they are accounted for in the design.
2. Basis of design
C-OBF is expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the design earthquake. C-OBFs designed per the specifications using $R=3$, are only permitted in SDCs C or below. This is in contrast to steel ordinary braced frames, which are permitted in higher SDCs. The design requirements are commensurate with providing minimal ductility in the members and connections.

7.2.5.6 Composite Special Concentrically Braced Frames

1. Scope
   Columns in C-SCBF shall be encased or filled composite. Beams shall be either structural steel or composite beams. Braces shall be structural steel or filled composite members. This section is applicable to braced frames that consist of concentrically connected members. Minor eccentricities are permitted if they are accounted for in the design.
2. Basis of design
   C-SCBF shall be designed to provide inelastic drift capacity through cyclic buckling and tension yielding of braces when subjected to the forces resulting from the motions of the design earthquake. SCBF shall be designed so that inelastic deformations under the design earthquake will occur primarily as brace buckling and yielding in tension. Steel or composite columns or braces shall meet the requirements for highly ductile members. Steel or composite beams shall meet the requirements for moderately ductile members. In order to satisfy the compactness requirement the actual width-to-thickness ratio of square and rectangular filled composite braces may be multiplied by a factor [(0.264 + 0.0082Kl/r], for Kl/r between 35 and 90, Kl/r being the effective slenderness ratio of the brace.
3. Braces
   Structural steel and filled composite braces shall meet the requirements for SCBF. The radius of gyration shall be taken as that of the steel section alone.
   The required strength of brace connections shall meet the requirements of the load and resistance factor design (LRFD) method.
   Beam-to-column connections shall conform to one of the following:
   a. Beam-to-column connections shall meet the requirements for FR moment connections.
   b. Beam-to-column connections shall have sufficient rotation capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height.

7.2.5.7 Composite Eccentrically Braced Frames

1. Scope
   Columns in C-EBF shall be encased composite or filled composite. Beams shall be structural steel. Braces shall be structural steel. This section is applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace.
2. Basis of design
   C-EBF shall be designed so that inelastic deformations under the design earthquake will occur primarily as shear yielding in the links. Diagonal braces, columns, and beam segments outside of the link shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened link.
3. Links
   Links shall be unencased steel and shall meet the requirement for EBF links. Beams containing the link are permitted to act compositely with the floor slab using steel anchors along all or any portion of the beam if the composite action is considered when determining the nominal strength of the link.
4. Beam-to-column connections
   Beam-to-column connections shall conform to one of the following:
   a. Beam-to-column connections shall meet the requirements for FR ordinary moment connections.
   b. Beam-to-column connections shall have sufficient rotation capacity to accommodate the required rotation at a minimum story drift of 2.5% of the story height.

7.2.5.8 Composite Ordinary Reinforced Concrete Shear Walls with Steel Elements

1. Scope
   This section is applicable when reinforced concrete walls are composite with structural steel elements, either as infill panels, such as reinforced concrete walls in structural steel frames where unencased or concrete-encased structural steel sections acts as boundary members for the walls, or as structural steel or encased composite coupling beams that connect two or more adjacent reinforced concrete walls.
2. Basis of design
   Reinforced concrete walls shall meet the requirements of ACI 318 excluding Chapter 21. The design is based on limited inelastic action consistent with ACI 318 excluding Chapter 21.
3. Analysis
   Analysis shall meet the following requirements:
   a. Uncracked effective stiffness values for elastic analysis shall be assigned according to ACI 318 Chapter 10 for wall piers and composite coupling beams.
   b. When concrete-encased shapes function as boundary members, the analysis shall be based upon a transformed concrete section using elastic material properties.
   c. The flexibility of the connection between coupling beams and wall piers and the effect of shear distortions of the coupling beam and walls shall be taken into account.
   d. In coupled walls, coupling beams are permitted to yield over the height of the structure. The coupling beams–wall connection shall develop the expected flexural and shear strengths of the coupling beam.
   e. In coupled walls, it is permitted to redistribute coupling beam forces vertically to adjacent floors. The shear in any individual coupling beam should not be reduced by more than 20% of the elastically determined value. The sum of the coupling beam shear resistance over the height of the building shall be greater than or equal to the sum of the elastically determined values.
4. Boundary members
   Boundary members shall be the following requirements:
   a. The required axial strength of the boundary member shall be determined assuming that the shear forces are carried by the reinforced concrete wall and the entire gravity and overturning forces are carried by the boundary members in conjunction with the shear wall.
   b. When the concrete-encased structural steel boundary member qualifies as a composite column, it shall be designed as a composite column.
   c. Headed studs or welded reinforcement anchors shall be provided to transfer required shear strengths between the structural steel boundary members and reinforced concrete walls.
5. Steel coupling beams
   a. Structural steel coupling beams
      Structural steel coupling beams that are between adjacent reinforced concrete walls shall meet the following requirements:
      i. Steel coupling beams shall comply with the requirements or moderately ductile members.
      ii. Coupling beams shall have an embedment length that is sufficient to develop the nominal shear strength, V_n, computed from the following equation:

$$V_n = 2F_y M_p / g \le R_y V_p$$

where

V_n is the nominal shear strength, kip
$M_p = F_y Z$, kip-in.
$V_p = 0.6F_y A_{tw}$, kip
g is the coupling beam clear span, in.

iii. The embedment length shall be considered to begin inside the first layer of confining reinforcement in the wall boundary member

$$R_y V_n = 1.54\sqrt{f_c'}\left(\frac{b_w}{b_f}\right)^{0.66} \beta_1 b_f L_e \left[\frac{0.58 - 0.22\beta_1}{0.88 + g/2L_e}\right]$$

where

b_w is the thickness of wall pier (in.)
b_f is the beam flange width (in.)
f_c' is the concrete compressive strength (ksi)
L_e is the embedment length (in.)
β_1 is the factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, as defined in ACI 318

iv. Vertical wall reinforcement with nominal axial strength equal to the nominal shear strength of the coupling beam shall be placed over the embedment length of the beam with two-thirds of the steel located over the first half of the embedment length. This wall reinforcement shall extend a distance of at least one tension development length above and below the flanges of the coupling beam. It is permitted to use vertical reinforcement placed for other purposes, such as for vertical boundary members, as part of the required vertical reinforcement.

b. Composite coupling beams

Encased composite sections serving as coupling beams shall meet the following requirements:

i. Coupling beams shall have an embedment length into the reinforced concrete wall that is sufficient to develop the nominal shear strength, $V_{n,comp}$, computed from the following equation:

$$V_{n,comp} = 2M_{p,exp}/g \leq V_{comp}$$

where

$M_{p,exp}$ is the expected flexural strength of composite coupling beam. For concrete-encased or composite beams, $M_{p,exp}$ shall be calculated using the plastic stress distribution or the strain compatibility method. Appropriate R_y factors shall be used for different elements of the cross section while establishing section force equilibrium and calculating the flexural strength

V_{comp} is the nominal shear strength of the encased composite coupling beam as computed by the following equation:

$$V_{comp} = R_v V_n + (2\sqrt{f_c'} b_{wc} d + A_s f_{yt} d_c/E)$$

where

A_s is the area of transverse reinforcement (in.^{2})
b_{wc} is the width of concrete encasement (in.)
d_c is the effective depth of concrete encasement (in.)
f_{yt} is the yield strength of transverse reinforcement (ksi)
s is the spacing of transverse reinforcement (in.)

ii. The required embedment length shall be computed by using $V_{n,comp}$ instead of V_n.

7.2.5.9 Composite Special Reinforced Concrete Shear Walls with Steel Elements

1. Basis of design
   C-SRCW design is based on inelastic actions consistent with Chapter 21 of ACI 318.
2. Analysis
   Analysis shall meet the following requirements:
   a. Cracked effective stiffness values for elastic analysis shall be assigned according to ACI 318 practice for wall piers and composite coupling beams.
   b. Effects of shear distortion of the steel coupling beam shall be taken into account.
3. System requirements
   a. In coupled walls, coupling beams shall yield over the height of the structure followed by yielding at the base of the wall piers.
   b. In coupled walls, the total required compressive axial strength in a wall pier, computed as the sum of the required strengths attributed to the walls from the gravity load components of the lateral load combination plus the sum of the expected beam shear strengths increased by a factor of 1.1 to reflect the effects of strain hardening ($1.1R_yV_n$) of all the coupling beams framing into the walls, shall not exceed the axial design strength of the all at the balanced condition, P_b.
4. Ductile elements
   Coupling beams shall satisfy the requirements of protected zones. Welding on steel coupling beams is permitted for attachment of stiffeners.
5. Boundary members
   Concrete-encased structural steel boundary members that qualify as composite columns shall meet the highly ductile member requirements. Otherwise, such members shall be designed as composite compression members to meet the requirements for ACI 318 Section 10.13 including the special seismic requirements for boundary members in ACI 318 Chapter 21. Transverse reinforcement for confinement of the composite boundary member shall extend a distance of $2h$ into the wall, where h is the overall depth of the boundary member in the plane of the wall.
6. Steel coupling beams
   In addition to the requirements of Section H4/.5b, structural steel coupling beams shall meet the requirements of Section 13.5a. When required, the coupling rotation shall be assumed as 0.08 rad unless a smaller value is justified by rational analysis of the inelastic deformations that are expected under the design earthquake. Face bearing plates shall be provided on both sides of the coupling beams at the face of the reinforced concrete wall.

   Steel coupling beams shall comply with the requirements for highly ductile members. The expected shear strength for which the embedment length is calculated shall be increased by a factor of 1.1 to reflect the effects of strain hardening ($1.1R_yV_n$).

   Vertical wall reinforcement shall be confined by transverse reinforcement that meets the requirements for boundary members of ACI 318 Chapter 21.

   Embedded steel members shall be provided with two regions of vertical transfer reinforcement attached to both the top and bottom flanges of the embedded member. The first region shall be located to coincide with the location of longitudinal wall reinforcing bars closest to the face of the wall. The second shall be placed a distance of no less than d/2 from the termination of the embedment length. All transfer reinforcement bars shall be fully developed where they engage the coupling beam flanges. It is permitted to use straight, hooked or mechanical anchorage to provide development. It is permitted to use mechanical couplers welded to the flanges to attach the vertical transfer bars. The area of vertical transfer reinforcement required is computed by the following equation:

$$A_{tb} \geq 0.03 f'_c L_e b_f / f_y$$

where

A_{tb} is the area of transfer reinforcement required in each of the first and second regions attached to each of the top and bottom flanges (in.^{2})
b_f is the beam flange width (in.)
f'_c is the concrete compressive strength (ksi)
f_y is the yield strength of transfer reinforcement (ksi)
L_e is the embedment length (in.)

The area of vertical transfer reinforcement shall not exceed that computed by the following equation:

$$\Sigma A_{tb} < 0.08 L_e b_w - A_s$$

where

ΣA_{tb} is the total area of transfer reinforcement provided both the first and second regions attached to either the top or bottom flange (in.^{2})
A_s is the area of longitudinal wall reinforcement provided over the embedment length L_e (in.^{2})
b_w is the wall width (in.)

7. Composite coupling beams
For all composite encased coupling beams, the shear force, V_c, shall be calculated from the following equation:

$$V_c = 1.1 R_y V_n + 1.56 \left(2\sqrt{f'_c} b_{wc} d_c + \frac{A_s f_{yr} d_c}{s} \right)$$

8. Connections
Groove welds at column splices shall be demand critical welds.

7.2.5.10 Composite Steel Plate Shear Walls

1. Scope
Composite steel plate shear walls consist of steel plates with reinforced concrete encasement on one or both sides of the plate, or steel plates on both sides of reinforcement concrete infill, and structural steel or composite boundary members.
2. Basis of design
C-SPW is expected to withstand significant inelastic deformations in the webs. The HBEs and VBEs adjacent to the composite webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded steel webs along with the reinforced concrete webs after the steel web has fully yielding, except that plastic hinging at the ends of HBE is permitted.

Steel webs shall be designed to resists the seismic load E determined from the analysis required by the applicable building code. The analysis shall account for openings in the web.

Columns, beams, and connections in C-SPW shall be designed to resist seismic forces determined from an analysis that includes the expected strength of the steel webs in shear, $0.6 R_y F_y A_{sp}$, and any reinforced concrete portions of the wall active at the design story drift. The VBEs are permitted to yield at the base.

Steel plates with thickness less than 3/8 in. are not permitted. Boundary members shall be provided around openings in shear wall webs as required by analysis
3. Detailing requirements for HBE and VBE
Steel and composite HBE and VBE shall meet the requirements for highly ductile members.

4. Detailing requirements for web
   The design shear strength, φV_{ns}, for the limit state of shear yielding with a composite plate shall be taken as follows:

$$V_n = 0.6 A_{sp} F_y$$

$$\Phi = 0.90$$

   where
   V_n is the nominal shear strength of the steel plate (kip)
   A_{sp} is the horizontal area of stiffened steel plate (in.^{2})
   F_y is the specified minimum yield stress of the plate (ksi)

   The design shear strength of C-SPW with a plate that does not meet the stiffening requirements shall be based upon the strength of the plate.
5. Concrete stiffening elements
   The steel plate shall be adequately stiffened by encasement or attachment to a reinforced concrete panel. Conformance to this requirement shall be demonstrated with an elastic plate buckling analysis showing that the composite wall can resist a nominal shear force equal to V_{ns}.
   The concrete thickness shall be a minimum of 4 in. on each side when concrete is provided on both sides of the steel plate and 8 in. when concrete is provided on one side of the steel plate. Headed shear stud connectors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet or exceed the requirements in ACI 318. The reinforcement ratio in both directions shall not be less than 0.0025. The maximum spacing between bars shall not exceed 18 in.
6. Boundary members
   Structural steel and composite boundary members shall be designed to resist the expected shear strength of steel plate and any reinforced concrete portions of the wall active at the design story drift.
7. Detailing requirements
   HBE and VBE shall meet the requirements for highly ductile members.
8. Demand critical welds
   Welds that connect the shear wall webs to the boundary elements and groove welds at column splices shall be demand critical welds.
9. Connections of steel plate to boundary elements
   The steel plate shall be continuously welded or bolted on all edges to the structural steel framing and/or steel boundary members, or the steel component of the composite boundary members. Welds and/or slip-critical high-strength bolts are required to develop the nominal shear strength of the plate shall be provided.
10. Connections of steel plate to reinforced concrete panel
    The shear connectors between the steel plate and the reinforced concrete panel shall be designed to prevent its overall buckling. Shear connectors shall be designed to satisfy the following conditions:
    a. Tension in the connector
       The shear connector shall be designed to resist the tension force resulting from inelastic local buckling of the steel plate.
    b. Shear in the connector
       The shear connectors collectively shall be designed to transfer expected strength in shear of steel plate or reinforced concrete panel, whichever is smaller.

7.3 PREQUALIFIED SEISMIC MOMENT CONNECTION

In high seismic applications of moment frames with $R > 3$, use of SMFs or IMFs becomes mandatory. These systems are typically designed to induce plastic hinges within the beam span by moving them away from the face of columns. By this strategy, the connection at the beam–column interface remains elastic while the inelastic deformation occurs away from the connection. Thus, large inelastic strain demands at the connections that may lead to brittle failure are prevented.

The AISC Seismic Provisions provide four options for determining the suitability of a particular moment connection for use in an SMF or IMF:

1. Project-specific testing. Generally, this option is used if there is thought to be an economic advantage that can be gained through a project-specific test (or when none of the other following options works for the project).
2. Tests of similar connections that previously have been performed within the limits specified in the AISC Seismic Provisions can be used.
3. Connections that have been prequalified according to the AISC Seismic Provisions can be used.
4. A connection listed in ANSI/AISC 358-10 (AISC Prequalified Connections for Special and Intermediate Moment Frames for Seismic Applications, called the AISC Prequalified Connections for short) can be used.

When it was first introduced in 2005, AISC Prequalified Connections covered three connection types reduced beam section (RBS) moment connections, bolted un-stiffened extended end-plate (BUEEP) moment connections, and bolted stiffened extended end-plate (BSEEP) moment connections.

The AISC 358-10 has extended the number prequalified connections to six. These are as follows:

1. Reduced beam section (RBS)
2. Bolted unstiffened extended end plate (BUEEP)
3. Bolted stiffened extended end plate (BSEEP)
4. Bolted flange plate (BFP)
5. Welded unreinforced flange-welded web (WRF-W)
6. Kaiser bolted bracket (KBB)

7.4 LIST OF SIGNIFICANT TECHNICAL PROVISIONS OF AISC 341-05/10

A concise list of significant technical provisions published first in the AISC 341-05 and subsequently retrained in the AISC 341-10 are as follows:

1. Clarified that all steel buildings designed with an R factor greater than 3 must comply with the AISC Seismic Provisions.
2. Added new ASTM material specifications that are commonly used in the metal building industry.
3. Added R_t values for all materials to be used in determining susceptibility of connections to fracture failure modes.
4. Relaxed the limitations of OVS holes in bolted joints.
5. Defined a new term, “demand critical welds” that have additional quality and toughness requirements. For each system, welds considered to be demand critical are defined.
6. Defined a new term, “protected zones,” to ensure that areas expected to be subjected to significant inelastic strains are not disturbed by other building operations. For each system, areas that are considered to be protected zones are defined.

7. Made the requirements on column splices that are not part of the SLRS apply to all systems, not just moment frames.
8. Improved the provisions related to column bases.
9. Made the stability bracing requirements more consistent throughout the document.
10. Added references to the new AISC standard on Prequalification Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (ANSI/AISC 358) (AISC, 2005) as one means for SMF, IMF and EBF (link-to-column) connection acceptance. ANSI/AISC 358 is also adopted into ASCE 7-05 with Supplement No. 1 (ASCE, 2005) by reference.
11. Decreased the column splice shear capacity requirements for SMF systems.
12. Increased the stability bracing requirements for IMF systems.
13. Clarified that connections meeting the requirements for SMF or IMF systems are also acceptable for OMF applications.
14. Increased the requirements on SCBF systems that employ high Kl/r ratio braces.
15. Reduced the connection force demand on OCBF bracing to allow the use of the amplified seismic load factor.
16. Eliminate the requirement to design all members in OCBF systems for the amplified seismic load. This was done in concert with a commensurate reduction in the R factor for this system in ASCE 7-05.
17. Added special bracing configuration requirement checks for V and inverted V bracing configurations to OCBF systems.
18. For BFBF systems, increased the deformation that the system is required to demonstrate to accommodate from 1.5 to 2.0 times the design story drift.
19. Added a strong column–weak beam check for frames in SPSW systems.
20. Added requirements for welded joints including the following:
    a. Qualifications for inspection personnel
    b. Acceptable nondestructive testing procedures
    c. Intermixing of filler metals
    d. Acceptable limits of diffusible hydrogen in filler metal
    e. Maximum wind speeds for gas shielded welding
    f. Limitations on maximum interpass temperatures
    g. Proper application of weld tabs
    h. Additional requirements for demand critical welds related to welding processes, filler metal packaging and exposure, and the use of tack welds

7.5 ADDITIONAL COMMENTS ON SEISMIC DESIGN OF STEEL BUILDINGS

In the event of a large earthquake, structural steel and composite buildings, similar to their counterpart concrete buildings, are expected to dissipate seismic energy through inelastic deformations of certain designated structural elements. Their design is governed by the Seismic Provisions given in AISC 341-10. The provisions are intended to be mandatory for buildings in SDC D and higher. For buildings in SDC B, or C, the designers have a choice to either exclusively use AISC 360-05/10 with a typical R factor of 3 or assign a higher R factor and detail the system following the requirements of AISC 341-10.

For seismic design category A, SEI/ASCE 7-10 does specify lateral forces to be used as the seismic loads and effects, but these calculations do not involve the use of an R factor. Thus for Seismic Design Category A, it is not necessary to define a seismic force resisting system that meets any special requirements. Therefore, provisions of AISC 341-10 do not apply to SDC A buildings.

7.5.1 Concentric Braced Frames

Just about any bracing system that has identifiable load paths for gravity and lateral loads is permitted without any height limits for buildings assigned to SDC A, B, or C. These may be designed by

using only the provisions of AISC 360-05/10. The design of braced frames in buildings assigned to SDC D, E, or F is performed according to the Seismic Design Provisions of AISC 341-10. A brief outline of the provisions, including selected characteristics of braced frame design, is as follows:

1. A variety of braced frame configurations are permitted by AISC-Seismic. Some of these are shown in Chapter 1.
2. Subject to certain limitations, AISC-Seismic permits seismic design of braced frames either as an OCBF or as a SCBF The only difference between the two is in detailing of the connections and some prescriptive requirements for SCBF intended to enable them to respond to seismic forces with greater ductility.
3. Both the V-braced and inverted V-braced frames, often referred to as chevron braces, have been poor performances during past earthquake because of buckling of braces and excessive flexure of beam at mid-span where the braces intersect the beam. Buildings with single or multistory X-braces or V-braces with zipper columns are deemed better performers and, hence, should be considered for braced frame configurations in buildings assigned to higher SDCs.
4. Braced frames with single diagonals are also permitted by AISC-Seismic. However, there is a heavy penalty since the braces must be designed to resist 100% of the seismic force in compression, unless multiple single-diagonal braces are provided along a given brace frame line.
5. A preferred but difficult-to-achieve behavior in an SCBF is the in-plane buckling of the brace. Given a choice, a brace would buckle out-of-plane rather than buckling in the plane of the braced frame because (1) placement of braces in a flat position is generally not permitted for architectural reasons and (2) the presence of infill metal studs or other types of materials above and below the braces adds considerable in-plane stiffness to the braces. Recognizing these features, AISC-Seismic permits out-of-place buckling of braces, provided an uninterrupted yield line can develop in gusset plates at each end of the brace connection. This is achieved by prescribing the following detailing requirements:
   a. Provide a minimum of $2t$ and maximum of $4t$ offset from the end of brace to the yield line, where t is the thickness of gusset plate
   b. Provide a 1 in. minimum offset from the brace to the edge of gusset plate
   c. Isolate gusset plate yield line from the floor slab, as noted in item 6 below
6. In detailing the SCBF's gusset plate, the potential restraint that occurs due to the floor slab must be considered. To keep the gusset plate as small as possible, the plate may be isolated from the slab to allow the yield line to extend below the concrete surface. Note that the entire gusset plate does not have to be isolated, just the area where the yield line is expected to occur. A compressible material 2 to 3 in thick on each side of the plate may be used to isolate the plate, as shown in Figure 10.48.
7. Beams or columns of braced frames should not be interrupted at the brace intersections. This is to ensure out-of-plane stability of the bracing system at those locations. However, mere continuity of columns or beams at the brace intersections may not be sufficient to provide the required stability. Typical practice is to provide bracing perpendicular to the plane of brace. The beams are tied to floor diaphragms to provide out-of-plane strength and stiffness, and resistance to lateral torsional buckling of beams.
8. SCBFs are expected to achieve trilinear hysteretic behavior in a large earthquake by going through three ranges of displacement of its brace: the elastic range, the post-buckling range, and the tensile yielding range.
9. Basic design concept for an OCBF is to limit its response to elastic behavior. Therefore, a higher seismic force (using a lower value of $R = 3¼$ versus $R = 6$ for an SCBF) and lower brace capacity are used in the design. Increase in capacity is achieved by limiting $Kl/r \leq 720/\sqrt{F_y}$ versus $1000/\sqrt{F_y}$ for an SCBF.

10. Compactness requirements for braces are the same for OCBFs and SCBFs.
11. Basic design concept for an SCBF is to mitigate brittle modes of failure by controlling its behavior through better detailing. Therefore, connections are designed to develop yield capacity of brace.
12. In a chevron-configured SCBF, instead of increasing earthquake load by 1.5, the beam is designed for unbalanced load requirements by assuming one brace at the beam intersection fails in compression. This is not a design requirement for an OCBF.
13. Special requirements apply to the design of chevron-braced frames. Because braces meet at the mid-span of beams, the vertical force resulting from the unequal compression and tension strengths of these braces can have a considerable impact on the cyclic behavior of the frame. The resulting vertical force introduces flexure in the beam and possibly formation of a plastic hinge that may ensue a plastic collapse mechanism. Therefore, beams in chevron-braced frames must be continuous between columns and must be designed for the unbalanced force.
14. Seismic provisions require that beams in chevron-braced frames be capable of resisting their tributary gravity load, neglecting the presence of the braces, and that each beam in an SCBF be designed to resist a maximum unbalanced vertical load calculated using full-yield strength or the brace in tension, and 30% of the brace buckling strength in compression. In an OCBF, consideration of this latter provision need not be required. However, braces in the OCBF must be designed to have 1.5 times the strength required by load combinations that include seismic forces, which is equivalent to designing chevron-braced frames for a smaller value of R to compensate for their lower ductility.
15. To prevent instability of a beam bottom flange at the intersection of the braces in a chevron-braced frame, the top and bottom flanges of beams in both SCBF and OCBF must be designed to resist a lateral force equal to 2% of the nominal beam flange strength (i.e., $0.02A_fF_y$ where A_f is the area of beam flange). This requirement is best met by the addition of a beam perpendicular to the chevron-braced frame.
16. The preceding concepts explain why a K-type braced frame configuration is prohibited in buildings assigned to higher SDCs. The unequal buckling and tension-yielding strengths of the braces would create an unbalanced horizontal load at the mid-height of the columns, jeopardizing the ability of the column to carry gravity loads if a plastic hinge forms at the mid-height of the column.
17. CBFs are expected to undergo inelastic response during large earthquakes. Specially designed diagonal braces in these frames can sustain plastic deformations and dissipate hysteretic energy in a stable manner through successive cycles of buckling in compression and yielding in tension. The preferred design strategy is, therefore, to ensure that plastic deformations only occur in the braces, leaving the columns, beams, and connections undamaged, thus allowing the structure to survive strong earthquakes without losing gravity-load resistance.
18. The plastic hinge that forms at mid-span of a buckled brace may develop large plastic rotations that could lead to local buckling and rapid loss of compressive capacity and energy dissipation characteristic during repeated cycles of inelastic deformations. Locally buckled braces can also suffer low-cycle fatigue and fracture after a few cycles of severe inelastic deformations, especially when braces are cold-formed rectangular hollow sections. For these reasons, braces in SCBFs must satisfy the width-to-thickness ratio limits for compact sections. For OCBFs, braces can be compact or non-compact, but not slender. In particular, the width-to-thickness ratio of angles (b/t), the outside-diameter-to-wall thickness ratio of unstiffened circular hollow sections (D/t), and the outside-width-to-wall-thickness ratio of unstiffened rectangular sections must not exceed $52/\sqrt{F_y}$, $1300/\sqrt{F_y}$, and $110/\sqrt{F_y}$, respectively.
19. When a brace is in tension, net section fracture and block shear rupture at the end of the brace must be avoided. Likewise, the brace connections to beams and columns must be

stronger than the braces themselves. Using capacity design, calculation of brace strength must recognize that the expected yield strength of the brace, F_{ye}, will typically exceed its specified minimum yield strength, F_y. Thus, connections must be designed to resist an axial force equal to $R_y F_y A_g$, where R_y is the ratio of expected yield strength to specified yield strength F_y. Connections must also be able to resist the forces due to buckling of the brace. If strong connections permit the development of a plastic hinge at each end of a brace, they should be designed to resist a moment equal to $1.1R_yM_p$ of the brace in the direction of buckling. Otherwise, the connecting elements will themselves yield in flexure (such as gussets out of their plane); these must then be designed to resist the maximum brace compression force in a stable manner while undergoing the large plastic rotations that result from brace buckling. Providing a clear distance of twice the plate thickness between the end of the brace and the assumed line of restraint for the gusset plate permits plastic rotations and precludes plate buckling.

20. Beams and columns in braced frames must be designed to remain elastic when braces have reached their maximum tension or compression capacity ($1.1R_y$ times the nominal strength) to preclude inelastic response in all components except the braces. This requirement could be too severe for columns of a multistory frame because the braces along the height of the frame do not necessarily reach their capacity simultaneously during an earthquake. AISC-Seismic provisions address this issue using special load conditions with the further specification that the maximum axial tension forces in columns need not be taken larger than the value corresponding to foundation uplift. For SCBFs, the provision also requires that columns satisfy the same width-to-thickness ratio limits as braces.
21. Partial penetration groove welds in column splices have been observed to fail in a brittle manner. When a welded column splice is expected to be in tension under the loading combination, the AISC-Seismic provisions mandate that the partial joint penetration groove welded joins in SCBFs be designed to resist 200% of the strength required by elastic analysis. Column splices also need to be designed to develop at least the nominal shear strength of the smaller connected member and 50% of the nominal flexural strength of the smaller connection section.

8 Seismic Rehabilitation of Existing Steel Buildings

PREVIEW

Perhaps, the general public may not be aware, but it is common knowledge among engineers that the forces experienced in a building during a major earthquake are much larger than the forces they are typically designed for. This is because, it is neither practical nor economically feasible to design buildings to remain elastic during major earthquakes. Instead, we design buildings to remain elastic under considerably lower forces, and by using certain stringent details we expect them to ride out larger earthquakes without collapse. The rationale for this procedure is based on our observation of buildings that have lived through large earthquakes in the past.

In earthquake design, the ability of a building to withstand deformations well beyond the elastic range without losing its gravity carrying capacity is termed ductility. Ductile structures may deform excessively under seismic lateral loads but by-and-large they do not fall down. Therefore, in seismic design, providing capacity for lateral displacement well into inelastic range without causing collapse is the primary goal. What this does, in effect, is to offer engineers a range of design options to swap strength requirement for ductility.

Consider Figure 8.1 that shows idealized behavior of a building subject to three levels of forces F_L, F_U, and F_C corresponding to lower, upper, and collapse-level earthquake. Also, shown in the figure is the force F_E experienced by the structure if it were to remain elastic for the entire duration of a given earthquake. The structure designed using the lower-level earthquake force F_L deforms elastically from 0 to E, and in-elastically from E to U. The same structure designed for force F_U deforms 0 to U, responding elastically all through the displacement range. Both are capable of attaining the anticipated deformation of Δ_U. However, a building designed force F_L will require a more ductile system than a building designed for the fully elastic force F_E. It will, however, suffer heavier damage because of inelastic response, should the postulated event occur. Nevertheless, both systems achieve the primary goal: Both remain stable without collapse under the expected deformation Δ_U. Therefore, it is possible to design the structure using any reasonable level of force between F_L and F_E with the understanding that a corresponding ductility is provided for in detailing the system. For example, a structure designed for force level F_U requires a higher strength but less ductility than the one designed for force level F_L. Hence, it is a matter of choice as to how much strength can be traded off for ductility and, conversely, ductility traded for strength. Expressed another way, structural systems of limited ductility may be considered valid, provided they are capable of resisting correspondingly higher seismic forces.

This is the approach used in seismic retrofit and in Performance-Based Design, PBD. The purpose in both is to establish ductility levels that can be traded off in part, for excess strength.

There exists now a standard—Seismic Rehabilitation of Existing Buildings, ASCE/SEI 41-06—that provides an authoritative technical platform of consensus criteria on how to deal with different levels seismic rehabilitation, as may be required by geographic location, performance objective, type of building, use or occupancy, or other relevant considerations.

The core argument for seismic rehabilitation is above all, protection of life and property in future earthquakes. Other worthy goals include protecting investments, lengthening a buildings usable

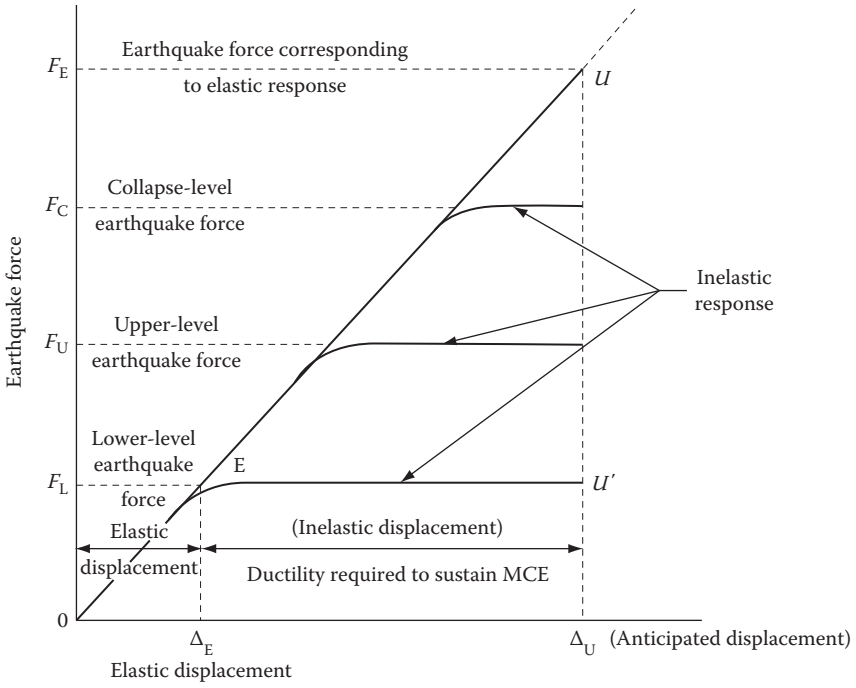


FIGURE 8.1 Idealized force–displacement relationship. Δ_U = Anticipated building displacement irrespective of ductility. This displacement can be achieved as a combination of elastic + inelastic deformation, or by a totally elastic response.

life, reducing demands on post-earthquake search and rescue resources, and shortening business interruption time. Because rehabilitation deals with existing and usually occupied buildings, the range of socioeconomic issues likely to be encountered can be formidable.

Provisions of ASCE/SEI 41-06 are based on Performance-Based Design (PBD) methodology. The performance levels are defined in terms of specifically limiting damage status. This tiered specification of building performance at predetermined earthquake intensities forms the basis for design.

We begin this chapter with a discussion of typical societal issues associated with seismic rehabilitation followed by a presentation of general steps used in the rehabilitation process. Next, analysis procedures including linear and nonlinear methods are presented with an emphasis on the so-called pseudo-lateral force procedure and corresponding target displacements.

Differences between a code-sponsored design for new construction and seismic rehabilitation of existing structures are discussed to highlight shortcomings of the former for performance evaluation of non-code complaint structures. We then move on to discuss performance-based methodology of Seismic Rehabilitation Standard, ASCE/SEI 41-06.

Seismic performance levels, earthquake risk, and the effectiveness of—and tradeoffs between—varying rehabilitation standards are explained. The text emphasizes that enhanced performance levels are only probabilistic, rather vague and somewhat abstract, while the cost of seismic rehabilitation is very real and not abstract at all. This calls for intuitive judgment rather than reliance only on analytical results.

The next section of the chapter addresses systematically the ASCE/SEI 41-06 methodology of seismic rehabilitation. Capacity versus demand comparisons for structural elements using deformation-controlled and force-controlled actions are demonstrated using realistic examples. The chapter includes a discussion on seismic base isolation and passive energy dissipation.

8.1 SOCIAL ISSUES IN SEISMIC REHABILITATION

Because rehabilitation deals with existing and usually occupied buildings, the range of socioeconomic issues likely to be encountered and to be solved can be formidable. Moreover, the intensity, nature, and complexity of such problems will vary somewhat from building to building even though sections or neighborhoods of cities and towns are slated for seismic rehabilitation. Accordingly the rehabilitation process varies depending on the demographic and socioeconomic characteristics of the designated areas.

An overriding concern in seismic rehabilitation has to do with accommodating the building's intended use. Obviously one has to accommodate the owner's intended uses of the candidate building. However, seismic rehabilitation projects often are technically tricky and part of their success depends on achieving an effective balance between improved earthquake safety and functionality.

The amount of rehabilitation needed is a function of the building type and existing conditions, the seismicity of the building site, and the Rehabilitation Objective desired by the owner.

Due to the advancement of earthquake engineering in recent decades, new codes and design standards are continually being developed and adopted. With each advance, however, the number of existing buildings that have not been designed consistent with the current technology increases. Although many of these buildings will perform well in future earthquakes, many others are likely to suffer damage, partial collapse or complete collapse, causing injuries and deaths.

Due to the cost and impracticality of implementing design codes for new buildings in the rehabilitation of existing buildings, procedures for rehabilitation will be different from those for the design of new buildings. The length of remaining building life, the consequence of economic loss due to major earthquake damage and cost-to-benefit ratio for each structure must be considered when designing seismic rehabilitation of existing buildings and proceeding with the rehabilitation. Therefore, procedures and guidelines specifically for the seismic rehabilitation of existing buildings, different from codes for new buildings are appropriate in order to permit the most effective use of existing elements and components that do not comply with modern seismic detailing requirements.

Evaluation methods are useful generally as criteria for the rehabilitation decision. Merely correcting deficiencies is a viable alternative for certain simple buildings, although it is often conservative and may be an expensive approach.

The seismic design for rehabilitation of an existing building must consider each seismic element and component of the building as well as the building system as a whole. Many of the desirable features of new buildings, such as a complete load path, redundancy, regular building configuration, and separation from adjacent buildings, which are all fundamental to satisfactory performance, may not be present in existing buildings and will have to be compensated for by the rehabilitation process.

A comprehensive rehabilitation design must include consideration of the nonstructural systems and components (such as architectural elements and mechanical and electrical systems) and the cost of their rehabilitation, as well as the structural components of the building.

8.2 GENERAL STEPS IN SEISMIC REHABILITATION

The general steps in the seismic rehabilitation process are as follows:

1. Review initial status
   Review characteristics that define the rehabilitation work for the building under investigation. This includes information such as global structural characteristics, seismic-site hazards, occupancy, historic status, and estimated range of costs for the rehabilitation work. Much of this information may be available from past evaluations. If not, it must be determined before the rehabilitation procedures can be implemented efficiently.

2. Select a rehabilitation objective
   Together with the building owner, select the desired performance level and the level of ground shaking that define the Rehabilitation Objective to be used as the basis for rehabilitation design.
3. Select a rehabilitation method
   Determine the appropriate Rehabilitation Method preliminarily by using Simplified Rehabilitation Method. Then proceed using the more comprehensive Systematic Rehabilitation Method.
4. Develop a rehabilitation strategy
   Develop a plan to mitigate structural deficiencies that considers the global characteristics of the building system.
5. Select an analysis procedure
   Select an appropriate Analysis Procedure according to the provisions of ASCE/SEI 41-06.
6. Perform preliminary rehabilitation design and analysis.
7. Verify rehabilitation design
   Check system and component acceptability and review for economic acceptability.
8. Iterate
   Refine the rehabilitation design or reconsider the Rehabilitation Objective until the system and components meet the acceptability criteria.
9. Construction documents and quality assurance
   Prepare construction documents, including plans, details, and specifications, with an appropriate quality assurance program.

8.2.1 Initial Considerations

Prior to embarking on the seismic rehabilitation process, the engineer must review and clearly understand the building characteristics that define the required rehabilitation work. This includes information such as global structural characteristics, seismic-site hazards, occupancy, historical status, and range of costs for the rehabilitation work.

Global structural characteristics: The global structural characteristics include the primary lateral-force-resisting system, other structural and nonstructural systems, the building size, and the deficiencies that are to be mitigated. Once the lateral force resisting system is identified, the extent of the rehabilitation measures necessary to mitigate the deficiencies will provide insight into the cost needed to rehabilitate the structure.

In addition to the structural concerns, the scope of the nonstructural rehabilitation, if any, must be understood. As in structural rehabilitation costs, the cost associated with nonstructural rehabilitation is driven by existing conditions and both seismicity and performance level.

Seismic-site hazards: Seismicity is defined in terms of the level of ground shaking expected at the site. Three seismicity zones are defined: low, moderate, and high. In general, the higher the seismicity, the greater the cost of the rehabilitation, as a higher level of strengthening will be needed to achieve the same performance. The potential for other seismic-site hazards such as ground failure should be determined also.

Occupancy: The engineer needs to determine whether or not the building will be occupied during the rehabilitation, and if so, what level of disruption will be tolerated by the occupants. If the building is to remain occupied, measures may have to be taken during construction to minimize disruption and, as a result, the cost of the rehabilitation may significantly increase.

Historic status: The rehabilitation of historic buildings may involve additional concerns beyond those normally considered in the rehabilitation of a typical building. Limitations may be placed on the rehabilitation strategies used, and special care may have to be taken to preserve the historic

fabric of the structure. All of these factors may contribute to increasing the cost of the rehabilitation. The advice of a specialist in historic preservation may be required.

8.2.2 Rehabilitation Objective

The building owner and the design team should meet in the early stages of a rehabilitation project to set forth the Rehabilitation Objective. The Rehabilitation Objective consists of expected Performance for a defined level of ground sharing. In selecting the Rehabilitation Objective, the engineer should carefully consider the rehabilitation costs, disruption to the occupants, and the extent of architectural remodeling in order to incorporate all aspects of the project early in the planning stages.

8.2.2.1 Performance Levels

Four Building Performance Levels are defined in ASCE 41-06. These are Operational, Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). A generalized description of the damage states associated with these performance levels is given later in this chapter.

8.2.2.2 Seismic Hazard

The most common and significant cause of earthquake damage to buildings is ground shaking. The structural response to ground shaking is dependent on many factors, such as the proximity to the fault, the soil type, and the period of vibration of the building. Ground shaking hazards for use in seismic rehabilitation can be determined in accordance with either the General Procedure, or the Site-Specific Procedure. In addition to ground shaking, other seismic-site hazards such as ground failure should also be considered.

8.2.2.3 Selecting a Rehabilitation Objective

In addition to the Basic Safety Objective (BSO), there are descriptions of Enhanced Objectives and Limited Objectives. The BSO is a benchmark that some owners may choose to exceed by setting an Enhanced Objective, and others may choose to limit by specifying a Limited Objective. The Rehabilitation Objective is distinct to each project, and preliminary technical guidance from a structural engineer, experienced in seismic design, to assist the owners in their decision, may be necessary at the beginning of the project.

The selection of the Rehabilitation Objective is entirely the choice of the owner, unless the appropriate jurisdiction has the authority to influence the selection. A few factors that may persuade the owner to select an Enhanced Objective are the occupancy type of the building, the value of the building contents, the operation of the building, or code-mandated or legislation requirements affecting the building. An Enhanced Objective may be beneficial in structures that house hospitals, fire stations, and any post-earthquake response and recovery operations. It also may be desirable for museum or buildings with historic fabric, where the cost to replace these items, if replacement is even possible, far exceeds the cost of upgrading the building to this Rehabilitation Objective. An Enhanced Objective also may be beneficial for structures that house function essential to the business, where the cost due to downtime of the operation is critical. These structures may include data processing centers, manufacturing plants, or oil refineries.

Any Rehabilitation Objective intended to provide performance inferior to that of the BSO is termed a Limited Objective. A Limited Objective may consist of either Partial Rehabilitation or Reduced Rehabilitation. Partial Rehabilitation may be chosen as a part of a program implemented as a series of incremental rehabilitations intended to result in finally achieving the BSO. Reduced Rehabilitation may be selected for buildings with limited remaining life span, or low or infrequent occupancy levels. In Reduced Rehabilitation, designers will typically prepare a rehabilitation scheme that will reduce the damage to the structure for moderate events that are likely to occur in the remaining life of the building.

8.2.2.4 Rehabilitation Method

Two basic methods for proceeding with the rehabilitation process are given in ASCE/SEI 41-06: the Simplified Rehabilitation Method and the Systematic Rehabilitation Method.

The Systematic Rehabilitation Method involves a complete analysis of the building to the specified Rehabilitation Objective. This analysis could be linear or nonlinear, static or dynamic. Choice of the analysis method is generally left to the engineer. It is appropriate to use a nonlinear procedure for irregular or complex buildings, or both. The Systematic Rehabilitation Method may be used for any structure, and requires checking the acceptability of individual components in the building, including those that are part of the lateral-force-resisting system and others such as gravity framing and nonstructural components.

8.2.2.5 Rehabilitation Strategy

Once the building deficiencies have been identified, a mitigation program, or strategy, that considers the global characteristics of the structural system must be developed. General Rehabilitation Strategies include

- Local modification of components
- Correcting existing irregularities and discontinuities
- Global structural stiffening
- Global structural strengthening
- Mass reduction
- Seismic isolation
- Supplemental energy dissipation

These strategies may each be more effective for certain buildings and Rehabilitation Objectives, and may result in substantially different rehabilitation costs. If possible, in every rehabilitation strategy, adding redundancy to the lateral-force-resisting system in the structure should be considered.

Historic buildings may require special consideration when developing a rehabilitation strategy. For example, the removal of an irregularity by adding a braced frame to balance the strength and/or stiffness of a structure may not be acceptable for a building that has historic features. Seismic isolation may be appropriate alternative to added bracing for limiting the inter-story drifts or floor accelerations.

8.2.3 Analysis Procedures

An analysis procedure that is suitable for the building and selected Rehabilitation Objective must be chosen. The analysis should be used to verify the rehabilitation strategy selected for implementation, and may need to be repeated several times with perhaps new strategies to refine the rehabilitation design.

There are four Analysis Procedures available for use with the Systematic Rehabilitation Method: the Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), and Nonlinear Dynamic Procedure (NDP).

Linear procedures are appropriate for buildings with regular configurations and those buildings that will behave in a predominantly elastic manner. If the demand-capacity ratio (DCR) of all components is less than 2.0, the linear procedure is an appropriate analysis method. Linear procedures are not recommended for those buildings for which energy dissipation or seismic isolation is a possible rehabilitation strategy, or where extensive nonlinear behavior in the structural system is expected. The LSP should not be used when the height of the building is greater than 100 ft, or where there are significant vertical or torsional irregularities.

The Nonlinear Procedures may be used for any structure. When the NDP is used, however, the project should be subject to an independent third-party review by a registered structural engineer with substantial experience in seismic design.

8.2.4 Verification of Rehabilitation Design

The philosophy behind the Analysis Procedure for the Systematic Rehabilitation Method is (1) to check the capacity of the rehabilitated lateral-force-resisting system, (2) to check the capacity of the gravity system, and (3) to check other potential areas of damage, such as the nonstructural components. The general analysis approach is to calculate the demand (force, moment, deformation, or any combination) on an individual component, resulting from the design level of ground motion, and to compare that demand with the calculated capacity of the component at the specified Rehabilitation Objective. If the demand is less than the capacity, the component is acceptable. If not, the rehabilitation design must be modified until the component demonstrates sufficient capacity.

Components are categorized as deformation-controlled or force-controlled, depending on which is most likely to be critical to the component's acceptability. If ductility is most likely to be critical, the component is deformation-controlled. Similarly, if a component is most likely to perform in a brittle manner, it is characterized as force-controlled.

8.2.5 Nonstructural Risk Mitigation

A significant portion of building performance is dependent upon the amount of nonstructural damage that may endanger lives or delay the reoccupancy of a building. The amount of nonstructural risk mitigation is more dependent on the occupancy type of a structure, than its structural building type. For instance, institutional buildings such as hospitals may have significantly more lighting and equipment bracing than a commercial building.

Cost may be incurred due to removing and replacing the various finishes that are affected by the installation of new structural members during a seismic rehabilitation effort. The architectural "cover-up" that may be needed is very difficult to determine, and a quantitative calculation method is not available.

8.2.5.1 Disabled Access Improvements

The requirements of the federally mandated Americans with Disabilities Act (ADA) may be triggered by the seismic rehabilitation work, or the owner may voluntarily choose to improve accessibility for the disabled, in and around the building.

8.2.5.2 Hazardous Material Removal

Hazardous materials such as asbestos, lead-based paint, or contaminated soil may be encountered when performing seismic rehabilitation. The removal of these materials is usually performed under contracts that are separate from those for the seismic rehabilitation, but the user may want to consider the impact of the added costs.

8.2.5.3 Design, Testing and Inspection, and Management Fees

Design fees cover the costs of design professionals such as architects, structural engineers, geotechnical engineers, civil engineers, surveyors, and cost estimators who may be required for studies and design efforts. Testing is often required to establish the strength of existing materials during the design phase. Testing and inspection are necessary during the construction phase to verify compliance with the building codes and project specifications.

8.2.5.4 Historic Preservation Costs

Historic buildings are typically placed on a city, state, or national register of historically significant buildings, and all construction and rehabilitation work on them is subject to review by a regulatory agency. Restrictions are usually placed on alteration to the historic facades, architectural columns, or special ceilings that may need rehabilitation. The historic status of a structure may influence the rehabilitation strategies employed in the seismic rehabilitation work.

8.3 SEISMIC REHABILITATION OF EXISTING BUILDINGS ASCE/SEI STANDARD 41-06

This Standard is the latest generation of performance-based seismic rehabilitation methodology. Developed from FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, it represents the state-of-art knowledge in earthquake engineering. It serves as a "technical platform" of consensus criteria on how to deal with some of the major engineering aspects of the seismic rehabilitation of buildings.

The Systematic Rehabilitation method outlined in the Standard allows practitioners to choose design approaches consistent with different levels of seismic safety as required by geographic locations, performance objective, type of building, use of occupancy, or other relevant considerations.

The goals of seismic rehabilitation are important. They include, above all, protecting life and property in future earthquakes as well as protecting investments, lengthening a building's usable life, reducing demands on post-earthquake search and rescue resources, protecting historic structures, shortening business interruption time, maintaining inventories and customers, and reducing relocation needs/demand. Other worthy goals include limiting the need for post-earthquake emergency shelter and temporary housing, minimizing the release of hazardous substances, conserving natural resources, avoiding the costly processes of settling insurance claims and applying for post-disaster aid, protecting savings and contingency funds, reducing the amount of debris to be removed, and facilitating earthquake-stricken community's return to normal patterns of activity.

A four-step iterative process may be used to outline a set of decision points to determine whether seismic rehabilitation efforts are needed and, if so, their potential scope.

The four-step decision process includes the following:

- Defining the problem by conducting preliminary and, if needed, detailed analyses of the risk.
- Developing and refining the alternatives for addressing seismic rehabilitation.
- Adopting an approach and an implantation strategy.
- Securing the needed resources and implementing the seismic rehabilitation measures.

The economic, social, and political complexities and the varying seismic environments of the United States are such that seismic rehabilitation programs will have to be tailored to thousands of individual situations.

The core argument for the seismic rehabilitation of buildings is that rehabilitated buildings will provide increased protection of life and property in future earthquakes, thereby resulting in fewer casualties and less damage than would otherwise be the case. It is a classic mitigation strategy not unlike preventive medicine. On the human level, more earthquake-resistant buildings will mean fewer deaths and injuries in an event and therefore lower demand on emergency medical services, urban search and rescue teams, fire and law enforcement personnel, utilities, and the providers of emergency shelter. In the commercial sector, less damage to structures will mean enhanced business survival and continued ability to serve customers and maintain markets or market shares. More specifically, for commercial enterprises seismic rehabilitation will better protect physical and financial assets; reduce inventory loss; shorten the business interruption period; avoid the need for relocation; and minimize secondary effects on suppliers, shippers, and other businesses involved in support services or product cycles. In short, seismic rehabilitation as a pre-event mitigation strategy actually will improve post-event response by lessening life loss, injury, damage, and disruption.

As stated earlier, rehabilitation of existing buildings to better resist future damaging earthquakes truly is "preventative medicine." While seismic rehabilitation costs money, it can significantly reduce future losses and, in economic terms, can be considered an investment to protect assets currently at risk.

Despite the fact that each building has "its own story to tell" when it comes to seismic rehabilitation, the generic, four-step process outlined earlier is generally useful. As in many processes of this type, this generic four-step model emphasizes the feedback function at every step because no existing building seismic rehabilitation effort can possibly succeed in isolation, no matter how splendid the technical components. Seismic rehabilitation takes place in a wide variety of socioeconomic and political contexts, and continuous feedback and adjustments are necessary for success.

Step 1. "Defining the Problem," actually comprises two substeps: "preliminary analysis" and "detailed analysis." Preliminary analysis entails an initial and perhaps even cursory survey of the general issues raised by an identified earthquake threat. Because earthquake-induced life and property losses tend to be concentrated in building types already known to be vulnerable, once a relatively specific degree of seismic risk and likely consequences have been identified, the issue of seismic rehabilitation arises almost immediately.

Step 2. "Develop and Refine Alternatives," involves using the data assembled under Step 1 to develop and refine alternative approaches. It provides a "menu" delineating seismic rehabilitation options and is usually a very long and involved process because it addresses key variables such as the desired performance levels and the scope of the approach.

Step 3. "Adopt and Approach and Implementation Strategy," is the decision point at which the building owner receives input from other sources and weighs the alternatives (not to be ignored is the alternative of doing nothing). It is a decision that allocates scarce resources, costs, and benefits. Finally, the decision to act sets in motion the necessary organizational routines to actually yield activity, in this case seismic rehabilitation.

Step 4. "Secure Resources and Implement" is the critical process that turns a decision to rehabilitate into its physical result—safer, more seismically resistant buildings. Without resources to carry out seismic rehabilitation, the adoption of an approach is simply "a piece of paper."

It is worth while to note that earthquake-vulnerable buildings exist worldwide, but the earthquake hazard is not uniform across the globe. Moreover, awareness of the earthquake hazard, the precursor to any action, varies even more than the hazard itself. Therefore, tackling the earthquake-vulnerable building problem takes place in an incredibly diverse set of geographic, social, economic, and political environments. Further complicating the situation is the fact that no two buildings ever seem to present exactly the same problems. Each building has its own earthquake-vulnerability profile—location, architecture, structural system, occupancy, economic role, and financing. In other words, each seismically deficient building has its own story asking for one of a kind retrofit.

Abating the risk posed by earthquake-hazardous buildings often brings into play social, economic, psychological and various other considerations that make seismic rehabilitation very complex. The most recent examples of unpleasant earthquake lessons came from the 1994 Northridge earthquake, and the 1995 Kobe earthquake, which revealed as vulnerable steel frame buildings, long believed to be the most earthquake resistant type of construction.

The damage to as many as 400 steel buildings during the north ridge earthquake and collapse of several buildings in the Kobe earthquake raised many serious questions for the design profession. Because many damaged structures were designed using the latest building codes and modern construction practices, seismic building codes for steel construction have been essentially invalidated.

In sum, earthquake teach, usually painfully if not tragically, but the learning generates state-of-the-art advances in earthquake engineering.

As stated previously, the Standard Seismic Rehabilitation of Existing Buildings, ASCE/SEI 41-06 specifies nationally applicable provisions for the seismic rehabilitation of buildings. Seismic rehabilitation is defined as improving the seismic performance of structural and nonstructural components of a building by correcting deficiencies identified in a seismic evaluation.

It is a process or methodology of evaluating deficiencies in a building, which prevent the building from achieving a selected Rehabilitation Objective. It is expected that most buildings rehabilitated in accordance with this standard would perform within the desired levels when subjected to the design earthquakes. However, compliance with this standard does not guarantee such performance; rather, it represents the current standard of practice in designing to attain this performance.

The procedures contained in ASCE/SEI 41-06 Standard are specifically applicable to the rehabilitation of existing buildings and, in general, are more appropriate for that purpose than are new building codes. New building codes are primarily intended to regulate the design and construction of new buildings; as such, they include many provisions that encourage or require the development of designs with features important for good seismic performance, including regular configuration, structural continuity, ductile detailing, and materials of appropriate quality. Many existing buildings were designed and constructed without these features and contain characteristics such as unfavorable configuration and poor detailing and preclude application of building code provisions for their seismic rehabilitation.

This standard is intended to be generally applicable to seismic rehabilitation of all buildings—regardless of importance, occupancy, historic status, or other classifications of use. However, application of these provisions should be coordinated with other requirements that may be in effect, such as ordinances governing historic structures or hospital construction. In addition to the direct effects of ground shaking, this standard also addresses the effects of local geologic site hazards such as liquefaction.

The standard is arranged such that there are four analysis procedures that can be used, including the LSP, and NDP. The linear analysis procedures are intended to provide a conservative estimate of building response and performance in an earthquake, though they are not always accurate. Since the actual response of buildings to earthquakes is not typically linear, the nonlinear analysis procedure should provide a more accurate representation of building response and performance. In recognition of the improved representation of building behavior when nonlinear analysis is conducted, the nonlinear procedures have less-conservative limits on permissible building response than do linear procedure. Building that is found to be seismically deficient based on linear analysis may comply with this standard if a nonlinear analysis is performed. Therefore, performing a nonlinear analysis can minimize or eliminate unnecessary seismic rehabilitation and potentially lower construction costs.

In addition to techniques for increasing the strength and ductility of systems, this standard provides techniques for reducing seismic demand, such as the introduction of isolation or damping devices.

However, once the decision to rehabilitate a building has been made, this standard can be referenced for detailed engineering guidance on how to conduct a seismic rehabilitation analysis and design. Featured in this standard are descriptions of damage states in relation to specific performance levels. These descriptions are intended to aid design professionals, and owners in selecting appropriate performance levels.

1. Design basis
   Provisions of ASCE/SEI 41-06 for seismic rehabilitation, as stated previously, are based on a PBD methodology that differs from seismic design procedures for the design of new buildings currently specified in national building codes and standards.

   The framework in which these requirements are specified is purposefully broad so that Rehabilitation Objectives can accommodate buildings to different types, address a variety of performance levels, and reflect the variation of seismic hazards across the United States and U.S. territories. The provisions are based primarily on and are intended to supersede FEMA 356: Prestandard and Commentary on the Seismic Rehabilitation of Buildings.

2. Rehabilitation process
   The steps for establishing seismic rehabilitation process are typically in the order in which they would be followed in the rehabilitation process. First, prior to embarking on a rehabilitation program, an evaluation is performed to determine whether the building, in its existing condition, has the desired seismic performance capability. ASCE 31-03, Seismic Evaluation of Existing Buildings, contains an evaluation methodology that may be used for this purpose. It should be noted, however, that a building may meet certain performance objectives using the methodology of ASCE 31, but may not meet those same performance objectives when an evaluation is performed using the procedures of ASCE/SEI 41-06. This is largely because ASCE 31 is specifically intended to accept somewhat greater levels of damage within each performance level than permitted by ASCE/SEI 41-06. This is consistent with the historic practice of evaluating existing buildings for slightly lower criteria than those used for design of new buildings. ASCE 31 typically quantifies this difference with the use of a 0.75 factor on demands. This essentially lowers the reliability of achieving the selected performance level from about 90% to about 60%. This practice minimizes the need to rehabilitate structures with relatively modest deficiencies relative to the desired performance level.
3. Initial considerations
   The process of building rehabilitation will be simplified and made more efficient if information that significantly affects the rehabilitation design is obtained and considered prior to beginning the process. The building owners and the design team should be aware of the range of costs and impacts of rehabilitation, including both the variation associated with different Rehabilitation Objectives and the potential additional costs often associated with seismic rehabilitation, such as other LS upgrades, hazardous material removal, work associated with the Americans with Disabilities Act, and nonseismic building remodeling.

   Seismic hazards other than ground shaking may exist at the building site. The risk and possible extent of damage from geologic site hazards identified should be considered before undertaking rehabilitation aimed solely at reducing damage due to seismic shaking. In some cases it may be feasible to mitigate the site hazard or rehabilitate the building and still meet the selected performance level. In other situations, the risk due to site hazards may be so extreme and difficult to control in which case the rehabilitation is neither cost-effective nor feasible.

   There are many ways to reduce seismic risk, whether the risk is to property, LS, or post-earthquake use of the building. The occupancy of vulnerable buildings can be reduced, redundant facilities can be provided, and non-historic buildings can be demolished and replaced. The risks posed by nonstructural components and contents can be reduced. Seismic-site hazards other than shaking can be mitigated.

   Most often, however, when all alternatives are considered, the options of modifying the building to reduce the risk of damage should be studied. Such corrective measures include stiffening or strengthening the structure, adding local components to eliminate irregularities or tie the structure together, reducing the demand on the structure through the use of seismic isolation or energy dissipation devices, and reducing the height or mass of the structure.
4. Selection of rehabilitation objective
   The concepts and terminology of PBD are relatively new. The terminology used for target Building Performance Levels is intended to represent goals of design. In most events, designs targeted at various damage states may only determine relative performance. Variations in actual performance could be associated with unknown geometry and member sizes in existing buildings, deteriorations of materials, incomplete site data, variation of ground motion that can occur within a small area, and incomplete knowledge and simplifications related to modeling and analysis.

The determination of the Rehabilitation Objective differs depending on whether the rehabilitation is mandated or voluntary. For voluntary building rehabilitation, the building owner selects a seismic rehabilitation of the building. In a mandated rehabilitation project, the rehabilitation objective is typically stipulated directly by local code or ordinance.

Building performance can be described qualitatively in terms of the safety afforded to building occupants during and after the event. These performance characteristics are directly related to the extent of damage that would be sustained by the building.

The extent of damage to a building is categorized as a Building Performance Level. A broad range of target Building Performance Levels may be selected when determining Rehabilitation Objectives.

Probabilistic Earthquake Hazard Levels frequently used and their corresponding mean return periods (the average number of years between events of similar severity) are as follows:

Earthquake Having Probability of Exceedence	Mean Return Period (Years)
50%/50 year	72
20%/50 year	225
10%/50 year	474
2%/50 year	2,475

The Rehabilitation Objective selected as a basis for design will determine, to a great extent, the cost and feasibility of any rehabilitation project, as well as the benefit to be obtained in terms of improved safety, reduction in property damage, and interruption of use in the event of future earthquakes. Table 8.1 indicates the range of Rehabilitation Objectives that may be used.

5. Basic safety objectives (BSO)
   The BSO is intended to approximate the earthquake risk to LS traditionally considered acceptable in the United States. Buildings meeting the BSO are expected to experience little damage from relatively frequent, moderate earthquakes, but significantly more damage and potential economic loss from the most severe and infrequent earthquakes that could affect them. The level of damage and potential economic loss experienced by buildings rehabilitated to the BSO may be greater than that expected in properly designed and constructed new buildings.
6. Enhanced rehabilitation objectives
   Rehabilitation that provides building performance exceeding that of the BSO is termed an Enhanced Objective. Enhanced Rehabilitation Objectives can be obtained by designing for higher target Building Performance Levels or by designing using higher Earthquake Hazard Levels, or a combination of these methods.
7. Limited rehabilitation objectives
   Rehabilitation that provides building performance less than that of the BSO is termed a Limited Objective. Limited Rehabilitation Objectives may be achieved using Reduced Rehabilitation, or Partial Rehabilitation.

   Rehabilitation that addresses the entire building structural and nonstructural systems, but uses a lower seismic hazard or lower target Building Performance Level than the BSO, is termed Reduced Rehabilitation Objective.

   Rehabilitation that addresses a portion of the building without rehabilitating the complete lateral-force-resisting system is termed Partial Rehabilitation.

TABLE 8.1
Rehabilitation Objectives

		Target Building Performance Levels			
		Operation Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
Earthquake hazard level	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~10%/50 year)	i	j	k	l
	BSE-2 (~2%/50 year)	m	n	o	p

Each cell in the above matrix represents a discrete Rehabilitation Objective.
The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives defined in Sections 1.4.1, 1.4.2, and 1.4.3, as follows:

Basic Safety Objective (BSO)	*k and p*
Enhanced objectives	*k and m, n, or o*
	p and i or j
	k and p and a, b, e, or f
	m, n, or o alone
Limited objectives	*k alone*
	p alone
	c, d, g, h, or I alone

Source: ASCE/SEI 41-06, Table C1-1.

The objectives of Limited Rehabilitation are as follows:

a. The rehabilitation measures shall not result in a reduction in the performance level of the existing building.
b. The rehabilitation measures shall not create a new structural irregularity or make an existing structural irregularity more severe.
c. The rehabilitation measures shall not result in an increase in the seismic forces to any component that is deficient in capacity to resist such forces.
d. All new or rehabilitated structural components shall be specially detailed and connected to the existing structure in compliance with seismic standards.

8. Intended (or expected) building performances
Building performance is a combination of the performance of both structural and nonstructural components. It is described in Table 8.2 with approximate limiting levels of structural and nonstructural damage that may be expected of buildings rehabilitated to the levels defined earlier. On an average, the expected damage would be less than those indicated in Table 8.2. Similarly structural performance descriptions given in Tables 8.3 and 8.4 for vertical and horizontal elements of steel buildings are estimates rather than precise predictions and variation among buildings of the same intended Building Performance must be expected.

This targeted Building Performance is discrete damage states selected from among the infinite spectrum of possible damage states that buildings could experience during an earthquake. The particular damage states identified as expected Building Performances have been selected because they have readily identifiable consequences associated with the

TABLE 8.2
Damage Control and Building Performance Levels

	Target Building Performance Levels			
	Collapse Prevention Level	**Life Safety Level**	**Immediate Occupancy Level**	**Operational Level**
Overall damage general	Severe Little residual stiffness and strength, but load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Infills and unbraced parapets failed or at incipient failure. Building is near collapse	Moderate Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls or tipping of parapets. Some permanent drift. Damage to partitions. Building may be beyond economical repair	Light No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable	Very light No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional
Nonstructural components	Extensive damage	Falling hazards mitigated but many architectural, mechanical, and electrical systems are damaged	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities	Negligible damage occurs. Power and other utilities are available, possibly from standby sources

Source: ASCE/SEI 41-06, Table C1-2.

post earthquake disposition of the building that are meaningful to the building community. These include the ability to resume normal functions within the building, the advisability of post earthquake occupancy, and the risk to LS.

Due to inherent uncertainties in prediction of ground motion and analytical prediction of building performance, some variation in actual performance should be expected. Compliance with the methodology of ASCE/SEI 41-06 should not be considered a guarantee of performance.

ASCE/SEI-06 endorses the use of PBD solutions for seismic rehabilitation of buildings. The chosen performance of the building may vary from preventing collapse to a near-perfect building that would survive an expected earthquake without a scratch. The standard allows owners to select their desired performance objective and permits designers to choose their own approaches to achieve the desired results rather than strictly adhering to the prescriptive requirements of codes. Instead of dictating how to achieve a given design goal, PBD emphasizes the goals that must be met and sets the criteria for acceptance. This way, engineers are free to innovate without running afoul of specific code provisions, within certain limits.

The ASCE/SEI 41-06 outlines criteria and methods for ensuring the desired performance of buildings at various performance levels selected by the owners with input from their design professionals. The guidelines allow owners to select a level of seismic upgrade that not only protects lives, a goal of all building codes, but also protects their investment.

ASCE/SEI 41-06 is a radical departure from current practice in that it seeks to provide the structural engineering profession with tools to explicitly, rather than implicitly, design for multiple,

TABLE 8.3
Structural Performance Levels and Damage: Vertical Elements

		Structural Performance Levels		
Elements	**Type**	**Collapse Prevention**	**Life Safety**	**Immediate Occupancy**
Steel moment frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members
	Secondary	Same as primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent
Braced steel frames	Primary	Extensive yielding and budding of braces. Many braces and their connections may fail	Many braces yield or buckle but do not totally fail. Many connections may fail	Minor yielding or buckling of braces
	Secondary	Same as primary	Same as primary	Same as primary
	Drift	2% transient or permanent	1.5% transient; 0.5% permanent	0.5% transient; negligible permanent
Foundations	General	Major settlement and tilting	Total settlements <6 in. and differential settlements <1/2 in.	Minor settlement and negligible tilting

Source: Table C1-3, ASCE/SEI 41-06. See Table C1-3 for footnotes a–c.

specifically defined, levels of performance. These performance levels are defined in terms of specifically limiting damage states, against which a structure's performance can be objectively measured. Recommendations are developed as to which performance levels should be attained by buildings of different occupancies and use. This tiered specification of performance levels at predetermined earthquake hazard levels becomes the design performance objective and a basis for design. It recognizes the importance of the performance of all the various component systems to the overall building performance and defines a uniform methodology of design to obtain the desired performance.

8.3.1 Overview of Performance Levels

ASCE/SEI 41-06 sets forth a menu of four rehabilitation objectives associated with four earthquake hazard levels. The rehabilitation objectives are

- Operational performance
- IO performance
- LS performance
- CP performance

Each of these performance levels is associated with defined levels of damage to structural, architectural, mechanical, and electrical building components as well as tenant furnishings. The designer is

TABLE 8.4
Structural Performance Levels and Damage[a,b]: Horizontal Elements

	Structural Performance Levels		
Element	**Collapse Prevention (S-5)**	**Life Safety (S-3)**	**Immediate Occupancy (S-l)**
Metal deck diaphragms	Large distortion with buckling of some units and tearing of many welds and seam attachments	Some localized failure of welded connections of deck to framing and between panels. Minor local buckling of deck	Connections between deck units and framing intact minor distortions

Source: Table C 1-4, ASCE/SEI 41-06.

[a] Damage states indicated in this table are provided to allow an understanding of the severity of damage that may be sustained by various structural elements where present in structures meeting the definitions of the Structural Performance Levels. These damage states are not intended for use in post-earthquake evaluation of damage or for judging the safety of, or required level of repair to, a structure following an earthquake.

[b] Drift values, differential settlements, and similar quantities indicated in these tables are not intended to be used as acceptance criteria for evaluating the acceptability of a rehabilitation design in accordance with the analysis procedures provided in this standard; rather, they are indicative of the range of drift that typical structures containing the indicated structural elements may undergo when responding within the various Structural Performance Levels. Drift control of a rehabilitated structure may often be governed by the requirements to protect nonstructural components. Acceptable levels of foundation settlement or movement are highly dependent on the construction of the superstructure. The values indicated are intended to be qualitative descriptions of the approximate behavior of structures meeting the indicated levels.

referred to ASCE/SEI 41-06 Tables C1.3 through C1.7 for an overview of where each performance level falls within the overall spectrum of possible damage states. From these tables, the designer may infer, for example, a building designed for top-of-the-line performance using higher earthquake hazard levels is likely to come out scratch-free, delivering performance well above the code minimum for LS level. On the other hand, much less is expected of a building rehabilitated to a CP performance level. It is deemed to have fulfilled its obligations if it remains standing during and after a large earthquake: Any other damage or loss is acceptable.

The four levels of earthquake levels hazard recognized in the development of design performance objectives are

1. Frequent earthquakes, having a 50% chance of exceedence in 30 years (43 year mean return period).
2. Occasional earthquakes, having a 50% chance of exceedence in 50 years (72 year mean return period).
3. Rare earthquakes, having a 10% change of exceedence in 50 years (475 year mean return period). Also called basic safety earthquake (BSE-1) and design basis earthquake (DBE).
4. Very rare earthquakes, having a 10% chance of exceedence in 100 years (950 year return period). Also called basic safety earthquake (BSE-2) and maximum considered earthquake (MCE).

In ASCE/SEI 41-06, a series of design parameters and acceptance criteria are given for each performance level for the various structural and nonstructural components. Design response parameters are defined at an element level in terms of element forces, interstory drifts, and plastic rotations. These can be derived from a structural analysis of building response to a particular design earthquake. Acceptance criteria are the limiting values for design parameters in order to attain a given performance level. For example, if interstory drift ratio is a design parameter used for a certain class of building, acceptance criteria would be the drift ratios defined for each performance level.

Typical drift ratios normally considered in design are 0.020 for the near collapse level, 0.015 for the LS level, 0.01 for the operational level, and 0.005 for the fully operational level. A wide variety of potential design parameters may need to be defined including deformation, strength, and energy-based parameters. The purpose of SEI 41-06 is to provide a consensus-backed, professionally accepted, nationally applicable, seismic rehabilitation standard. It can be used as a tool by design professionals, a reference document by building regulatory officials, and a foundation for the future development and implementation of building code provisions and standards related specifically to existing buildings. The absence of such a standard has been the primary barrier to widespread seismic upgrading of buildings in the United States.

In new buildings, the structural system can be controlled to fit a set of preconditions or a configuration to satisfy the design objectives prescribed by building codes. The degree of nonlinear behavior can be designed to be consistent throughout the structural system, allowing a single seismic reduction factor, R, to be used for the entire building.

Experience in seismic design over the past 100 years has shown that buildings designed to resist ground shaking from an earthquake with a 10% chance of exceedence in 50 years, at an LS level of performance, have been able to resist the strongest earthquake without collapse. This experience has given structural engineers enough confidence to design new structures in which ductile details are specified, properties of materials used in construction are controlled, and stringent requirements of testing and inspection are specified.

Assessing the seismic vulnerability of existing buildings is an entirely different problem. This is because, for existing buildings, structural details and the properties of materials must be confirmed or assumed from available information augmented by testing and inspection. Conservative assumptions consistent with the quality of the information available must be made prior to seismic evaluation. The engineer has no control over the structural system or its configuration. The existing building may not fit prescriptive details to permit code-type analysis. Nonlinear behavior of the components of the structural system will probably not be consistent. Thus, the properties of each component must be separately studied. Because of the inconsistent levels of reserve capacity in existing buildings and the differences between the 10% in a 50-year earthquake and the MCE in various regions of the country, it is inappropriate that rehabilitated buildings be designed to resist a single level of earthquake shaking. Therefore, using an entirely different approach, SEI-41 provides a basis of rehabilitation designs for a variety of structural performance levels, ranging from enhanced performance to CP. It emphasizes the idea that seismic rehabilitation should be directed to controlling deformation in order to minimize damage. Use of all existing seismic resistance is permitted in the evaluation. Acceptance criteria tailored to recognize the deformation capacity of all existing as well as enhanced or new components are provided.

The seismic loads used in the evaluation are based on a suite of USGS-developed acceleration maps including four key maps. Two of these are BSE-1 (basic safety earthquake-1) maps of acceleration response spectra having a 10% probability of exceedence in 50 years. The other two are BSE-2 (basic safety earthquake-2) maps of acceleration response spectra for the MCE—modified 2% probability of exceedence in 50 year maps: Both BSE-1 and BSE-2 maps are given for 0.2 s period (short period) and 1 s period buildings.

8.3.2 Permitted Design Methods

Two methods are permitted by ASCE/SEI 41-06, a simplified method and a systematic method. The simplified approach is for the rehabilitation design of small buildings of regular configuration, and is intended to fulfill limited objectives. Partial rehabilitation measures that seek to eliminate high-risk building deficiencies such as exterior falling hazards are included in the technique.

The systematic rehabilitation method discussed at length in this section is applicable to any building. It is a component- and element-based design. In this method, global seismic response of the building is sought with unreduced seismic loads (i.e., with a global R-factor of unity). In the

seismic evaluation, all components and seismic elements are considered with their individual deformation and force-resisting characteristics. It is a deformation-based design with the explicit rather than tacit acknowledgment that seismic elements and components behave in a nonlinear manner.

Any of the following analysis procedures may be used in the rehabilitation study and upgrade design:

- LSP. This procedure replaces the equivalent lateral force procedure included in most seismic design codes. It incorporates techniques for considering the nonlinear response of individual seismic elements. The distribution of forces is similar to equivalent lateral force procedures for new buildings.
- LDP. In this method, the modeling and acceptance criteria are similar to those of LSP. However, calculations are carried out using modal spectra analysis or time history analysis using response spectra or time-history records that are not modified to account for inelastic response for distribution of forces.
- NSP. This method is frequently referred to as a pushover analysis. It has been in use for some time without specific guidance from building codes and standards regarding modeling assumptions and acceptance criteria. This is now alleviated to some extent because previously FEMA 356, and currently ACSE/SEI 41-06 have set forth specific procedures.
- NDP. The modeling approaches and acceptance criteria for this method are similar to those of NSP. It differs from NSP in that response calculations are made using inelastic time history dynamic analysis to determine distribution of forces and corresponding internal forces and system displacements. Peer review by an independent engineer with experience in seismic design and nonlinear procedures is recommended and often times mandated because this method requires assumptions that are not included in ASCE/SEI 41-06.

8.3.3 Systematic Rehabilitation

The process of arriving at a systematic rehabilitation design includes the following steps:

1. Determination of seismic ground motions
2. Determination of as-built conditions
3. Classification of structural components into primary and secondary components
4. Setting up of analytical models and determination of design forces
5. Ultimate load combinations; combined gravity and seismic demand
6. Component capacity calculations, Q_{CE} and Q_{CL}
7. Capacity versus demand comparisons
8. Development of seismic strengthening strategies

First, the seismic hazard for the site is established by determining the probable ground shaking (spectral acceleration) from either seismic hazard maps or a site-specific investigation. Other site hazards such as liquefaction, lateral spreading, and land sliding are determined from site reconnaissance, existing documentation, or a subsurface investigation.

The desired performance level is then established. This requires close communication with the client, using damage descriptions for each performance level as a tool to get ideas across. The damage descriptions associated with each performance level can be used to inform and assist the client to make a decision of the preferred performance level. This distinction is required because the acceptance criteria are different for each type of component. The primary components are parts of the building's lateral-force-resisting system, whereas the secondary components are those not required for lateral-force resistance, although they may actually resist some lateral forces. The analysis is performed by considering general requirements such as $P\Delta$ effects, torsion, overturning, continuity, integrity of elements, and building separations.

New or modified components are evaluated using the same standards as existing components, and the designs are completed by comparing capacities with demands for each component. The components and connections are redesigned where demand exceeds capacity and analysis is iterated to confirm the design. Nonstructural components are verified for the performance level and rehabilitation objective selected.

It should be noted that selection of a rehabilitation strategy follows confirmation of seismic deficiencies. From among many possible strategies, the strategy most likely to meet requirements is selected. Some possible strategies are modification of components, removal of irregularities and discontinuities, global strengthening and stiffening, mass reduction, seismic isolation, and energy dissipation.

8.3.3.1 Determination of Seismic Ground Motions

Two characteristic earthquakes, referred to as BSE-1 and BSE-2, are of particular importance. These generally correspond to return periods of 474 and 2475 years, respectively, and are commonly referred to as earthquakes with a 10% chance of exceedence in 50 years and a 2% chance of exceedence in 50 years, respectively. At sites close to major faults, the probabilistic estimates of ground motion are capped by deterministic ones. The engineer has three choices for determining the acceleration response spectra corresponding to these earthquakes: (1) use spectral response acceleration contour maps developed by the USGS; (2) use CD-ROM available from the USGS; or (3) engage a geotechnical engineer to develop site-specific response spectra based on the geologic, seismologic, and soil characteristics associated with the specific site. For some sites option three may be the only permitted method. However, to define as precise a seismic demand as possible, it is common practice to engage a geotechnical engineer to perform a site-specific study for developing response spectra corresponding to specific return periods. The geotechnical report also typically addresses other seismic hazards such as liquification, lateral spreading, or potential for land sliding at the site.

It should be noted that acceleration response spectra for earthquake hazard levels corresponding to probabilities of exceedence other than the BSE-1 and BSE-2 earthquakes can be determined by following procedures specified in ASCE/SEI 41-06.

8.3.3.2 Determination of As-Built Conditions

In this step, the following tasks are performed:

- Field observation
- Review of available documents, including plans, specifications geotechnical reports, shop drawings, test records, and maintenance histories
- Review of information regarding material standards and construction practices for location and date of construction
- Destructive and nondestructive testing of selected building components for determination of material properties and configuration of details
- Interviews with people knowledgeable about the building (i.e., owners, tenants, maintenance personnel, architects, engineers, and builders)

As a measure of the knowledge gained from this investigation, engineers assign a numerical value to the knowledge coefficient κ. ($\kappa = 1.0$ if the available information is reliable; if not, $\kappa = 0.75$).

8.3.3.3 Primary and Secondary Components

Before setting up the analytical model, structural components are classified as either primary or secondary. Primary components are those that provide the structure's basic lateral resistance. Secondary components are those that do not, and as such are permitted to experience more damage and displace more than the primary components. Additionally, components are

further classified as either deformation-controlled, if they are capable of sustaining the loads when strained inelastically, or force-controlled, if they are not capable of sustaining load when strained inelastically.

8.3.3.4 Setting Up Analytical Model and Determination of Design Forces

An analytical model of the building is set up to represent the structure's dynamic behavior. Although two-dimensional models may be adequate, current practice is to use three-dimensional models to account for torsion, plan and vertical irregularities, and nonuniform distribution of building mass. Only the primary components are modeled, with the stipulation that the secondary elements, if used in the model, cannot exceed 25% of the total structural stiffness. If they do, then some of the secondary components must be reclassified as primary components.

8.3.3.4.1 Calculation of Building Period

The building period, T, is calculated by using (1) modal analysis procedure, Method 1, or (2) empirical equations, Method 2, or (3) Rayleigh's method, Method 3. Method 1 is the preferred method. The fundamental period T is obtained by an eigenvalue analysis using the analytical model. This is the more commonly used method, particularly in seismic vulnerability studies.

In Method 2, the period T is determined using the following equation:

$$T = C_t h_n^{\beta} \tag{8.1}$$

where

T is the fundamental period, in seconds, in the direction under consideration
$C_t = 0.035$ for steel moment frames
$= 0.030$ for steel eccentrically braced frames
$= 0.020$ for all other framing systems
h_n is the building height, in feet, above shear base
$\beta = 0.80$ for steel moment frames
$= 0.75$ for all other framing systems

However, there is a major difference worthy of note between building code procedures for new buildings and the ASCE/SEI 41-06 approach. Unlike the codes, there is no maximum limit on period calculated using Method 1. The intent of this omission is to encourage the use of more advanced analysis such as computer dynamic analysis. It is believed that sufficient controls on analysis and acceptance criteria are present within the ASCE/SEI 41-06 to provide reasonably conservative results even though there is no upper limit for the period obtained by Method 1.

8.3.3.4.2 Determination of Pseudo-Lateral Load (Base Shear)

The base shear, also referred to as pseudo-lateral force, for use in the design of new components and in the verification of existing components of the lateral-force-resisting system is given by

$$V = C_1 C_2 C_m S_a W \tag{8.2}$$

where

V is the pseudo-lateral load (the base shear)
C_1 is the modification factor that accounts for the difference in the structure's elastic and inelastic displacement amplitude. Its value is equal to 1.0 for $T > 1.0$ s. (See ASCE/SEI 41-06 Section 3.3.1.3 for further discussion of parameters C_1, C_2 and C_m.)
C_2 is the modification factor that represents the effect of strength and stiffness degradation of the components on maximum displacement response. For periods greater than 0.7 s, C2 = 1.0

C_m is the effective mass factor to account for higher modal mass participation equal to 1.0 if building fundamental period T is greater than 1.0 s (see ASCE/SEI 4-06, Table 3.1 for additional information)
= 0.9 for three or more story concrete frame buildings
= 0.8 for three or more story concrete shear wall or pier-spandrel buildings
S_a is the response spectrum acceleration at the fundamental period and damping ratio of the building
W is the effective seismic weight of the building, including the total dead load and applicable portions of other gravity loads listed as follows:

- In storage and warehouse occupancies, a minimum of 25% of floor live load.
- An allowance for partition load equal to the actual partition weight or a minimum of 10 psf of floor area, whichever is greater.
- The total operating weight of permanent equipment.
- The effective snow load equal to 20% of the design snow load if design snow load exceeds 30 psf. If not, the effective snow load may be taken to be zero.

8.3.3.4.3 Vertical Distribution of Base Shear

The lateral force F_x, applied at any level x, is determined in accordance with the following equations

$$F_x = C_{vx}V \tag{8.3}$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \tag{8.4}$$

where
C_{vx} is the vertical distribution factor
V is the pseudo-lateral force (base shear)
w_i and w_x are the portion of the total gravity load of the building W located or assigned to level i or x
h_i and h_x are the height in feet from the base to level i or x
k is an exponent related to the building period as follows:

If the building period T is 0.5 s or less, $k = 1$
If the buildings period T is 2.5 s or more, $k = 2$
Linear interpretation is used for intermediate values of the period T

8.3.3.4.4 Diaphragm Design Force F_{px}

Floor and roof diaphragms shall be designed to resist the combined effects of the inertial force F_{px} calculated at the diaphragm level combined with the horizontal forces resulting from offsets in the vertical seismic elements above and below the diaphragm. The design force shall not be less than

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_x + V_{px} \tag{8.5}$$

where

V_{px} is the horizontal force received by the diaphragm from discontinuous vertical elements, if any
F_{px} is the total diaphragm inertial force at level x
F_i is the lateral load at level i
w_i is the portion of the effective seismic weight W
w_x is the portion of the effective seismic weight W located at or assigned to floor level x

8.3.3.5 Combined Gravity and Seismic Demand

In this step, the earthquake actions Q_E obtained from the analysis for the unreduced response spectra are combined with the gravity actions, Q_G, to determine the demand imposed on the component. When the effects of gravity and seismic loads are additive, an upper-bound value for gravity loads is estimated by using the following load combinations:

$$Q_G = 1.1(Q_D + Q_L + Q_s) \tag{8.6}$$

And when the effects of gravity and seismic loads are counteracting, a lower-bound value of the gravity load is estimated by using 90% of the dead load:

$$Q_G = 0.9Q_D \tag{8.7}$$

where

Q_D is the dead load action
Q_L is the effective live load action equal to 25% of the unreduced design live load but not less than the actual live load
Q_s is the effective live load action equal to 20% of the design snow load where the design snow load exceeds 30 psf.

No part of the snow load need be included if the design snow load is less than 30 psf. Next, the gravity and seismic loads are combined together to determine the demand using the following equations:

For deformation-controlled actions:

$$Q_{UD} = Q_G \pm Q_E \tag{8.8}$$

For force-controlled actions:

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 C_2 J} \tag{8.9}$$

where

Q_{UD} is the deformation-controlled demand due to gravity loads and earthquake loads
Q_{UF} is the force-controlled demand due to gravity loads in combination with earthquake loads
J is the force delivery reduction factor, greater than or equal to 1.0. This factor is used to estimate the actual forces delivered to force-controlled components by other yielding components. The values of J are

J = 2.0 in zones of high seismicity
= 1.5 in zones of moderate seismicity
= 1.0 in zones of low seismicity

Alternatively, J may be taken as the smallest DCR for the components in the load path delivering force to the component being designed. The minimum value of J, the force-delivery reduction factor, is 1.0. The reader is referred to SEI 41-06, Section 3.4 for further definition of terms used in this section.

8.3.3.6 Component Capacities Q_{CE}, Q_{CL} and Design Actions

ASCE/SEI 41-06 specifies two different equations for evaluating component capacities depending upon whether the action of the component is deformation-controlled (Q_{CE}) or force-controlled (Q_{CL}). The subscript E in Q_{CE} stands for expected capacity, where as L in Q_{CL} stands for lower-bound capacity. The subscript C in both Q_{CE} and Q_{CL} stands for capacity. The term "design action" is used to define forces and moments in the components due to seismic and gravity effects.

The two types of actions—deformation-controlled and force-controlled—are defined in ASCE 41-06 by using as many as 25 equations. The reader is referred to the source document for further details.

Deformation-controlled actions in simple terms refer to forces and moments in a component that has recognizable nonlinear deformation characteristics. Because of possible anticipated nonlinear response, the design forces and moments in the component are permitted to exceed their plastic capacity. The acceptance criteria, discussed presently, take this into account through the use of an m-factor, which, in a conceptual sense, is an indicator of the nonlinear deformation capacity of the component.

Force-controlled actions differ from deformation-controlled actions in that they do not have a recognizable inelastic response. Therefore, demands for force-controlled actions must not exceed the calculated capacity (i.e., there are no m-factors in the acceptance criteria). It should be noted, however, that the calculated design force (demand) itself is reduced by the C_1, C_2, C_3, and J factors before demand is compared to capacity.

An ideal procedure for determining the magnitude of force-controlled actions is by identifying an inelastic limit state for the component and then, by statics, evaluation of the corresponding force-controlled action. For example, seismic shear in a frame beam is determined from equilibrium considerations of a free-body diagram of the beam with a moment equal to the expected moment strength plus gravity moments.

However, it is acceptable to determine force-controlled actions from equation given earlier, where it is not possible to identify a well-defined limit state.

8.3.3.6.1 Sample Capacity Calculations

Given

A W27 × 194 frame beam in an existing steel building constructed in 1992. The specified material is ASTMA-36, dual grade.

Required

Capacity Q_{CE} of the frame beam.

Solution

First, determine from Tables 8.5 and 8.6 (ASCE/SEI 41-06 Tables 5.1 and 5.2), the lower bound yield strength of steel manufactured in the year 1992 under ASTM A36 for structural size grouping 3 (see AISC Manual of Steel Construction, 9th edn., Table 2). This is equal to 52 ksi. Second, from Table 8.7 (ASCE/SEI 41-06 Table 5.3), find the value of the factor that translates lower-bound steel

TABLE 8.5
Default Lower-Bound Material Strengths for Archaic Materials[a,b]

Year	Material	Lower-Bound Yield Strength (ksi)	Lower-Bound Tensile Strength (ksi)
Pre-1900	Cast iron	18	—
Pre-1900	Steel	24	36

[a] Modified from unit stress values in AISC *Iron and Steel Beams from 1873 to 1952* (AISC 1983).
[b] Properties based on tables of allowable loads as published in mill catalogs.

properties to expected-strength properties. This is equal to 1.05. Next, the expected yield strength is obtained by multiplying the lower-bound yield strength of 52 ksi and the translation factor of 1.05. Thus, for the given W27 × 194 frame beam, $F_{ye} = 52 \times 1.05 = 54.6$ ksi.

The expected capacity Q_{CE} of the beam is given by

$$Q_{CE} = M_{CE} = M_{PCE} = ZF_{ye} \quad (Z \text{ is the plastic modulus} \le 628 \text{ in.}^3 \text{ for W27} \times 194)$$

$$= 628 \times 54.6 = 34,288 \text{ k-in.}$$

$$= 2857.4 \text{ k-ft}$$

8.3.3.7 Capacity versus Demand Comparisons

In this step, the component capacities are compared with the demand due to earthquake and gravity loads. If the capacity of a component exceeds the demand due to seismic and gravity load combinations, the component is judged to satisfy the performance criteria. If not, a more refined technique such as a pushover analysis is performed before declaring the component deficient.

Two equations are given in ASCE/SEI 41-06 for verifying the acceptance criteria.

$$\text{For deformation-controlled actions: } m\kappa \; Q_{CE} \ge Q_{UD} \tag{8.10}$$

$$\text{For force-controlled actions: } \kappa \; Q_{CL} \ge Q_{UF} \tag{8.11}$$

where

- m is the modifier given in Tables 5-5 of ASCE 41-06 for linear procedure. An abbreviated version of one such table is given here in Table 8.8. The table takes into account the expected ductility of the component action at the selected structural performance level
- Q_{CE} is the expected strength of the component at the deformation level under consideration for deformation-controlled actions
- κ is the knowledge factor defined earlier in Section 8.3.3.2
- Q_{CL} is the lower-bound strength of a component for force-controlled actions
- Q_{UD} is the deformation-controlled demand due to gravity and earthquake loads
- Q_{UF} is the force-controlled demand due to gravity and earthquake loads

TABLE 8.6
Default Lower-Bound Material Strengths[a,b]

Date	Specification	Remarks	Tensile Strength[c] (ksi)	Yield Strength[c] (ksi)
1900	ASTMA9	Rivet steel	50	30
	buildings	Medium steel	60	35
1901–1908	ASTMA9	Rivet steel	50	25
	buildings	Medium steel	60	30
1909–1923	ASTMA9	Structural steel	55	28
	buildings	Rivet steel	46	23
1924–1931	ASTMA7	Structural steel	55	30
		Rivet steel	46	25
	ASTMA9	Structural steel	55	30
		Rivet steel	46	25
1932	ASTM A140-32T issued as a tentative revision to ASTMA9 (buildings)	Plates, shapes, bars	60	33
		Eyebar flats (unannealed)	67	36
1933	ASTM A140-32T discontinued and ASTM A9 (buildings) revised October 30,1933	Structural steel	55	30
	ASTM A9 tentatively revised to ASTM A9-33T (buildings)	Structural steel	60	33
	ASTM A141-32T adopted as a standard	Rivet steel	52	28
1934–present	ASTM A9	Structural steel	60	33
	ASTMA141	Rivet steel	52	28
1961–1990	ASTMA36/A36M-04	Structural steel		
	Group 1		62	44
	Group 2		59	41
	Group 3		60	39
	Group 4		62	37
	Group 5		70	41
1961–present	ASTM A572/A572M-04, grade 50	Structural steel		
	Group 1		65	50
	Group 2		66	50
	Group 3		68	51
	Group 4		72	50
	Group 5		77	50
1990–present	ASTM A36/A36M-04 and dual grade	Structural steel		
	Group 1		66	49
	Group 2		67	50
	Group 3		70	52
	Group 4		70	49
1998–present	ASTMA992/A992M-04	Structural steel	65	50

Source: ASCE/SEI 41-06, Table 5-2.

[a] Lower-bound values for material prior to 1960 are based on minimum specified values. Lower-bound values for material after 1960 are mean minus one standard deviation values from statistical data.

[b] Properties based on ASTM and AISC structural steel specification stresses.

[c] The indicated values are representative of material extracted from the flanges of wide flange shapes.

TABLE 8.7
Factors to Translate Lower-Bound Steel Properties to Expected-Strength Steel Properties

Property	Year	Specification	Factor
Tensile strength	Prior to 1961		1.10
Yield strength	Prior to 1961		1.10
Tensile strength	1961–1990	ASTM A36/A36M-04	1.10
	1961–present	ASTM A572/A572M-04, Group 1	1.10
		ASTM A572/A572M-04, Group 2	1.10
		ASTM A572/A572M-04, Group 3	1.05
		ASTM A572/A572M-04, Group 4	1.05
		ASTM A572/A572M-04, Group 5	1.05
	1990–present	ASTM A36/A36M-04 and dual grade, Group 1	1.05
		ASTM A36/A36M-04 and dual grade, Group 2	1.05
		ASTM A36/A36M-04 and dual grade, Group 3	1.05
		ASTM A36/A36M-04 and dual grade, Group 4	1.05
	1998–present	ASTM A992/A992M-04	1.10
Yield strength	1961–1990	ASTM A36/A36M-04	1.10
	1961–present	ASTM A572/A572M-04, Group 1	1.10
		ASTM A572/A572M-04, Group 2	1.10
		ASTM A572/A572M-04, Group 3	1.05
		ASTM A572/A572M-04, Group 4	1.10
		ASTM A572/A572M-04, Group 5	1.05
	1990–present	ASTM A36/A36M-04, plates	1.10
		ASTM A36/A36M-04 and dual grade, Group 1	1.05
		ASTM A36/A36M-04 and dual grade, Group 2	1.10
		ASTM A36/A36M-04 and dual grade, Group 3	1.05
		ASTM A36/A36M-04 and dual grade, Group 4	1.05
	1998–present	ASTM A992/A992M-04	1.10
Tensile strength	All	Not listed[a]	1.10
Yield strength	All	Not listed[a]	1.10

Source: ASCE/SEI 41-06, Table 5-3.

[a] For materials not conforming to one of the listed specifications.

8.3.3.8 Development of Seismic Strengthening Strategies

If all of the components in the structure meet the basic acceptance criteria associated with their actions, no further analysis is necessary, and the building may be judged to meet the evaluation criteria. If not, typically a more refined study (including a pushover analysis) would be considered before deciding on a seismic rehabilitation program. The final evaluation should be based on a review of the qualitative and quantitative results. The evaluating engineer is urged to consider the issues carefully, to refrain from penalizing the building due to fine technical points beyond those contained in the ACSE 41-06 evaluation methodology. It is important to visualize the building in its ultimate condition in an earthquake, being aware of the risks of brittle failure and buckling. Due consideration should be given to the mitigating influences of good workmanship, structural integrity, and the strengths and redundancies that are not explicitly considered to be part of the lateral-force-resisting system. Most important, engineering judgment based on sound seismic design principles should be exercised before pronouncing

TABLE 8.8
Acceptance Criteria for Linear Procedures: Structural Steel Components

	m-Factors for Linear Procedures				
		Primary		Secondary	
Component/Action	**IO**	**LS**	**CP**	**LS**	**CP**
Beams—flexure					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \le \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \le \frac{640}{\sqrt{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				
Columns—flexure For $P/P_{CL} < 0.2$					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \le \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \le \frac{460}{\sqrt{F_{ye}}}$	1.25	1.25	2	2	3
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				
Columns—flexure For $0.2 \le P/P_{CL} \le 0.5$					
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{260}{\sqrt{F_{ye}}}$	1.25	9*x*	12*x*	15*x*	18*x*
b. $\frac{b_f}{2t_f} \le \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \le \frac{400}{\sqrt{F_{ye}}}$	1.25	1.25	1.5	2	2
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used				

Source: ASCE/SEI 41-06, Table 5.5.

Note: $x = \left(1 - \frac{5P}{3P_{CL}}\right)$

a building unsafe. Some of the questions engineers should ask themselves before declaring a building noncompliant are

1. What if the material properties are higher than assumed in the analysis?
2. What if we allow for a small amount of rocking and sliding at the base to absorb excess earthquake energy at little harm to structure?
3. What if we use composite properties for frame beams?

TABLE 8.9
Typical Deficiencies and Rehabilitation Measures

Deficiency in Existing Buildings	Typical Rehabilitation Measures
General structural attributes	• Complete discontinuous frames • Add additional elements to eliminate irregularities and add redundancy • Stiffen elements (e.g., add cover plates or concrete, encasement) • Add supplemental lateral-force-resisting elements to stiffen building • Add damping to reduce drift
1. Irregularities: vertical and horizontal	
2. Lack of redundancy	
3. Pounding potential	
Steel moment frames	• Increase capacity of elements (e.g., add cover plates or lateral bracing) • Add bracing to frames or additional frames to reduce loads on flexural/shear members • Increase capacity of elements (e.g., add cover plates or lateral bracing) • Strengthen connections and panel zones (e.g., add plates or stiffeners) • Increase capacity of columns (e.g., add cover plates or lateral bracing) • Add full penetration welds to column splices or splice plates • Strengthen connections and panel zones (e.g., add plates or stiffeners) • Stiffen elements (e.g., add cover plates or concrete) • Add supplemental lateral resisting elements to stiffen building • Add damping to reduce drift
4. Insufficient framing strength	
5. Noncompact sections	
6. Inadequate girder-column connections	
7. Strong beam–weak column	
8. Inadequate columns splices	
9. Nonductile joints	
10. Large drifts	
Braced frames	• Add lateral-force-resisting elements to reduce loads on existing members • Replace or add braces in proper configuration • Reduce unbraced lengths of bracing members • Strengthen connection (e.g., add plates, stiffeners, bolts, welds) • Add full penetration weld column splices or splice plates
11. Insufficient frame strength	
12. Improper brace configuration slender braces	
13. Insufficient connection strength	
Inadequate column splices	
Diaphragms and diaphragm connections	• Overlay existing slab with new reinforced topping slab with trim bars and collectors • Add drag struts or collectors at re-entrant corners and openings • Strengthen connections to vertical elements
14. Inadequate diaphragm strength/stiffness	
15. Inadequate collectors	
16. Lack of diaphragm tensile capacity at re-entrant corners	
17. Inadequate shear transfer to vertical elements	
Foundation and foundation connections	
18. Inadequate foundations	• Strengthen foundations • Strengthen base plate or column connections to foundations
19. Inadequate column-foundation connections	

4. For a moment frame building, what if we reanalyze the frame using different size rigid joints in the frame model? Does inclusion of an elastic spring to represent the stiffness of the joint result in a more favorable demand/capacity ratio?
5. What if we use slightly higher values for the *m*-factors?

SEI 41-06 has certain procedures to answer some of these questions. The author, however, recommends a parametric study of the acceptance criteria before declaring the building noncomplaint. This recommendation should not be constructed as sanctioning indiscriminate manipulation of the ASCE/SEI 41-06 procedure, but as a reminder to use that nonquantifiable, mysterious branch of engineering often called the art of design. It should be kept in mind that no matter how sophisticated an analysis is, it is hard to justify that its seismic behavior will be satisfactory if it has large vertical and horizontal discontinuities. Experience has taught us time and again that unfavorable seismic

characteristics mostly arise in a poorly balanced structural system. The seismic retrofit should, then, focus on removing irregularities and discontinuities.

In the evaluation and upgrading of an existing structure, it is sometimes difficult to identify an existing lateral-force-resisting system. Innovative analytical procedures and reliance on existing materials and systems that are not generally considered for new construction are required to determine the load paths and capacities of the existing structures. When an existing structure is not adequate to resist the prescribed lateral forces, strengthening of the existing lateral-force-resisting system will be required.

The selection of an appropriate strengthening technique for the upgrading of an existing building that does not comply with the acceptance criteria will depend upon the type of structural systems in the existing building and the nature of the deficiency. In some cases, the selection may be influenced by other than structural considerations. For example, a requirement that the building be kept operational during the structural modifications may dictate that the modification be restricted to the periphery of the building. On the other hand, it may be possible to temporarily relocate the occupants of a building that is to be upgraded. This, of course, provides more latitude in the selection of appropriate and cost-effective strengthening techniques. In many cases, seismic upgrading is accomplished concurrently with functional alterations, renovation, and/or energy retrofits. In these cases, the selected structural modification scheme should be the one that best suits the requirements of all the proposed alterations.

Determination of the seismic capacity of a structure includes consideration of all elements, structural and nonstructural, that contribute to the resistance of lateral forces.

Physical properties are generally obtained from available data; otherwise, assumptions and/or tests must be made. The analysis must include the evaluation of the most rigid elements resisting the initial lateral forces, as well as the more flexible elements that resist the lateral distortions after the rigid elements yield. Consideration must also be given to the interaction of various combinations of the structural framing systems and elements, which will contribute to the resistance of the lateral loads.

The results of the detailed structural analysis will identify the deficiencies with respect to the acceptance criteria of the various structural components and systems. These results should be carefully reviewed in the development of alternative upgrade concepts unless justification can be shown for a single solution. Each concept should be developed to the extent that will permit a reasonable cost estimate to be made. The extent of removal of existing construction should be considered, including the sizes and locations of new, replaced, or strengthened structural members. Typical structural connections with schematic details for upgrading nonstructural elements should be included in the study.

The following general considerations should be addressed in the development of the design concepts:

- Structural systems
- Configuration
- Horizontal diaphragms
- Eccentricity between the center of mass and the center of rigidity
- Deformation compatibility
- Foundations

8.3.3.8.1 Structural Systems

The development of the structural upgrading concepts requires a complete understanding of the existing vertical and lateral-load-resisting systems of the existing building. The designer must be able to determine the consequences that the removal, addition, or modification of any structural or nonstructural element will have on the performance of the strengthened building.

An evaluation of the existing vertical load-carrying structural system should be made to determine the effects that the seismic upgrading may have on the performance of the building to resist gravity loads. Vertical load resisting elements such as columns and framing systems may also be affected by seismic upgrading. If these framing elements are not used for the lateral-force-resisting system, they must be analyzed for deformation compatibility. This analysis should include the effects of the lateral displacements due to extreme seismic motion on their vertical load-carrying capacity.

8.3.3.8.2 Configuration

Severe problems may arise if the existing building is highly irregular in plan configuration or is composed of units with incompatible seismic response characteristic. An example is a flexible steel moment frame building connected to a relatively low rigid concrete shear wall building. If the resulting problem cannot be resolved by strengthening or upgrading the connection between two units, consideration should be given to separating them with a seismic joint. Each unit should have a complete system for resisting vertical as well as lateral loads. Structural members bridging the joint with sliding supports on the adjacent unit should be avoided. The criteria for new building separations also apply to existing buildings. Seismic joints should provide for the three-dimensional uncoupled response of each of the separate units of a building, but need not extend through the foundations.

8.3.3.8.3 Diaphragms

In most buildings, the horizontal framing systems (i.e., floors and roofs) will participate in the lateral-force-resisting system as diaphragms in addition to supporting the gravity loads. As part of the seismic upgrade, the floor and roof systems may require modifications (e.g., new topping or horizontal bracing), which will add to the dead load; thus, the capacity of the modified system must be evaluated for the new loading conditions. Every upgraded building should have either a rigid or a semi-rigid horizontal floor diaphragm.

8.3.3.8.4 Eccentricity

Provisions should be made for the increase in shear resulting from the horizontal torsional moment due to an eccentricity between the center of mass and the center of rigidity. In the development of upgrading concepts, when the vertical shear-resisting elements must be strengthened, supplemented, or replaced with new elements, consideration should be given to location of new or strengthened elements so as to reduce eccentricity between the center of rigidity and the center of mass.

8.3.3.8.5 Deformation Compatibility

The compatibility of the deformation characteristics of existing elements and the new strengthening elements should be considered. If the structure is to be strengthened to resist seismic forces, the new structural elements must be more rigid than the existing elements if they are to attract a major portion of the lateral forces and reduce the amount of force that is taken by the existing elements. Both the relative rigidities and strengths of all lateral-force-resisting elements must be considered.

Special consideration must be given in determination of relative rigidities of (1) concrete components: cracked versus uncracked; (2) shear walls: participation of intersecting walls (e.g., effective flange widths) and the effects of openings; and (3) steel frames: participation of concrete floor slab and framing, and infill walls. Structural elements that are not part of the lateral-force-resisting system should be evaluated for the effects of the deformation that occurs in the lateral-force-resisting system. Brittle elements are particularly susceptible to damage if they are forced to conform to the deformations of the lateral-force resisting system. In order to protect these elements from the possibility of being subjected to large distortions, provisions should be made to allow the structural system to distort without forcing distortion on the brittle elements. A good example is the isolation

of a masonry wall from the slab soffit. When rigid walls are locked in between columns, a similar method of isolation may be required at each end of the wall.

8.3.3.8.6 Foundations

If the seismic upgrade adds weight or redistributes the gravity loads, the foundations must be analyzed for the additional gravity loads combined with the horizontal and overturning forces associated with the seismic lateral force. Existing foundation ties that do not provide for adequate load transfer must be strengthened or replaced, unless proper justification can be provided for waiving the deficiency.

8.3.3.8.7 Base Isolation

Design strategies that significantly modify the dynamic response of a structure at or near the ground level are generically termed base isolation. This is usually achieved by introduction of additional flexibility at the base of the structure. The objective is to force the entire superstructure to respond to vibratory ground motion as a rigid body with a new fundamental mode based on the stiffness of the isolation devices. This strategy is particularly effective for short buildings (i.e., buildings with a fundamental period of less than about 1 s). For these buildings, it is feasible with isolation devices to develop a longer period of about 2–3 s. For most sites (e.g., those with a predominant site period less than 1 s), the longer period will occur beyond the portion of the response spectrum that is subject to dynamic amplification. Thus the response of the structures will be greatly reduced.

A typical base isolation installation consists of large pads of natural or synthetic rubber layers bonded to steel plates in a sandwich assembly or sliding bearings with either a flat or a single curvature spherical sliding surface made of polytetra fluordethylene (PTFE) or PTFE-based composites in contact with polished stainless steel. The isolator assembly, as well as all connecting elements and building services, must be capable of resisting the design spectral displacement corresponding to the new fundamental mode (some installations have base isolation assemblies that can deflect elastically up to 24 in.). Certain base isolation assemblies may have a lead core or other device to increase damping and thus decrease the response at the isolator. Because of the uncertainties associated with ground motion predictions, seismic base isolators are designed with fail-safe provisions to arrest the motion of the building to prevent instability due to excessive displacement. Base isolation can be an effective strategy to reduce the seismic response of a building, provided careful consideration is given to (1) the amplitude and frequency content of the expected ground motion, (2) the design of the pipes and conduits providing services to the building can accommodate the expected displacements, and (3) a fail-safe mechanisms is provided as described above.

However, in addition to the considerations just described, installation of base isolation in an existing building entails accurate determination of the magnitude and location of the vertical loads, design of a rigid diaphragm above the isolators to collect and distribute the lateral loads to the isolators, and careful underpinning and jacking of the existing structure in order to effect a systemic transfer of the existing loads to the base isolation device.

8.3.3.8.8 Passive Energy Dissipation

An effective means of providing substantial damping is through hysteretic energy dissipation. Some structures, for example, properly designed ductile steel and concrete frames, provide for additional damping as a result of the limited yielding of structural steel or concrete reinforcement.

In addition to the damping inherent in a ductile structure, passive energy-dissipating systems designed to increase structural damping have been in use for some time. This is an emerging technology that provides an alternate approach to conventional stiffening and strengthening schemes. The primary use of energy-dissipation devices is to reduce earthquake displacements in structure. These devices will also reduce the force in the structure, provided the structure is responding elastically, but would not be expected to reduce force in structures that are responding beyond yield.

8.3.3.8.9 Conclusions

Before concluding this section, perhaps it is beneficial to reflect on some of the performance scenarios presented in ASCE/SEI 41-06, particularly those buildings deemed to be top-of-the-line performers. It is the opinion of many engineers including the author that high-end building performance cannot be promised or achieved with 100% certainty. Top-of the-line-performance implies a near-perfect earthquake-proof building. Therefore, building owners and the public are likely to ask for it more frequently. Ask they should, but with the understanding that there is no such thing as earthquake-proof buildings, only earthquake-resistant buildings. It is therefore the structural engineers' responsibility to make this fact clear to the owners and to the public so that their expectations for building performance do not exceed those implied in the ASCE/SEI 41-06 performance definition. Although major advances have been made in analytical capability and in the synthesizing of experimental and earthquake performance data, prediction of building performance, relative to future earthquakes is still a risky and dangerous business. Thus, seismic rehabilitation continues to challenge the very core of conventional thinking.

Generally, seismic retrofit should be considered only if its entire cost is less than 70%–80% of replacement cost. It should be noted that it is impossible to bring an existing structure into conformance with current code requirements. Therefore, the cost assigned to the retrofit should not outweigh the seismic performance expected of the building.

It behooves the designer to consider more than one seismic retrofit strategy. One of these should be a conventional one. This ensures that designers are not carried away with a high-tech new approach when a more conventional retrofit strategy is more cost-effective.

Seismic retrofit design is invariably more expensive than new construction design. The extra design effort required for retrofit design should be communicated to the owner at the onset of the project. The cost of the retrofit design should be pegged to the complexity of the analysis required. Many designers do not assign sufficient design hours to projects that require NSP or NDP. The cost of developing and implementing material test recommendations should be considered at the start of the project. It is recommended that material test results be available to the designer before the design development phase is started. The design basis should already be stated and discussed with the owner and peer reviewers at the onset of the project.

It is emphasized that a number of parameters used in ASCE/SEI 41-06 remain a matter of discussion and research. It must remain at the discretion of engineers to modify the parameters as they deem appropriate. Agreement on key issues must be reached with peer reviewers as the analysis progresses.

8.3.4 ASCE/SEI 41-06: Design Example

Steel building with pre-Northridge moment frames and concentric bracing
Given

An existing 5-story office building located in downtown Los Angeles, California. The lateral system consists of a combination of moment frames and concentric braces. Built in 1990 the buildings moment connections suffered damage during the 1994 Northridge earthquake. The damaged connections were repaired by using notch-tough welds for the beam bottom flanges. No seismic evaluations were made at the time of moment connection repairs.

In 2011, the building is being acquired by new owners who desire an assessment of the building's expected seismic performance before making the final deal. You have been hired as the structural engineer of record to evaluate the seismic vulnerability of the building, and, if required, to come up with a preliminary seismic upgrade scheme. The selected design basis is that the building should be operational after a major earthquake, that is, the performance criterion is Immidate Occupancy, IO.

Let us assume that you, as the engineer of record has selected the following components for a preliminary seismic evaluation:

1. Frame beams
2. Frame columns
3. Beam–column connection, including column panel zones
4. Braces in concentric braced frames
5. Diaphragm components
6. Frame column-to-foundation connections

In the real world of seismic study, one would be required to evaluate seismic performance of all major structural components, including, quite possibly, certain nonstructural elements. A comprehensive presentation of such a study is, however, deemed beyond the scope of this work. The purpose here, as in the entire book, is to present the methodology, not detailed calculations.

That said, we select just one frame beam to asses seismic vulnerability. For this purpose, we assume the following:

1. The selected frame beam is a W30 × 116, $F_y = 50$ ksi.
2. Q_E is the action due to unreduced earthquake loads (i.e., $R = 1$), determined using a linear dynamic procedure (LDP). In our case, the beam action is bending. Assume, $M_E = Q_E = 3280$ k-ft for BSE-2 earthquake.
3. Q_D is the dead-load action (i.e., dead load moment). Assume $Q_D = M_D = 580$ k-ft.
4. Q_L is the effective live load action (equal of 25% of unreduced design live load, but not less than actual live load). Assume $Q_L = M_L = 145$ k-ft.
5. Q_S is the effective snow load. Assume $Q_S = 0$.

Solution

Step 1 is the determination of the characteristics of future ground motions likely to be experienced at the building site. Because ground motions are the most significant cause of earthquake damage, rehabilitation objectives are established using earthquake ground shaking hazards, usually defined on a probabilistic basis. Observe that the predicted seismic performance characteristics are functions of the severity of specified earthquakes and are directly related to the extent of estimated damage sustained by the building due to postulated earthquakes.

The owners intent is to have the building operational during and after the seismic events specified in ASCE 41-06 for the given rehabilitation objective. The engineer of record, hopefully with a golden tongue has, in nontechnical terms, described to the owners the broad range of expected building performance characteristics in terms of possible damage to both structural and nonstructural building components. Communication with the owner in lay terms is perhaps the most important step in a seismic rehabilitation study. The owners of our subject building are now well-informed about probable post-earthquake scenarios. Because the selected objective is Operational Level, IO, they expect

- Overall damage to the building to be light
- Structure to have no permanent lateral displacement
- Structure's original strength and stiffness to remain substantially unchanged
- Minor cracking in façade partitions and ceilings
- Minor local yielding of structural elements at a few places, without fracture
- Elevator and fire protection system remaining operable
- In terms of nonstructural components, equipment and contents to be generally secure, but perhaps not operable due to mechanical failure or lack of utilities

As stated previously, prediction of building performance in a future earthquake is a dangerous and risky business. Consider, for example, the top-of-the-line performance as set forth in ASCE 41-06. It implies that a building designed for this performance objective is likely to come out scratch—free after a high seismic event, giving the impression that the building is invincible. This may not be so even when the building is designed or retrofitted to IO level. This important difference should be brought to the attention of building owners before they embark on a seismic upgrade.

Because the stated objective of the seismic evaluation is Immediate Occupancy (IO), we select from Table 8.1 the following characteristics earthquakes represented by *k*, *p*, and *e* for the analysis of the building.

- BSE-1 earthquake with a 10% probability in 50 years (mean return period of 474 years, rounded to 500 years, used in most building codes)
- BSE-2 earthquake with a 2% probability in 50 years (mean recurrence interval of 2475 years, rounded to 2500 years)
- An earthquake with a 20% probability in 50 years (mean recurrence interval of 225 years)

For the example building, we assume that the project geotechnical engineer has, after conducting site-specific studies, developed acceleration response spectra for the earthquakes listed in the foregoing. Since the methodology is the same, we will for purposes of illustration, verify the Demand/Capacity Ratio, D/C, for the frame beam for only the BSE-2 earthquake.

Step 2 is the determination of as-built conditions in order to arrive at a value for the reliability coefficient κ. The building is fairly new, built in 1990. We will assume the as-built information pertinent to its seismic performance, including construction documents and material test reports, is available, and a visual survey has indicated that there are no site-related concerns such as pounding from neighboring structures. Because of the abundance of as-built information, the engineer is able to gain a comprehensive knowledge and understanding of the behavior of structural components allowing a value of $\kappa = 1.0$ in the analyses. κ is a reliability coefficient used to reduce the component strength value for existing components. Because $\kappa = 1.0$, there is no need to reduce the computed strength values of the component when making demand capacity comparisons.

Step 3 is the classification into primary and secondary components. It is a straightforward task for the example building. Observe that both frame beams and columns are primary components because they are essential for providing the structure's basic lateral resistance. This step also includes the classification of the response of lateral-resisting components into either deformation-controlled or force-controlled actions. In our case, it is evident that the flexural action of the frame beam is deformation-controlled. However, the classification of frame-columns subject to combined compression and bending is not so obvious. It could be either deformation-controlled or force-controlled, depending on the ratio of axial load in the column and its axial strength.

Step 4 entails setting up the analytical model; calculating the building period; and determining the base shear, its vertical distribution up the building height, and the forces in the floor and roof diaphragms. This task, an everyday occurrence in a design office, does not here require further explanation except to point out that

1. If an LSP is used for seismic analysis, then the base shear *V*, also referred to as the pseudo-lateral load is calculated using the unreduced spectral acceleration S_a without an upper limit on the building period, *T*.
2. If an LDP such as modal superposition is used, then the analysis is carried out using a response spectrum that is not reduced to account for the anticipated inelastic response of the building (i.e., $R = 1.0$).

The purpose of an LSP or LDP is to determine the distribution of forces and deformations induced in a structure by the design ground motion. Although an LSP is permitted for simple buildings

less than 100 ft in height, prevailing practice is most design offices is to use an LDP and the modal superposition method. Hence, we will assume that the forces and moments in the frame beam have been evaluated by performing linear dynamic analyses for each of the three response spectra selected for the study.

Step 5 is where the seismic and gravity loads are combined to determine Q_{UD}, the demand imposed on elements due to seismic and gravity loads. Because the actions of both frame beams and frame-columns of the example building are considered deformation-controlled, we use the following equation to calculate demand:

$$Q_{UD} = Q_G + Q_E$$

If, on the other hand, the action of an element under consideration is force-controlled, the corresponding equation for verifying the acceptance criteria would have been

$$Q_{UF} = Q_G + Q_E / C_1 C_2 C_3 J$$

where

Q_{UF} is the design action due to combinations of gravity and seismic loads
J is the *coefficient* that estimates the maximum earthquake force that a component can sustain and deliver to other components

The parameter J is in the denominator of the second term of the equation given above. Therefore, it is possible to recognize that in a nonlinear response, the actual force sustained by the components is likely to be less than earthquake force Q_E, determined by elastic analysis. The maximum value of J is 2. It is related to the seismicity of the site as follows:

$J = 1$ in zones of low seismicity
$J = 1.5$ in zones of moderate seismicity
$J = 2$ in zones of high seismicity
C_1, C_2, and C_3 are the modification coefficients explained earlier in Section 8.3.3.5

Step 6 is where the component capacities Q_{CE} or Q_{CL} are calculated, depending upon whether the action considered is deformation—or force-controlled.

Returning to the example frame beam, since bending is categorized as a deformation-controlled action, the demand Q_{UD} due to gravity and seismic is given by

$$Q_{UD} = Q_{UD} + Q_G + Q_E$$

$$Q_G = 1.1\,(Q_D + Q_L + Q_s)$$

$$= 1.1\,(580 + 145 + 0) = 797.5 \text{ k-ft (say, } 800 \text{ k-ft)}$$

$$Q_E = 3820 \text{ k-ft}$$

$$Q_{UD} = 3820 + 800 = 4620 \text{ k-ft}$$

Step 7 is the final step in which the acceptance criterion is verified for each component. Although we earmarked six distinct components for seismic assessment, as stated previously, we will check the acceptance criterion for the frame beam only.

Recall in step 3, it was determined that the limit state for the example beam was flexure. Observe that beam flexure (and for that matter, beam shear) with negligible axial loads is almost always considered a deformation-controlled action.

The design properties for the example frame beam are as follows:

W30 × 116F_y = 50 ksi, group 3, manufactured after the year 1990.
Default lower-bound yield strength = 52 ksi (Table 8.6)
F_{ye} = expected steel strength = 1.05 × 52 = 54.6 ksi (Table 8.7)

Expected capacity $Q_{CE} = ZF_{ye}$
= 378 × 54.6 (note z for w30 × 116 = 378 in.^{3})
= 20,639 k-in.
= 1720 k-ft

A_s = 34.2 in.^{2}, d = 30.01 in., t_f = 0.565 in., b_f = 10.495 in., t_f = 0.85 in.

$$b_f/2t_f = 10.495/2 \times 0.35 = 6.17$$

$$52/\sqrt{F_{ye}} = 52/\sqrt{53.55} = 7.10 > 6.17\ b_f/2t_f$$

$$h/t_w = 28.31/0.565 = 50.1,\ 418/\sqrt{F_{ye}} = 418/\sqrt{53.55} = 57.12 > 50.1$$

Therefore, the m-factors from Table 8.8 are

IO	LS	CP
2	6	8

The knowledge factor κ = 1.0 because the quality and extent of available information, as stated at the beginning of the design example, is comprehensive. Since we are verifying the component's acceptability for IO performance, m = 2, and

$$mkQ_{CE} = 2 \times 1 \times 1720 = 3440 \text{ k-ft}$$

The demand Q_{UD} from step 6 is 4620 k-ft, which is grater than the expected capacity of 3440 k-ft, indicating noncompliance. A similar evaluation is made for other actions of the beam. The analysis is repeated for the other two earthquakes. The results are reviewed, keeping in mind that values of m are only an approximate indicator of seismic performance. Reevaluation of the building using nonlinear analysis procedures with reevaluated gravity and lateral loads using a sharper pencil is a prudent course of action before deciding on a seismic rehabilitation.

The procedure for evaluating acceptance criteria is conceptually the same for the other components of the moment frame, listed earlier, such as columns, panel zones, beam–column connections and column-to-foundation connections. Hence, they are not discussed here.

Suppose the objective of the seismic evaluation of our example building is basic Life Safety, LS, instead of IO. Does the method of seismic study differ from the preceding procedure for IO performance? What if the target performance is CP instead of IO or LS?

It is reassuring to know that the procedure is generally the same, irrespective of the selected target performance. The major differences are in ground motion selected for the analysis and in the values of the m-factors used in the acceptance criteria.

Consider, for example, the basic LS objective as the target performance. A building that satisfies this objective will pose approximately the same earthquake risk traditionally considered acceptable for new buildings. The LS study includes verification of the building's behavior for two distinctly different earthquakes:

1. An earthquake with a 10% probability of occurrence in 50 years to satisfy LS (BSE-1)
2. An earthquake with a 2% probability of occurrence in 50 years to satisfy CP (BSE-2)

It is of interest to note here, the difference between CP and LS seismic objectives.

CP refers to the post-earthquake damage state of the building that may be on the verge of partial or total collapse but has not yet collapsed. Substantial damage to the structure has occurred, with considerable loss of stiffness and strength in the lateral-force-resisting system. However, all significant components of the gravity-load-resisting system continue to function.

If the building does not collapse, some engineers may wrongly consider that the LS objective has been met. It should be noted that the LS performance level includes a margin of safety against collapse for a lower level seismic event. Significant risk of injury from falling structural debris may exist. It may not be practical to repair the structure and it may not be safe for reoccupancy, because aftershock activity may induce collapse.

To satisfy the limited safety objective of CP, the building may be evaluated for a single earthquake chosen from a range of specified earthquake hazard levels. Building Safety Earthquake BSE-2, with a 2% probability of occurrence in 50 years, is one example. See ACSE 41-06, Table C1-1 for other specified earthquakes.

8.3.5 Summary

ASCE/SEI 41-06 permits both linear and nonlinear analysis procedures for evaluation of existing and rehabilitated construction. Whether or not ASCE 41-06 is applicable to design of new construction is a matter of discussion between the Engineer of Record, Peer Reviewer, and the Building Official. Observe that PBD does strongly promote such a procedure for the design of new buildings.

Rehabilitation strategies described in ASCE 41-06 include:

1. Global modifications such as
   a. Increasing stiffness and strength by adding new elements
   b. Increasing damping using supplemental damping devices
   c. Isolating the structure from seismic ground motions by using seismic isolation
   d. Decreasing mass
2. Local modification of components examples include
   a. Local strengthening by adding new plates or angles to existing steel members
   b. Jacketing or compositing of existing steel members

For a list of typical deficiencies and rehabilitation measures the reader is referred to Table 8.9.

The Linear Static Procedure, LSP is similar to the equivalent lateral procedure included in most seismic standards. However, the pseudo-lateral loads $V = C_1C_2C_mS_aW$ incorporates techniques for considering the nonlinear response of individual elements and components, and is based on the unreduced spectral acceleration S_a. In a manner of speaking, we are using a value of unity for the Seismic Response Modification, R.

A Linear Dynamic Procedure, LDP may be used for modal spectral analysis or linear time history analysis. In both cases, the results are modified with coefficients similar to those in the LSP. The acceptance of performance is judged for each component by defining component action as either deformation-controlled or force-controlled.

A Nonlinear Static Procedure, NSP, commonly referred to as a pushover analysis, is performed to develop the relationship between lateral forces and displacement at the roof. Static lateral loads are applied incrementally and the element properties are adjusted for yielding or failure. The seismic global displacement demand is determined and deformation-controlled components are judged acceptable if their gravity plus earthquake deformation demand is less than or equal to the permissible deformation capacity given in ASCE/SEI 41-06.

9 Special Topics

PREVIEW

Structural design is in the period of mixing and perfecting structural systems such as mega frames, interior or exterior super diagonally braced frames, spine wall structures, etc., to name a few. Using this vocabulary and numerous case studies, we introduce the reader to those aspects of conceptualization and engineering judgment that are timeless constants of the design process. These traits can be as important and valuable for understanding structural design as are the latest computer software. The case histories presented in this chapter are based on information contained in various technical publications and periodicals. Frequent use is made of personal information gathered from the structural engineers of record.

A discussion of human tolerance to wind-induced dynamic motions of tall slender buildings is the subject of the next section. Also explained are the U.S. and International design guidelines for limiting building motion perception and the effect of structural damping in determining the serviceability limit state. Next, we consider the fundamental aspects of performance-based design, PBD, which has become the cutting edge technology in building design. This is because owners like them for they cost less, architects love them because they offer more design freedom, and engineers being thrifty go for it because PBD can result in high-quality structures with the least amount of materials.

Next, we focus on analyses techniques that include portal and cantilever methods for preliminary sizing of 2D frames subject to lateral loads. Included also are preliminary procedures for analyzing frame tube structures and Vierendeel trusses.

Calculation of wind and seismic loads using approximate procedures is the subject of next section. Design response spectra for 50 selected locations in the continent of United States are presented in a concise format to assist engineers in the preliminary evaluation of seismic loads. The chapter concludes with a discussion of differential shortening of columns and determination of unit quantity of structural steel as it relates to the height of a building.

9.1 ARCHITECTURAL REVIEW OF TALL BUILDINGS

Building architecture in the twentieth century can be traced to several nineteenth-century roots. One was the advent of new forms of structural and other materials so strikingly displayed in the building technology of skyscrapers. This has allowed greater scope of aesthetic expression and innovation in architectural practice. Another was the development of expressive new idioms of form and space. The development of metal trusses made it possible to roof column-free interior spaces easily and economically. Such roofs were used for railroad stations, market halls, exhibition palaces, and domes.

The nineteenth century was one of the most technically inventive centuries. It witnessed the application of new techniques and of new mechanical means in virtually every human activity. It became clear in time that the innovation in architecture would come from those who grasped the possibilities of the new materials and techniques. Revolutionary methods of building with wood were developed in the 1830s to meet the demands for speedy construction and to overcome the shortage of skilled labor. Cast iron was developed into a building material lighter and more

adaptable than masonry, and combined with other inventions, notably the elevator, paved the way for tall buildings unprecedented not only for height but also for ease of construction.

In Chicago, during the later part of the nineteenth century, a school of architects of whom Louis Sullivan and Frank Lloyd Wright were the most famous members originated a new American style of domestic architectures. Their designs, primarily houses, were long, informal, and organic. Their ideas were ignored for more than a decade in America, but were taken up aboard and developed into the so-called International Style. The International Style was also influenced by the German Bauhaus school, founded by Walter Adolf Gropius and by the abstract artists interested in using pure forms for buildings. This style was the architectural response to the machine age. Simplicity meant elegance derived from "pure" forms. Display of the structural muscle beyond the tightly stretched curtain wall was widely accepted as the "in thing." Structures designed in this era incorporated three distinct elements of the new style: (1) a new vocabulary of forms borrowed largely from abstract art, consisting of planes, lines, and rectangles without ornaments or moldings; (2) the representation of interior space and exterior façade as a cohesive unit; and (3) the use of new structural materials such as steel.

The new style, with its angular forms, plane surfaces, and lack of conventional ornament, met with some resistance from the public, which tended to regard it as bare and inhuman. But by the middle of the twentieth century the style had become dominant across the country. Bold use of modern construction methods and structural materials became common. Noteworthy among the latter are glass tinted to reduce glare; glass brick designed to admit additional light while preventing glare and furnishing effective insulation against heat, cold, and noise; artificial stone; plastics; chromium, aluminum, and other metals; and above all steel.

The early stages of American architecture lacked truly monumental structures. The monumental idea was gradually added to American architectural forms, reaching its apex with the construction of Rockefeller Center in New York City. The center represented a new concept of building a city within a city, containing a towering 60-story structure surrounded by a number of smaller high-rise office buildings and recreational facilities. This complex of skyscrapers has exercised increased influence since 1931, the year work on the center was started. The building represents a departure in architectural thinking from a single-use, single-building concept to multi-use, multi-complex structures on a community scale. Because of that practical example, architects have responded more and more creatively to such demands of contemporary life style as rapid intercommunication and integration of city and surrounding region. Another example of multi-building planning is the now nonexistent World Trade Center in New York City, consisting of two 110-story towers and four smaller buildings grouped around the plaza.

During the period of 1950 to the mid-1960s, the International Style of architecture was embraced by prominent American architects and resulted in sleek boxlike glass and steel high rises, which integrated the concept of purity of design into the architecture of the structure. Notable examples are the Seagram Building (1950) and the Whitney Museum (1966), both in New York City, and the John Hancock Center (1968) in Chicago.

During the mid-1960s, a reaction developed to the International Style that emphasized greater freedom of design. Figuratively speaking, the concept of glass box was beginning to shatter. It was no longer a misdeed to hide a structure behind a more aesthetic exterior. The building and construction industry saw the advent of new forms of structural and other materials, which allowed greater scope for aesthetic expression and innovation. Within the last decade many cities, outside of the United States have had imaginative new forms thrusting above their skylines using plan configurations that are other than prismatic. Large corporations have built a new generation of flamboyant headquarters building that are altering the urban skyline and bringing new vigor to cities around the world. Many are giant architectural logos that draw enormous public attention with expected increases in revenues to the companies that build them. These new buildings are emerging as good investments, serving as advertising symbols and marketing tools.

The distinguishing architectural features for this new generation of buildings are sculptural shapes at the tops and elaborate articulation at the bases.

9.2 EVOLUTION OF HIGH-RISE ARCHITECTURE

The high-rise architecture in the twentieth century shows such a wide diversification as to defy distinct classification. Nevertheless, its development can be traced albeit imprecisely, in five phases. In the early 1940s, before the advent of air conditioning and fluorescent lights, the building form was controlled by the need for natural daylight and ventilation and thus required a layout somewhat similar to those of contemporary apartments and hotels. The building width was limited to ensure that light and air reached all parts of the building, resulting in a width of 55–60 ft (16.7–18.3 m) that offers office space on either side of a double-loaded corridor. To achieve more leasable space on a given site, plan configurations with a central core and radiating wings were conceived.

The second phase of building configuration is the result of the interactions between the desire to create an increasing range of rentable area in a given space and the advent of air conditioning and fluorescent lighting. This period is also characteristic of the modern movement in architecture stressing the aesthetic value of simplicity in façade treatment and simple cubic shapes, such as rectangles, squares, and circles. At the exterior, the curtain wall was stretched tightly over the skin and the building shot up skywards in one regular prismatic shape. In keeping with the International Style, it was not offensive—in fact, it was highly desirable to display the structural muscle. Glass boxes with exposed structural members constituted the backbone of the International Style.

The third phase of the high-rise architectural development is a result of interaction between marketing experts and a mild boredom of the public toward the repetitive nature of the boxes on the cityscapes. The simplest prismatic shape has only four corners and, therefore, could offer at best only four corner offices. Even those corner offices more than likely displayed corner columns. Now, however, corner offices have become one of the most sought after lease space. To capture this market, the trend in planning is to have as many corner offices as possible. This is achieved by undulating the exterior, providing nicks, notches, and other convolutions at the perimeter. Additionally, to create visual identity and interest, setbacks are provided at selected levels. An otherwise simple shape is sliced and diced to create vertical lines to emphasize the verticality of the building while simultaneously providing for additional corner offices.

A fourth phase of office design known as postmodern architecture is bringing in daringly articulated buildings. These buildings not only have step backs, notches, and curves, but the resulting articulations are so severe as to preclude use of any single structural system for the entire building. This phase of design, which began around 1970, is considered primarily an aesthetic reaction to the cubism period. It has evolved gradually in three stages. First, the flat roof, which is all that is necessary from the functional point of view, started receiving architectural attention to gain identity in the city skyline. Today, many buildings' tops take on any number of articulations. The second stage is characterized by the elaborate entrances to the building in an effort to give it a street-level identity and allure. The third stage is a continuation in the battle for identity. Articulations at the extremities are no longer sufficient to create a building's identity. The entire architectural façade needs to proclaim the identity of the building. To this end, terracing of building plans, cutouts, slicing and dicing, and overhanging features are added to the building throughout its height.

The fifth phase can be looked upon as a modification to building shapes in the energy conservation context. We are witnessing buildings suitable for natural daytime lighting with courtyards, light wells, and skylights. Energy conservation efforts have brought about an understanding of spaces as a whole, especially in relation to how light influences the space. Instead of depending totally on mechanical heating and cooling and electric light, architects are considering the possible solar controls outside and in as an integral part of architectural design.

It is however reassuring to know that the present day flamboyance in high-rise architecture has not intimated structural engineers from offering support systems that are at once elegant and economical. In fact, it has stimulated the structural engineering profession to give almost total freedom to architectures in their quest of new and exciting avatars for tall and ultra tall buildings. Today, with the use of computers, buildings are planned and designed that have little or no historic precedent. New structural systems are conceived and applied to extremely tall buildings in a practical demonstration of the engineer's confidence in the predictive ability and reliability of computer solutions.

9.3 TALL BUILDINGS

Nine years after many soothsayers predicted the indelible images of September 11, 2001, would stifle humanities enthusiasm for iconic skyscrapers it is now certain they are wrong. Contrary to their prophecy, the landscape not only in the United States but also in the entire world is moving up. Buildings that would either eclipse or stand spire-to-spire with the 1.250 ft tall Empire State Building of the 1930s, are promising to reshape skylines. The collective introspective analysis of the topology that begged the question "are tall buildings a viable part of our cities—or they are not?" seems to have resulted in a resounding "yes."

Compared to the skyscrapers that were erected in the 1970s and 1980s, the new crop of high rises is more likely to be expressions of civil or even national pride than symbols of corporate wealth. Their purpose is to project a certain status for a city on a world stage. This has become especially true as developing countries in Asia and the Middle East started erecting super-tall skyscrapers that made the iconic American Towers, the Empire State Building, the Sears Tower (now called Willis Tower), and the nonexistent World Trade Center Towers, look average in comparison.

In today's high-rise architectural vocabulary, remarkably, the repetition of structural and architectural elements is losing importance. Many nonorthogonal skyscrapers are designed to be iconic, to proclaim their presence in cities, regions, and even countries on the global map. Their facades are curved, or organically shaped for aesthetic reasons, and often the entire building sports a twisted elevation. Although complexity of fabricating and constructing superstructures of nonprismatic shapes increases, various major projects constructed within the past decade demonstrate the feasibility of large-scale application of nonstandard products. High-rise shaping is strongly influenced by the CAD modeling tools that architects have available. The conventional rectilinear box form of skyscrapers is gone for good, and unlikely to make a comeback any time soon. Most developers and architects are very focused in creating exciting shapes and forms that would have once made these projects unbuildable. Today, these geometric complexities are better understood and can be dealt with efficiently. Probably the most significant factor in the cost of a tall building is the choice of the structural system chosen for resisting lateral loads. Tall buildings are flexible structures. Therefore, their dynamic response to wind excitation is critical in assessing their loading and performance particularly with respect to buildings' deflections and accelerations at the top occupied floors. In this context, it is interesting to note that the across-wind response, rather than the along-wind response dominates its behavior. Although analytical methods exist for predicting wind response, the only method for predicting their dynamic behavior with any degree of certainty is through a wind tunnel study.

There are a number of methods designers can use to minimize building response to wind loads.

- Use openings at top floors integrated with architectural features of the building to break up eddies that may cause vortex-shedding.
- Orient the building so that its weak direction does not line up with the strongest wind direction.
- Use rounded plan shapes instead of sharp-edged plans.
- Use step-backs along the building's height to break up flow of wind, which may cause vortex-shedding.

In spite of the methods mentioned earlier known for some time to reduce adverse effects of winds, the current trend in tall building design has embraced shapes and forms that would have once made these projects unbuildable. Today these complexities are better understood by using computer generated geometrics. Only a few years ago, many of the high-rise shaping would have seemed mere figments of architect's imagination; now, many have been built and new ones planned. It seems CAD-based modeling tools and structural calculations have changed the shaping of high rises forever.

Innovative solution in a broad sense is one that achieves desired function at the lower cost, without sacrificing quality. Our problem with innovation is that it has been an elusive quality and one rarely captured in our educational process. Our education is mostly devoted to developing analytical skills, mainly through the use of computers and very little to developing their opposite, inductive skills. Advancement in commercial computer software has contributed heavily to the development of sophisticated methods of analysis and to optimization methods for well-defined structural problems. However, the conceptual selection of an engineering system is by and large still an elusive skill with no established learning procedure. The ability to conceive and generate design alternatives requires both a feeling for behavior and approximate design skills, best acquired by carefully examining structural behavior of buildings of the past through discussions and techniques such as "load path" concept.

The purpose of presenting case studies of certain selected tall buildings is to highlight the aspects of conceptualization that have been timeless constants in the design of tall and super tall buildings. It is as important to understand how engineers in the past have conceptualized the building skeleton using such ideas as mega frames, interior and exterior braced tubes, spine-wall structures, to name a few, as is the skill of navigating through the latest computer software. Many of the examples given here are once-in-a-life time, high profile, dream projects that you would give your right hand to design. Hop on our tour bus: Let us experience at once the architectural grace and the elegance of structures supporting tall and ultra tall buildings on our planet.

9.3.1 World Trade Center Towers, New York

We start our tour of learning the mystery and romance of tall buildings by first paying homage to World Trade Center Towers, New York, built in 1973 and destroyed on September 11, 2001 when two hijacked commercial airlines were flown, one into each tower. Although the towers have disappeared from Manhattan skyline, they continue to command respect from public in general and structural engineers in particular.

Of the seven buildings of the World Trade Center (WTC) complex of New York City, the WTC towers, known as WTC 1 and WTC 2, were the most visible and recognized tall buildings throughout the world (Figure 9.1a). Each of the towers encompassed 110 stories above plaza level and seven levels below. WTC 1, the north tower, had a roof height of 1368 ft, while WTC 2 stood nearly as tall at 1362 ft. Each building had a square floor plate 207 ft 2 in. (63.14 m) long on each side, with chamfered corners measuring 6 ft 11 in. (2.10 m). A rectangular service core of approximately 137 by 87 ft (41.75 × 26.52 m) was present at the center of each building.

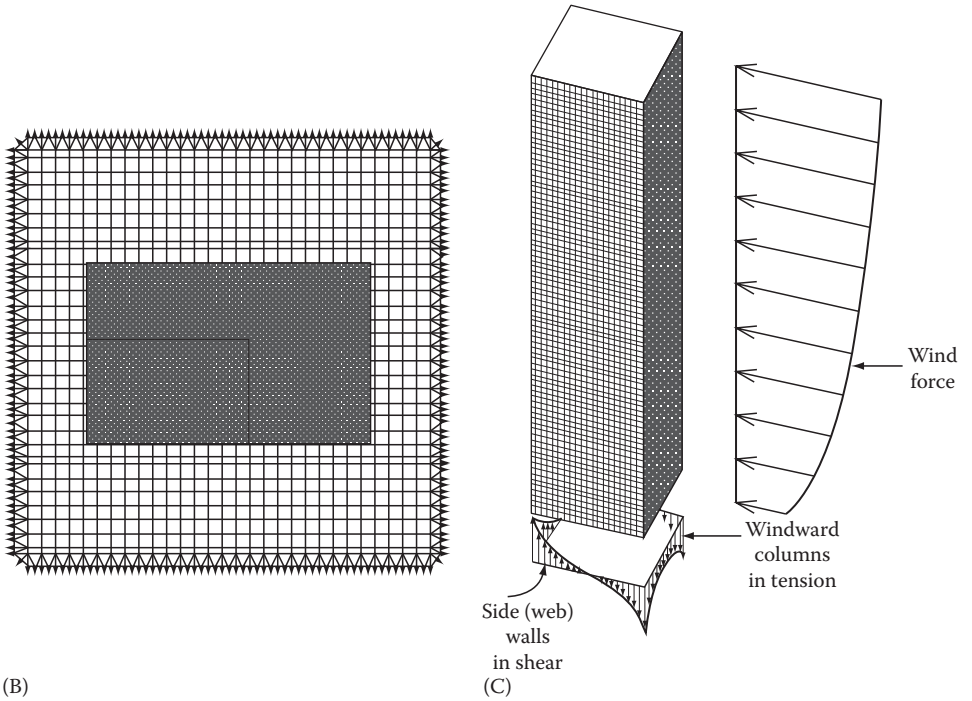
The buildings signature architectural design feature was the vertical fenestration that featured a series of closely spaced built-up box columns, as shown in Figure 9.1b. At typical floors, a total of 59 of these columns were present on each of the flat faces of the building, placed at 3 ft 4 in. (1.0 m) on centers. Adjacent perimeter columns were connected at each floor level typically by 52 in. (91.32 m) deep spandrel plates (see Figure 9.1d and e). In alternate stories, an additional column was present at the center of each of the chamfered building corners. The resulting configuration of closely spaced columns interconnected with deep spandrel plates created a perforated perimeter tube (see Figure 9.1e).

Twelve grades of steel, having yield strengths varying from 42 to 100 ksi (191 to 455 kN), were used to fabricate the perimeter columns and spandrel plates. In the upper stories of the buildings,



(A)

FIGURE 9.1 World Trade Center (WTC) Towers, New York: (A) Photographs;



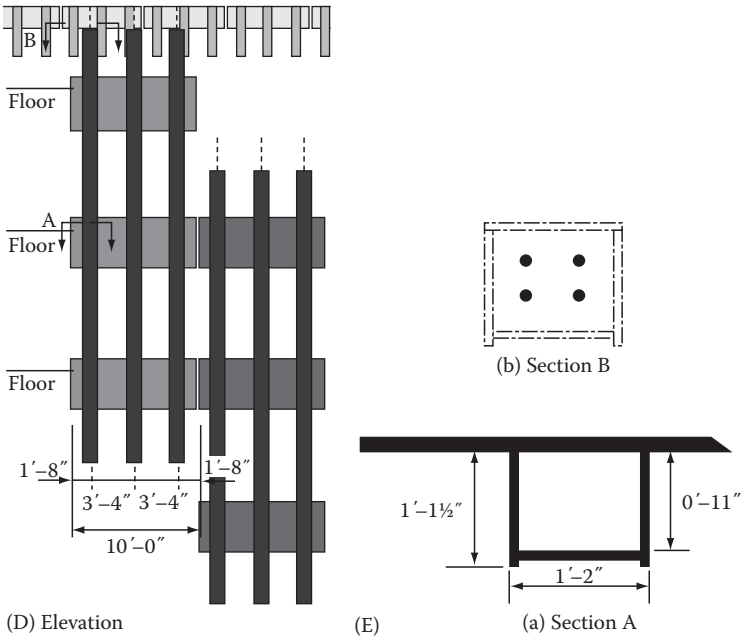
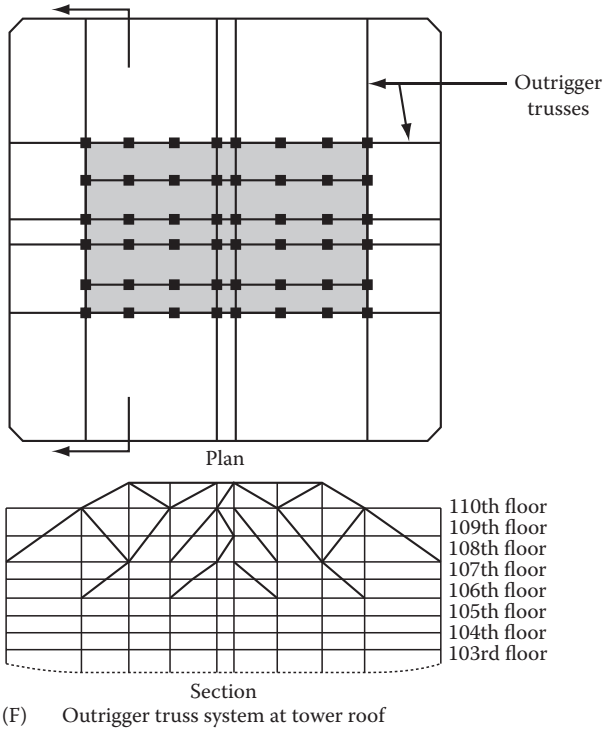
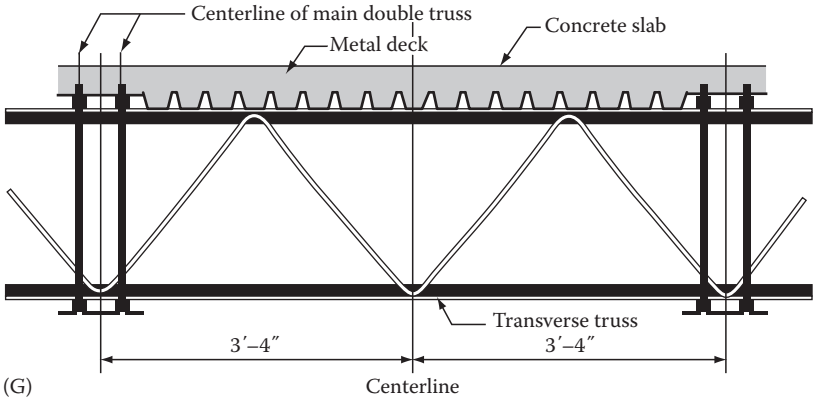


FIGURE 9.1 (continued) (B) framing plan; (C) column axial load due to wind; (D) prefabricated column and spandrel assembly; (E) (a) Section A through spandrel; (b) Section B through perimeter column;

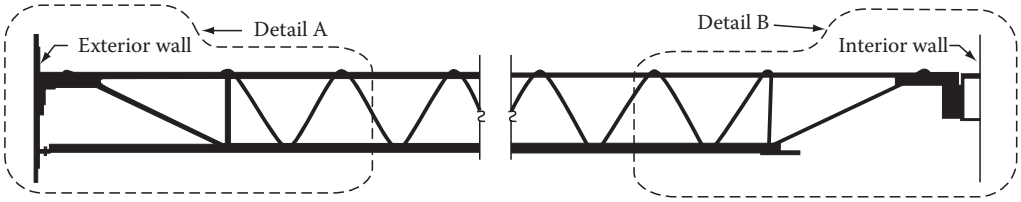
(*continued*)



(F) Outrigger truss system at tower roof



(G)



(H) Typical floor truss

FIGURE 9.1 (continued) (F) outrigger truss at tower roof, plan and section; (G) floor framing system; (H) typical floor truss;

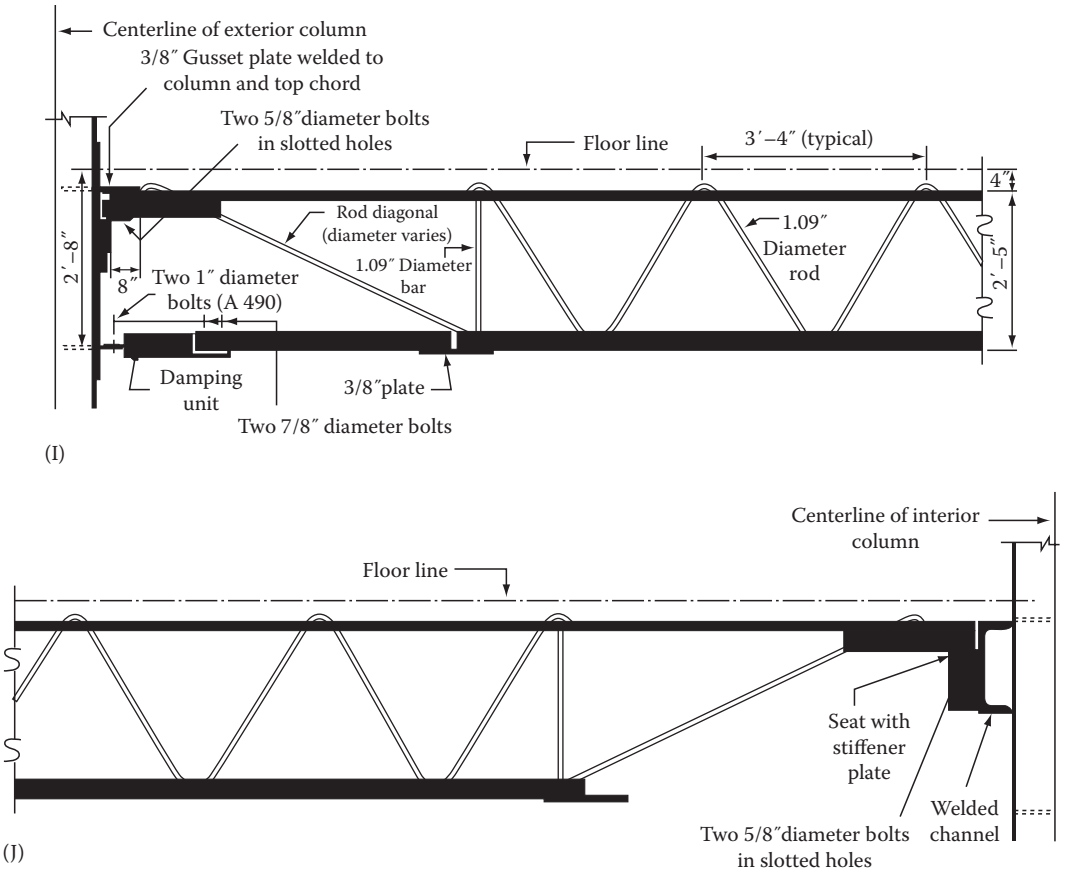


FIGURE 9.1 (continued) (I) detail A, exterior wall end detail; (J) detail B, interior wall end detail.

plate thickness in the exterior wall was generally ¼ in. (6.35 m), and at the base, column plates as thick as 4 in. (101.6 mm) were used.

The structural system was considered to constitute a tubular system, acting essentially as a cantilevered hollow tube with perforated walls. The side walls acting as stiff webs transfer shear between the windward and leeward walls, thus creating an efficient 3D structure for resisting lateral loads. In the lower seven stories of the towers where there are fewer columns, vertical diagonal bracing in the building cores provided the lateral stiffness.

Floor construction typically consisted of 4 in. (101.6 mm) of concrete on 1½-in. (38.1 mm) deep, 22-guage noncomposite metal deck. The slab thickness was 5 in. (127 mm) in the core area. Outside the central core, the floor deck was supported by a series of composite floor trusses, 29 in. (0.74 m) deep, open-web joist-type trusses with ASTM A36 steel chord angles and steel rod diagonals. Composite behavior of the truss with the floor slab was achieved by extending the diagonal truss members above the top chord so that they would act much like shear stud (see Figure 9.1g–j). Detailing of theses trusses was similar to that typically used in open-web joist fabrication, but the floor system design was not typical of open-web-joist floor systems. It was considerably more redundant and was well braced with transverse member. Trusses placed in pairs, at 6 ft-8 in. (2.03 m) spacing spanned approximately 60 ft (18.29 m) to the sides and 35 ft (10.67 m) at the ends of the central core. Metal deck spanned parallel to the main trusses and was supported by continuous transverse bridging trusses spaced at 13 ft 4 in. (4.06 m) from the transverse trusses.

In approximately 10,000 locations in each building, viscoelastic dampers extended between the lower chords of the trusses and gusset plates attached to exterior columns (see Figure 9.1i). These dampers were provided to reduce occupant perception of wind-induced motion. Pairs of flat bars extended diagonally from the exterior wall to the top of chord of adjacent trusses. These diagonal flat bars, which were typically provided with shear studs, provided horizontal shear transfer between the floor slab and exterior wall, as well as out-of-plane bracing for perimeter columns not directly supporting floor trusses.

The core framing consisted of 5 in. (127 mm) concrete fill on metal deck supported by floor framing of rolled structural shapes, in turn supported by wide-flange shape and built-up box section columns. Some of these columns measured 14 by 36 in. (0.35 × 0.91 m). For the upper levels, these box columns transitioned into wide-flange shapes. At the top, a total of 10 outrigger trusses were present, six extending across the long direction of the core and four extending across the short direction (see Figure 9.1f). In addition to providing support for a transmission tower (WTC 1 had a transmission tower; WTC 2 did not, but was designed to support such a tower), this outrigger system provided stiffening of the frame for wind resistance.

Prior to construction, the site was underlain by deep deposits of fill material, placed over a period of several hundred years to reclaim the shoreline. In order to construct the towers, perimeter walls for the subterranean structure were constructed using slurry wall and tieback techniques. Tieback anchors were drilled diagonally down through the wall and grouted into position into the rock deep behind the walls.

Floors within the substructure were of reinforced concrete flat-slab construction, supported by structural steel columns. These floors also provide lateral support for the perimeter walls, holding back the earth and water pressures from the unexcavated side of the excavation. The tiebacks, which had been installed as temporary stabilizers, were decommissioned by cutting off their end anchorage hardware and repairing the pockets in the slurry wall where these anchors had existed.

In slurry wall construction, a trench is dug in the eventual location of the perimeter walls. A bentonite slurry is pumped into the trench as it is excavated, to keep the trench open against caving of the surrounding earth. Prefabricated reinforcing steel is lowered into the trench, and concrete is placed through a tremie to create a reinforced concrete wall around the site perimeter. After the concrete is cured, excavation of the structure begins. As the excavation progresses below surrounding grade, tiebacks are drilled through the exposed concrete wall and through the surrounding soil into the rock below to provide stability for the excavation.

Tower foundations beneath the substructure consisted of massive spread footings, socketed into and bearing directly on the massive bedrock. Steel grillages, consisting of layers of orthogonally placed steel beams, were used to transfer the column loads in bearing to the reinforced concrete footings.

On September 11, 2001, two commercial airlines were hijacked, and one was flown into each of the towers. The structural damage sustained by each tower from the impact, combined with the ensuing fires, resulted in the total collapse of both buildings. The north tower was struck between floors 94 and 98, with the impact roughly centered on the north face. The south tower was hit between floors 78 and 84 toward the east side of the south face. Both planes banked steeply with estimated speeds of 470 and 590 mph at the time of impacting the north and south towers, respectively. The population on September 11, 2001, of the seven buildings of the WTC complex has been estimated at 58,000 people. Almost everyone in WTC 1 and 2 who was below the impact area was able to evacuate the buildings, due to the length of time between the impact and collapse of the individual towers.

In each case, the aircraft impacts resulted in severe structural damage, including some localized partial collapse, but did not result in the initiation of global collapse. In fact, WTC 1 remained standing for a period of 1 h 43 min following the initial impact, and WTC 2 for approximately

56 min. The fires heated the structural systems and over a period of time, resulted in additional stressing of the damaged structure, as well as additional damage and strength loss to initiate a progressive sequence of failures that culminated in total collapse of both structures.

Design experts from ASCE and FEMA who investigated the WTC destruction have agreed that it would be futile to create a "terror code" to try to out-design terrorists. The WTC buildings were not required to protect their occupants during the disaster, but on 9/11 did so stunningly. Despite being subjected to stresses that never could have been anticipated, the structural design of the towers kept them standing long enough for more than 20,000 people to evacuate.

9.3.2 Empire State Building, New York

The Empire State Building was the tallest building in the world for more than 40 years, from the day of its completion in 1931 until 1972 when the Twin Towers of New York's World Trade Center exceeded its 1280 ft (381 m) height by almost 120 ft (37 m). The structural steel frame consisting of moment and braced frames with riveted joints, although encased in concrete, was designed to carry 100% of the gravity and wind loads. The concrete encasement, although neglected in strength analysis, stiffened the frame considerably against wind loads. Measured frequencies of the building have estimated the actual stiffness at 4.8 times the stiffness of the bare frame. A schematic elevation of the structural framing is shown in Figure 9.2.

The Empire State Building was completed in the year 1931 and with 102 stories stood no less than 1250 ft high. The height was increased by the addition of a 222 ft TV aerial in 1951. Among the many astonishing features of this building is the fact that it has 73 elevators although not all

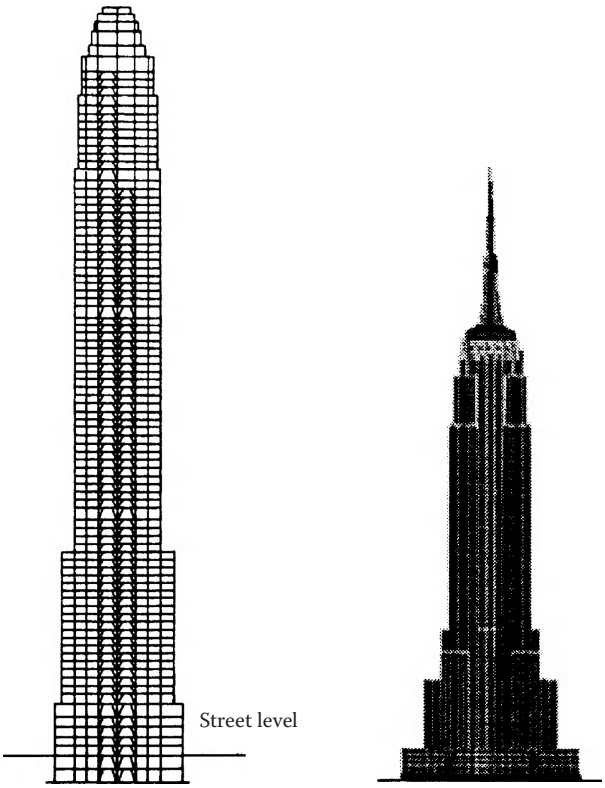


FIGURE 9.2 Empire State Building System; riveted steel frame encased in concrete.

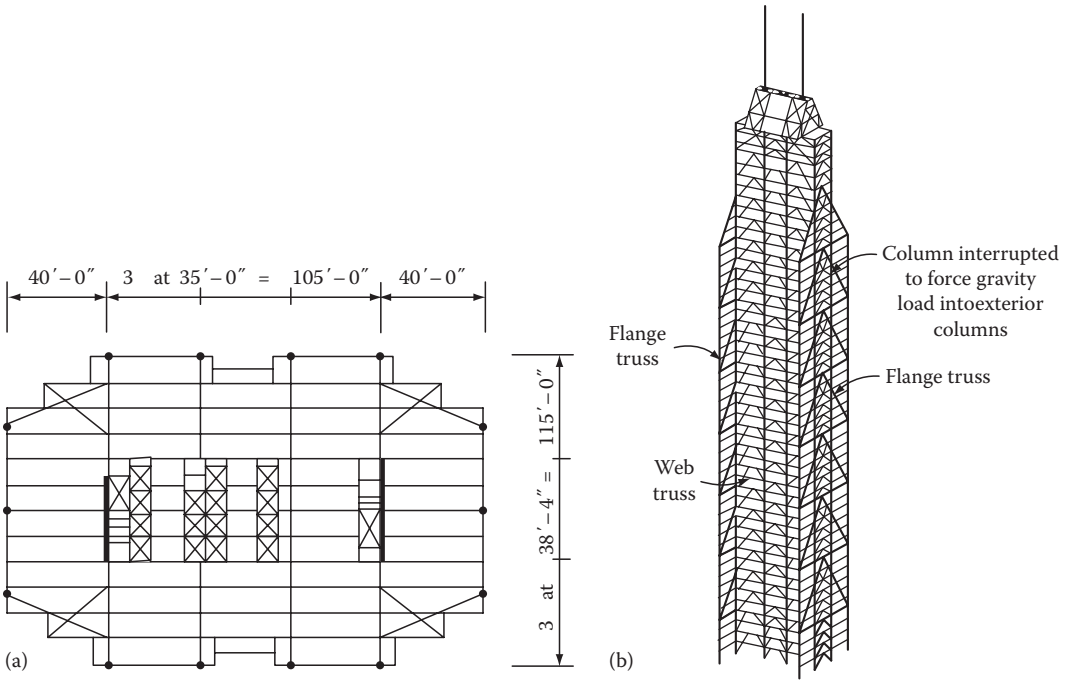


FIGURE 9.3 Bank One Center, Indianapolis: (a) plan; (b) lateral system.

traverse the full height. Amazingly, it was built in a relatively short period of 410 days, this being made possible by the extensive use of prefabricated steel sections so that at one point it was rising by four or five stories a week.

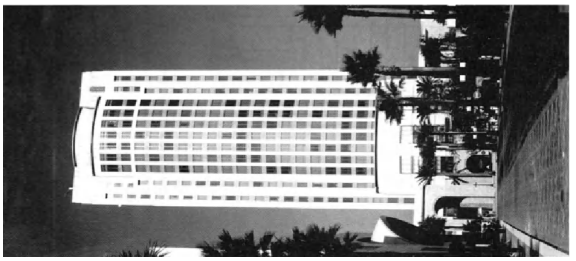
9.3.3 Bank One Center, Indianapolis, Indiana

This is a 52-story steel-framed office building that rises to a height of 623 ft (190 m) above the street level. In the plan, the tower is typically 190 × 120 ft (58 × 37 m) with set-backs at the 10th, 15th, 23rd, 45th, and 47th floors (Figure 9.3).

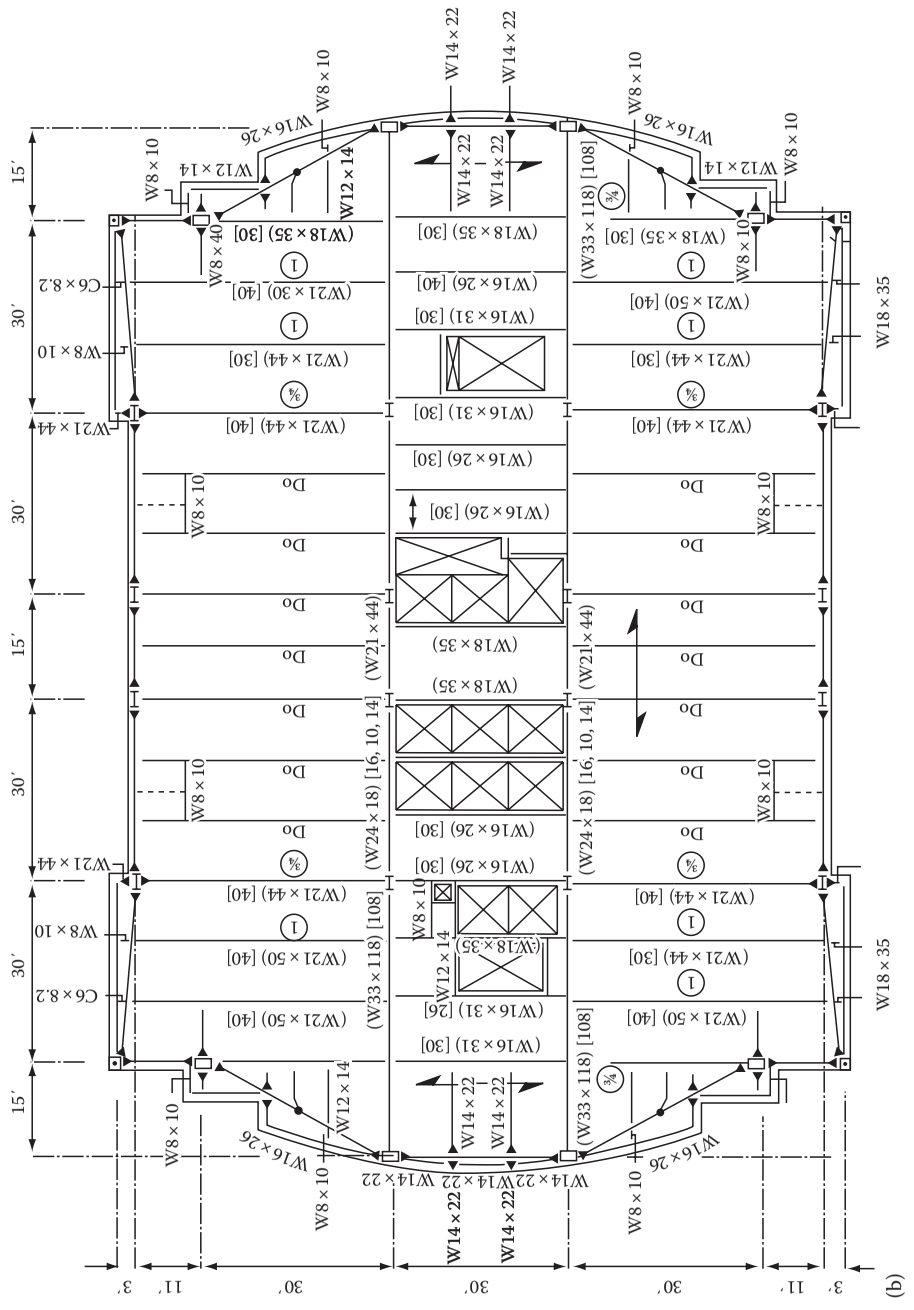
The structural system for resisting lateral forces consists of two large vertical flange trusses in the north-south direction and two smaller core braces in the east-west direction acting as web trusses connecting the flange trusses. The flange trusses, while providing maximum lever arm for resisting the overturning moments, also serve to transfer gravity loads of the core to the exterior columns. The resulting equalization of axial stresses in the truss and nontruss perimeter columns keeps the differential shortening between the two to a minimum. To assure a direct load path for the transfer of gravity load from the core to the truss columns, a core column is removed below the level of braces at every twelfth level, as shown in Figure 9.3b. In addition, the step-back corners are cantilevered to maximize the tributary area of gravity load and thus compensate for the tensile force due to overturning moments. The structural design is by LeMessurier Consultants, Inc., Cambridge, Massachusetts.

9.3.4 MTA Headquarters, Los Angeles, California

This 28-story office building, shown in Figure 9.4, has a four-level subterranean structure that will serve as a common base for the MTA Tower and two future office buildings. The basement extends beyond the footprint of the tower and consists of precast reinforced concrete columns and girders with a cast-in-place concrete slab.



(a)


W14 × 22
W8 × 10
W16 × 26
W12 × 14
W8 × 40
(W18 × 35) [30]
W8 × 10
W12 × 14
W16 × 26
(W33 × 118) [108]
W18 × 35
W21 × 44
W8 × 10
W8 × 10
W21 × 44
W8 × 10
W8 × 10
W21 × 44
W8 × 10
W8 × 10

The plaza underneath the tower consists of a composite floor system with a 4½ in. (114 mm) normal-weight concrete topping on a 3 in. (76 mm) deep, 18-guage composite metal deck. The metal deck spans between composite steel beams spaced typically at 7 ft 6 in. (2.29 m) outside the tower, which has a heavy landscape. The beam spacing is at 10 ft (3.04 m) on centers within the tower footprint.

The building is essentially rectangular in plan, 118 × 165 ft (36 × 50.3 m) with a slight radius on the short faces. The height is 400 ft (122 m), resulting in a fairly low height-to-width ratio of 3.39. Typical floor framing consists of 21 in. (0.54 m) deep composite beams spanning 41 ft (12.5 m) from the core to the exterior. A 3 in. (76 mm) deep metal deck with a 3¼ in. (83 mm) thick lightweight concrete topping completes the floor system (see Figure 9.4b).

The lateral system consists of a perimeter tube with widely spaced columns tied together with spandrel beams. The exterior columns on the broad faces vary from W30 × 526 at the plaza to W30 × 261 at the top. The spandrels vary from WTM36 × 286 at the plaza level to W36 × 170 at the top floors. The columns on the curvilinear faces are built up, 34 × 16 in. (0.87 × 0.40 m) box columns while 24 × 24 in. (0.61 × 0.61 m) box columns are used at the corners. Plates varying in thickness from 4 in. (102 mm) at the bottom to 1 in. (25 mm) at the top are used for the built-up columns.

The architecture for the building is the McLarand Vasquez and Partners, Inc., while the structural engineering is by John A. Martin and Associates, both of Los Angeles, California.

9.3.5 AT&T Building, New York City, New York

The basic lateral-force-resisting structural system for the building shown in Figure 9.5 consists of a rigid-frame steel tube at the building perimeter. Additional stiffness is added along the width of the building by means of four vertical steel trusses. At every eighth floor, two I-shaped steel plate walls, with holes cut for circulations, extend from the sides of the trusses to the exterior columns on the same column line. The steel walls act as outrigger trusses mobilizing the full width of the building

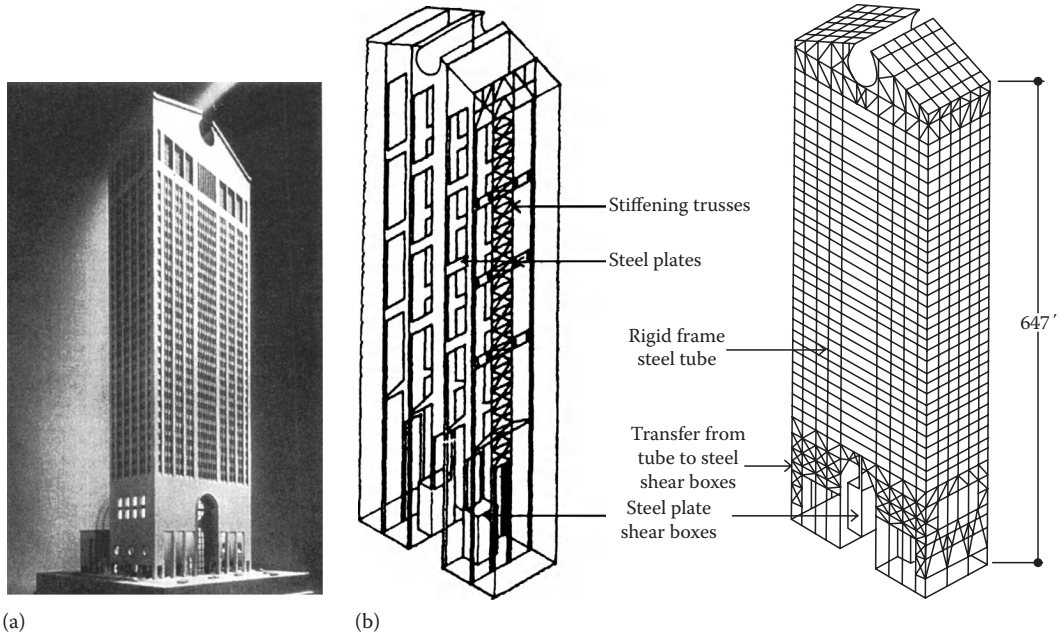


FIGURE 9.5 AT&T Building, New York: (a) building elevation; (b) lateral system. Perimeter steel tube interacts with interior braces. Steel plate outriggers interact with tube and braces.

in resisting lateral forces. The horizontal shear at the base of the building is transferred to two giant steel plate boxes (Figure 9.5). Structural design is by Leslie Robertson and Associates, New York.

9.3.6 Miglin-Beitler Tower, Chicago, Illinois

The proposed Miglin-Beitler Tower, designed by the New York Office of Thornton-Tomasetti Engineers, will rise to the height of 1486.5 ft (453 m) at the upper sky room level, 1584.5 ft (483 m) at the top of the mechanical areas, and finally to 1999.9 ft (609.7 m) at the tip of the spire. An elevation and the schematic plan of the proposed building are shown in Figure 9.6. The structural system consists of five major components as shown in Figure 9.6c.

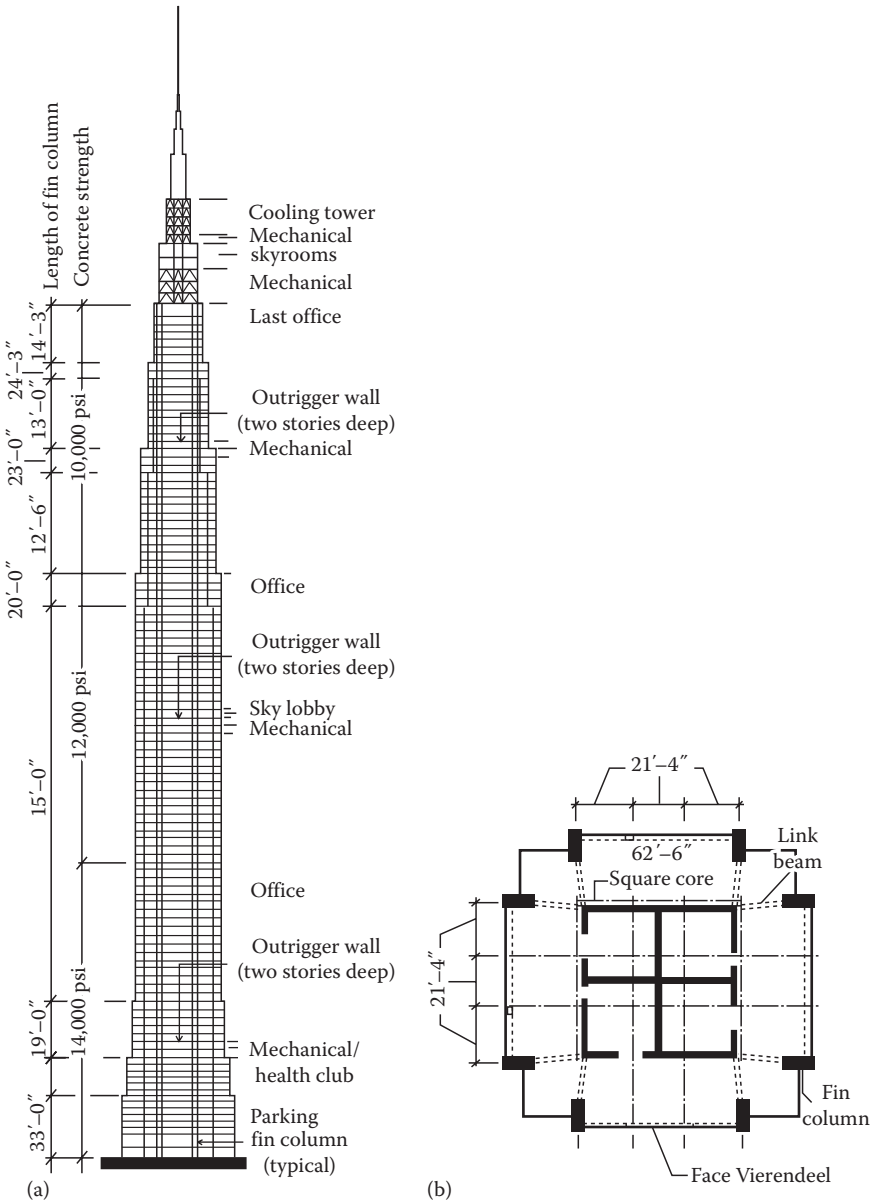


FIGURE 9.6 Miglin-Beitler Tower, Chicago: (a) elevation; (b) plan;

(*continued*)

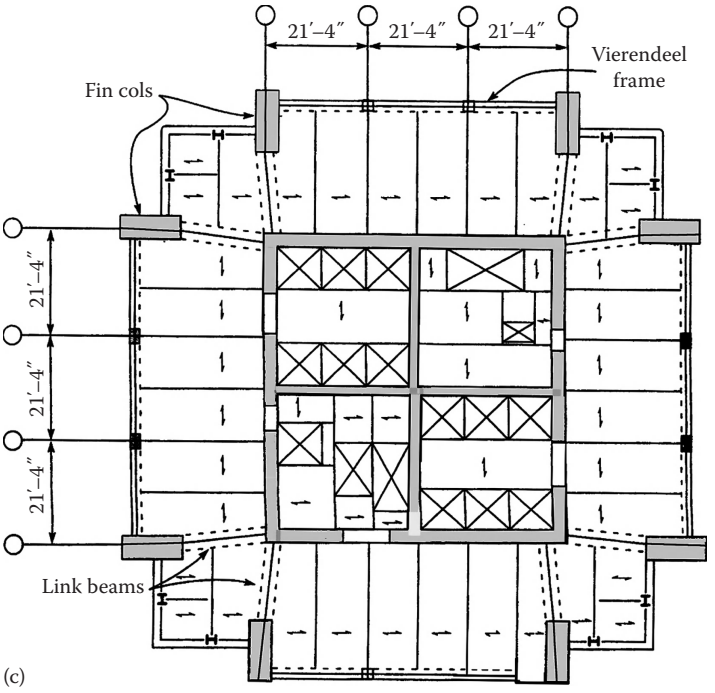


FIGURE 9.6 (continued) (c) typical floor framing plan.

1. A 62 ft–6 in. × 62 ft–6 in. (19 × 19 m) concrete core with walls varying from a maximum thickness of 3 ft (0.91 m) to a minimum thickness of 1 ft–6 in. (0.46 m).
2. A conventional structural steel composite floor system consisting of 18 in. (0.46 m) deep-rolled steel sections spaced 10 ft (3.05 m) on center with 3 in. (74 mm) deep corrugated metal deck and a 3½ in. (89 mm) thick normal-weight concrete topping. The steel floor system is supported on light steel erection columns that allow the steel construction to precede 8–10 floors ahead of concrete operation.
3. Concrete fin columns, each of which encases a pair of steel erection columns located at the face of the building. These fin columns, which extend 20 ft (6.10 m) beyond the 140 × 140 ft (42.7 × 42.7 m) footprint of the building, vary in dimension from 6½ × 33 ft (2.0 × 10 m) at the base, 5½ × 15 ft (1/68 × 4.6 m) at the middle, to 4½ × 13 ft (1.38 × 4 m) near the top.
4. Concrete link beams that interconnect the four corners of the core to the eight fin columns at every floor. These beams tie the fin columns to the core, thus engaging the full structural width of the building to resist lateral loads. In addition to the link beams at each floor, there are two-story-deep outrigger walls located at the 16th, 56th, and 91st stories. These outrigger walls further enhance the structural rigidity by linking the exterior fin columns to the concrete ones.
5. Exterior Vierendeel trusses comprising a horizontal spandrel and two columns at each of the 60 ft (18.3 m) faces on the four sides of the building. These Vierendeels supplement the lateral force resistance and also improve the torsional resistance of the structural system. Additionally, these trusses transfer gravity loads to the exterior fin columns, thus minimizing uplift forces.

The proposed foundation system is rock caisson varying in diameter from 8 to 10 ft (2.44 × 3.0 m). The caisson will have a straight shaft steel casing and will be embedded into rock for a minimum of

6 ft (1.88 m). The length of these caissons is 95 ft (29 m). A 4 ft (1.22 m) thick concrete mat will tie the caissons and provide a means for resisting the shear forces at the base of the building. The bottom of the mat will be cast in a two-directional groove pattern to engage the soil in shear. Passive pressure on the edge of the mat and on the projected side surface of the caisson will provide additional resistance to shear at the base.

9.3.7 One Detroit Center, Detroit, Michigan

This is a 45-story office tower with a clear 45 ft-6 in. (13.87 m) span between the core and the exterior (Figure 9.7). The structural system consists of eight composite concrete columns measuring 7 ft-6 in. × 4 ft-9 in. (2.28 × 1.45 m) at the base, placed 20 ft (6.1 m) away from the corners to provide column-free corner offices and also to optimize the free-span of the Vierendeel frames. The composite columns are connected at each face by a system of perimeter columns and spandrels acting as Vierendeel frames. The Vierendeels are stacked four stories high and span between composite super-columns to provide column-free entrances at the base of the tower. At each fourth level, the Vierendeels are linked by hinges to transfer only horizontal shear between adjoining Vierendeels and not gravity loads. The reason for this type of connection is to reduce: (1) the effect of creep and shrinkage of super-columns on the members and connections of the Vierendeel, and (2) gravity load transfer due to arch action of the Vierendeel with associated horizontal thrusts. The four-story Vierendeel achieves uniformity in the transfer of moment and shear between horizontal steel beams and composite super-columns throughout the height of the tower.

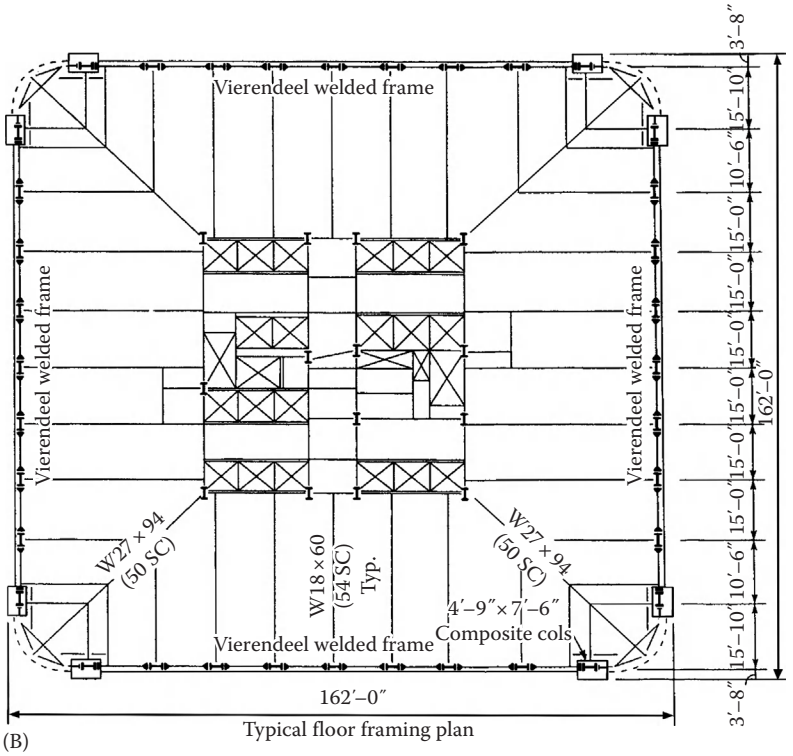
A schematic representation of the structural system is shown in Figure 9.7a through d. Figure 9.7d shows the connection details for the Vierendeel frame. The structural design is by CBM Engineers, Inc., Houston, Texas.



(A)

FIGURE 9.7 One Detroit Center: (A) building elevation;

(*continued*)



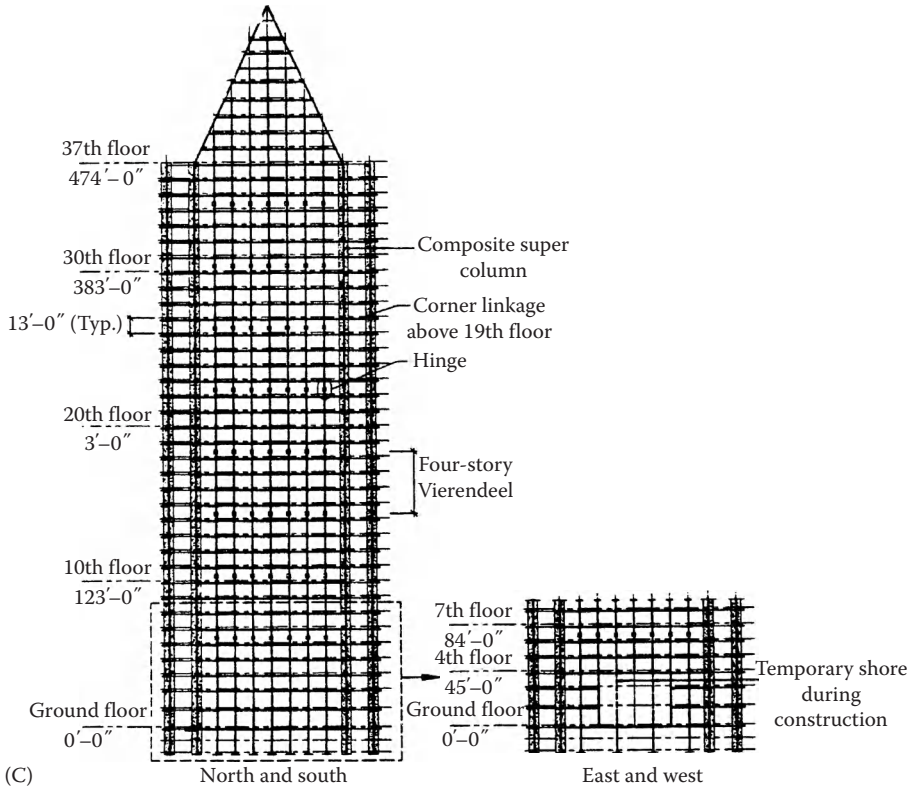


FIGURE 9.7 (continued) (B) typical floor framing plan; (C) free-spanning Vierendeel frame;

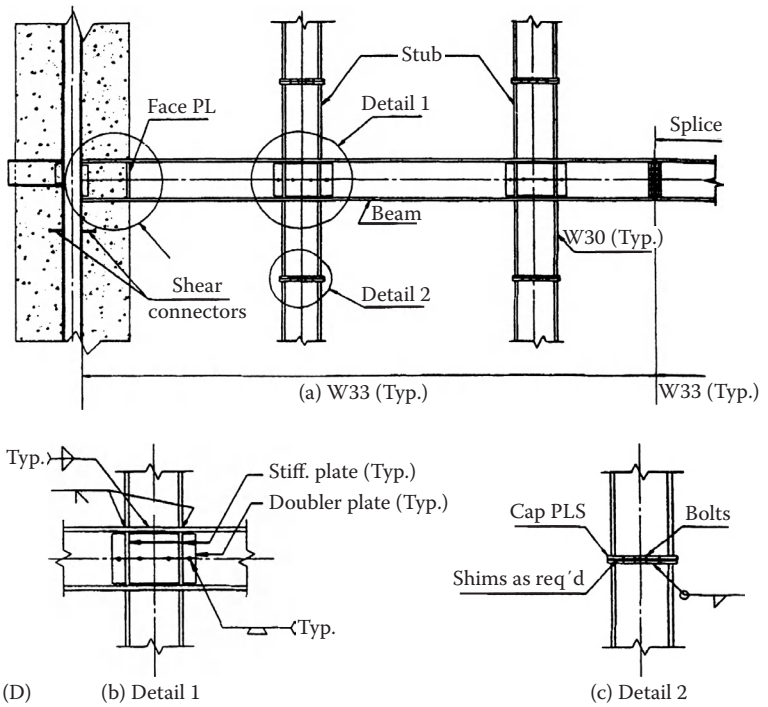


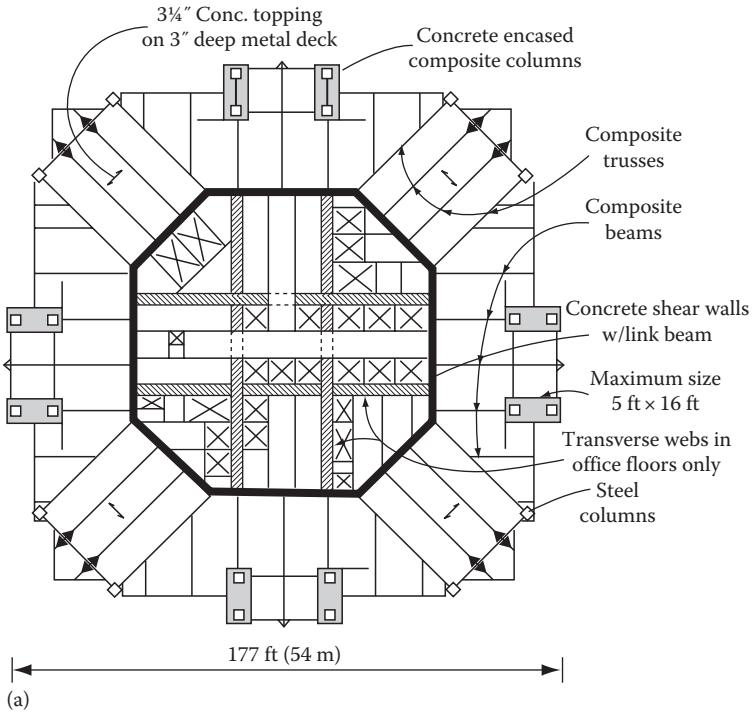
FIGURE 9.7 (continued) (D) structural details for Vierendeel frame, (a) partial elevation, (b) detail 1, (c) detail 2.

9.3.8 Jin Mao Tower, Shanghai, China

This building consists of a 1381 ft (421 m) tower and attached low-rise podium for a total gross building area of approximately 3 million square feet (278,682 m^2). The building includes 50 stories of office space topped by 36 stories of hotel space with two additional floors for a restaurant and an observation deck. Parking for automobiles and bicycles is located below grade. The podium consists of a retail space as well as an auditorium and exposition spaces.

The superstructure is a mixed use of structural steel and reinforced concrete with many major structural members composed of both steel and concrete. The primary components of the lateral system include a central reinforced concrete core linked to exterior composite mega-columns by outrigger trusses (Figure 9.8). A central shear-wall core houses the primary buildings functions including elevators, mechanical fan rooms, and washrooms. The octagon-shaped core, nominally 90 ft (27.43 m) from centerline to centerline of perimeter flanges, is present from the foundation to level 87. Flanges of the core typically vary from 38 in. (84 m) thick at the foundation to 18 in. (46 m) at level 87 with concrete strengths varying from 7500 to 5000 psi (51.71 to 34.5 MPa). Four 18 in. (46 cm) thick interconnecting core wall webs exist through the office floors. The central area of the core is open throughout the hotel floor, creating an atrium that leads into the spire with a total height of approximately 675 ft (206 m). The size of composite mega-columns varies from 5 × 16 ft (1.5 × 4.88 m) with a concrete strength of 7500 psi (51.71 MPa) at the foundation to 3 × 11 ft (0.91 × 3.53 m) with a concrete strength of 5000 psi (34.5 MPa) at level 87.

The shear-wall core is directly linked to the exterior composite mega-columns by structural steel outrigger trusses. The outrigger trusses resist lateral loads by maximizing the effective depth of the structure. Under bending, the building acts as a vertical cantilever with tension in the windward columns and compression in the leeward columns. Gravity load framing minimizes uplift in the exterior



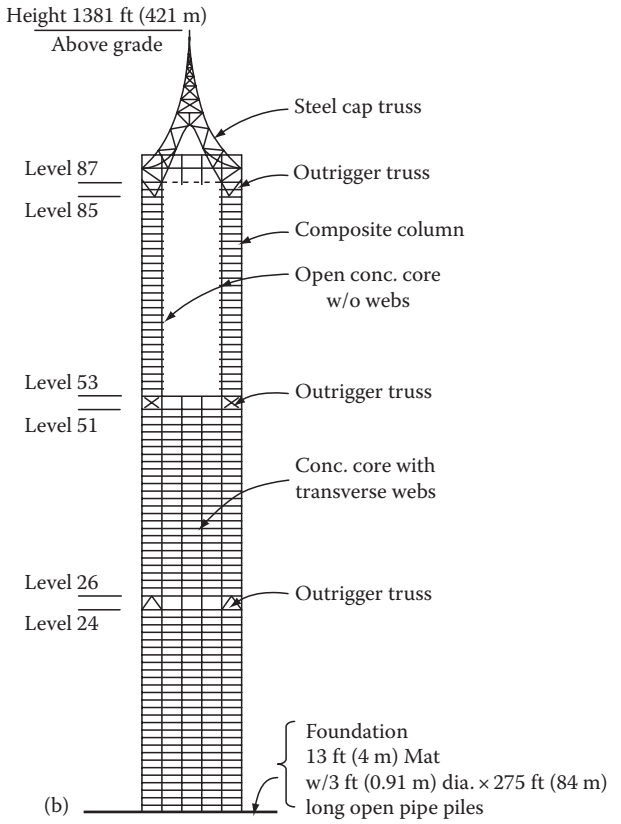


FIGURE 9.8 Jin Mao Tower, Shanghai, China: (a) typical office floor framing plan; (b) structural system elevation;



(c)

FIGURE 9.8 (continued) (c) photograph.

composite mega-columns. The octagon-shaped core provides exceptional torsional resistance, eliminating the need for any exterior belt or frame systems to interconnect exterior columns.

The outrigger trusses are located between levels 24 and 26, 51 and 53, and 85 and 87. The outrigger truss system between levels 85 and 87 is capped with a 3D steel space frame that provides for the transfer of lateral loads between the core and the exterior composite columns. It also supports gravity loads of heavy mechanical spaces located in the penthouse floors.

The structural elements for resisting gravity loads include eight structural steel built-up columns. Composite wide-flange beams and trusses are used to frame the floors. The floor-framing elements are typically 14 ft 6 in. (4.4 m) on center with a composite 3 in. (7.6 cm) deep metal deck and a 3¼ in. (8.25 cm) thick normal-weight concrete topping slab spanning between the steel members.

The foundation system for the Tower consists of high-capacity piles capped with a reinforced concrete mat. High-water conditions required the use of a 3 ft–3 in. (1 m) thick, 100 ft (30 m) deep, continuous reinforced concrete slurry wall diaphragm along the 0.5 mile (805 m) perimeter of the site.

The high-capacity pile system consists of a 3 ft (0.91 m) diameter structural steel open-pipe pile with a 7/8 in. (2.22 cm) thick wall typically spaced 9 ft (2.75 m) on center capped by a 13 ft (4 m) deep reinforced concrete mat. Since soil conditions at the upper strata are so poor, the piles were driven into a deep, stiff sand layer located approximately 275 ft (84 m) below grade. The individual design-pile capacity is 1650 kip (7340 kN).

Wind speeds can average 125 mph (56 m/s) at the top of the building over a 10 min time period during a typhoon event. The earthquake ground accelerations compare to 1994 UBC zone 2A. The overall building drift index for a 50 year return wind with a 2.5% structural damping is 1/1142. This increases to 1/887 for a future developed condition in which two tall structures are proposed adjacent to the Jin Mao Building. The drift index based on specific Chinese code-defined winds, which were equivalent to a 3000 year wind, is 1/575.

The structural design for the tower is governed by its dynamic behavior under wind and not by its strength or its overall or interstory drift. The calculated fundamental translational periods are 5.7 s. for each principal axis. The torsional period is 2.5 s.

In a force-balance and aeroelastic wind tunnel study, the accelerations at the top floors were evaluated using a value of 1.5% for structural damping. The accelerations measured in the wind tunnel were between 9 and 13 milli-g's for a 10 year return period, and between 3 and 5 milli-g's for a 1 year return period—well within the generally accepted range of 20–25 milli-g's for a 10 year return. Only the passive characteristics of the structural system including its inherent mass, stiffness, and damping are required to control the dynamic behavior. Therefore, no mechanical damping systems are used.

Since the central core and composite mega-columns are interconnected by outrigger trusses at only three 2-story levels, the stresses in the trusses due to differential shortening of the core relative to the composite columns were of concern. Therefore, concrete stress levels in the core and mega-columns were controlled in an attempt to reduce relative movements. To further reduce the adverse effect of differential shortening, slotted connections were used in the trusses during the construction period of the building. Final bolting with hard connections was done after completion of construction to relieve the effect of differential shortening occurring during construction. The architecture and structural engineering of the building is by the Chicago office of Skidmore, Owings, and Merrill.

The Jin Mao Tower is a landmark skyscraper in the Lujiazui area of Pudong District of Shanghai. Its architecture draws the traditional architectures such as tiered pagoda, gently sloping back to create a rhythmic pattern as it raises. As stated previously, the tower is built around an octagon shaped shear wall core surrounded by eight exterior composite super-columns and eight steel columns. Three sets of two-story high outrigger trusses connect the columns to the core at six of the floors to provide additional resistance to overturning.

9.3.9 Petronas Towers, Malaysia

Two 1476 ft (450 m) towers and a sky bridge connecting the twin towers characterize the buildings in Kuala Lumpur, Malaysia (Figure 9.9).

The towers have 88 numbered levels but are in fact equal to 95 stories when mezzanines and extra-tall floors are considered. In addition to 6,027,800 ft^2 (560,000 m^2) of office space, the project includes 1,501,000 ft^2 (140,000 m^2) of retail and entertainment space in a six-story structure linking the base of the towers, plus parking for 7000 vehicles in five below-ground levels.

The lateral system for the towers is of reinforced concrete consisting of a central core, perimeter columns, and ring beams using concrete strength up to 11,600 psi (80 MPa). The foundation system consists of pile and friction barrette foundations with a foundation mat.

The typical floor system consists of wide-flange beams spanning from the core to the ring beams. A 2 in. deep composite metal deck system with a 4¼ in. (110 mm) concrete topping completes the floor system.

Architecturally, the towers are cylinders 152 ft (46.2 in.) in diameter formed by 16 columns. The façade between columns has pointed projections alternating with arcs, giving unobstructed views through glass and metal curtain walls on all sides. The floor plate geometry is composed of two rotated and superimposed squares overlaid with a ring of small circles. The towers have setbacks at levels 60, 72, 82, and 88 and circular appendages at level 44. Concrete perimeter framing is used up to level 84. Above this level, steel columns and ring beams support the last few floors and a pointed pinnacle.

The towers are slender with an aspect ratio of 8.64 (calculated to level 88). The design wind speed in Kuala Lumpur area is based on 65 mph (35.1 m/s) peak, 3 s gusts at 33 ft (10 m) above grade for a 50 year return. In terms of the old U.S. standard of fastest mile wild, the corresponding wind speed is about 52 mph (28.1 m/s).

The mass and stiffness of concrete are taken advantage of in resisting lateral loads, whereas the advantages of speed of erection and long-span capability of structural steel are used in the floor framing system. The building density is about 18 lb/ft^3 (290 kg/m^3).

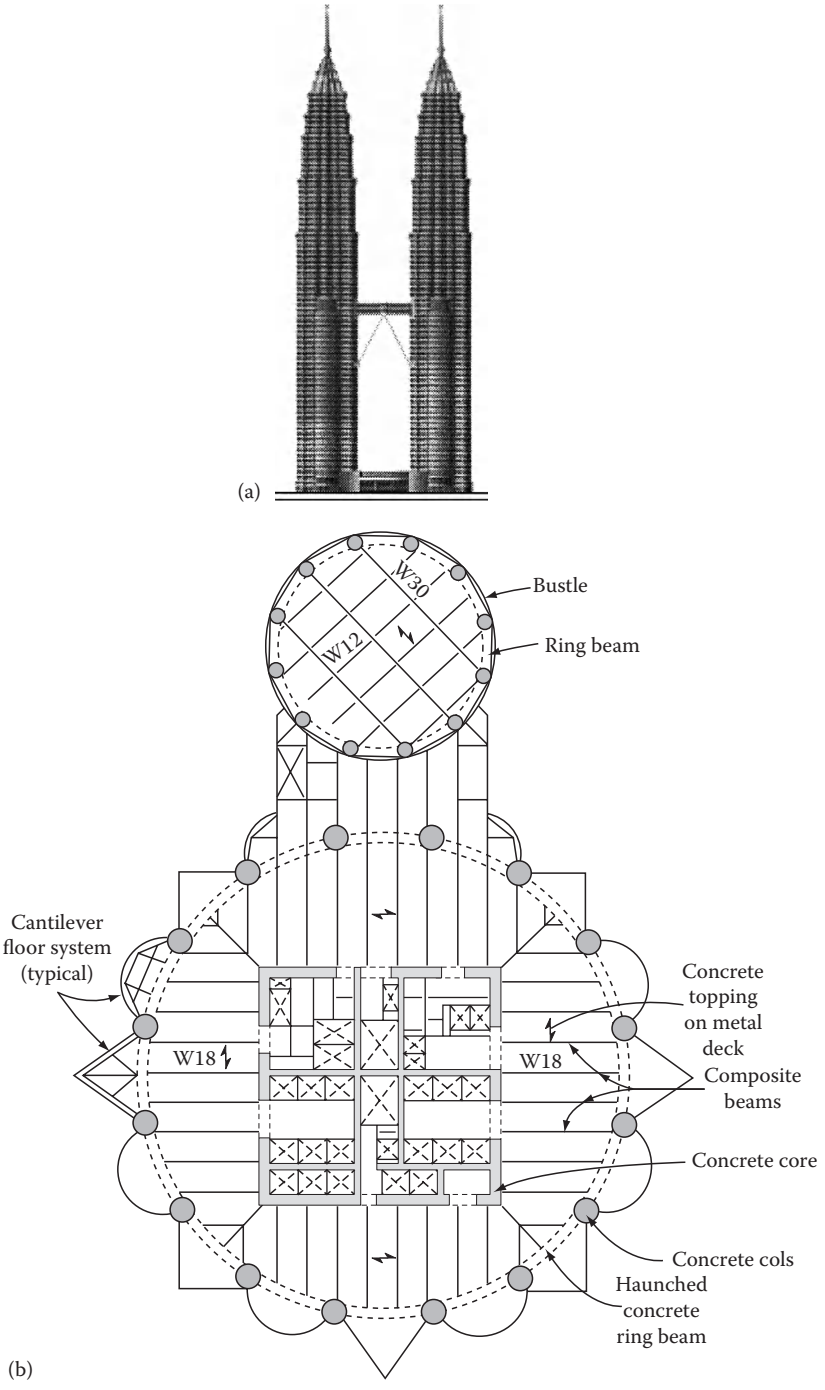


FIGURE 9.9 Petronas Twin Towers, Kuala Lumpur, Malaysia: (a) elevation; (b) structural system plan.

As is common for tall buildings of high aspect ratios, the towers were wind-tunnel tested to determine dynamic characteristics of the building in terms of occupant perception of wind movements and acceleration on the upper floors. The 10 year return period acceleration is in the range of 20 mg, within the normally accepted criterion of 25 mg. The periods for the primary lateral modes are about 9 s, while the torsional mode has a period of about 6 s. The drift index for lateral displacement is of the

order of 1/560. Movement between the towers is facilitated by a bridge at mid-level. The bridge is not a part of the bracing structure. It allows the towers to move independently in response to natural forces.

Because the limestone bedrock lies 200 ft (60 m) to more than 330 ft (100 m) below dense silty sand formation, it was not feasible to extend the foundations to bedrock. A system of drilled friction piers was designed for the foundation, but barrettes (slurry-wall concrete segments) proposed as an alternative system by the contractor were installed. A 14.8 ft (4.5 m) thick mat supports the 16 tower columns and 12 bustle columns. The floor corners of alternating right angles and arcs are cantilevered from the perimeter ring beams. Hunched ring beams varying from 46 in. (1.17 m) deep at columns to 31 in. (0.78 m) at midspan are used to allow for ductwork in office space outside of the ring beams. A similar approach with a midspan depth of 31 in. (0.78 m) is used in the bustles. The haunches are used primarily to increase the stiffness of the ring beams.

The central core for each tower houses elevators, exit stairs, and mechanical services, while the bustles have solid walls. The core and bustle walls carry about one-half the overturning moment at the foundation level.

Each core is 75 ft (23 m) square at the base, rising in four steps to 62 × 72 ft (18.8 × 22 m). Inner walls are constant 14 in. (350 mm) thick while outer walls vary from 30 to 14 in. (750 to 350 mm). The concrete strength varies from 11,600 to 5,800 psi (80 to 40 MPa).

To increase the efficiency of the lateral system, the interior core and exterior frame are tied together by a two-story-deep outrigger truss at the mechanical equipment room (level 38). A Vierendeel type of truss with three levels of relatively shallow beams connected by a midpoint column is used to give flexibility in planning of building occupancy.

The tower floors, Figure 9.9b, typically consist of composite metal deck with concrete topping varying from 4½ in. (110 mm) in offices to 8 in. (200 mm) on mechanical floors, including a 2 in. (53 mm) deep composite metal deck. Wide-flange beams frame the floors at spans up to 42 ft (12.8 m), and are W18 or shallower on most floors to provide room for ductwork, sprinklers, and lights.

Cantilevers for the points beyond and ring beams are 3.28 ft (1 m) deep prefabricated steel trusses. For the arcs, the cantilevers are beams propped with kickers back to the columns. Trusses and beams are connected to tower columns by embedded high-strength bolts. The structural engineering is by Thornton-Tomasetti Engineers, and Ranhill Bersekutu Sdn. Bhd.

9.3.10 One-Ninety-One Peachtree, Atlanta, Georgia

This 50-story building (Figure 9.10a), uses the concept of composite partial tube, as shown in Figure 9.10b. The partial tubes, which extend uninterrupted from the foundation to the 50th floor, consist of concrete columns encasing steel erection columns with cast-in-place concrete spandrels. The building interior is an all-steel structure with composite steel beams supported on steel columns (Figure 9.10b).

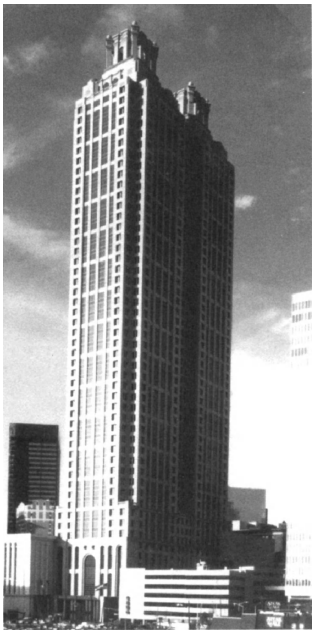
Since the building did not achieve the lateral resistance until after the concrete had reached a substantial strength, a system of temporary bracing was provided in the core. The erected steel was allowed to proceed 12 floors above the completed composite frame with 6 floors of metal deck and six floors of concreted floors. The structural design is by CBM Engineers, Inc., Houston, Texas.

9.3.11 Nations Bank Plaza, Atlanta, Georgia

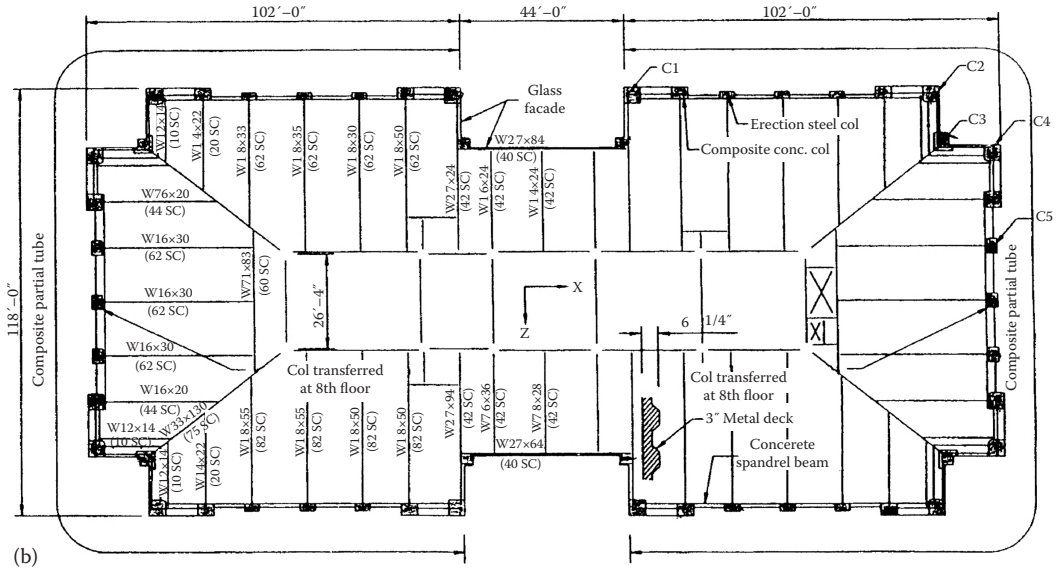
The 57-story office building has a square plan with the corners to create the desired architectural appearance and to provide for more corner offices (Figure 9.11). The typical floor plan (Figure 8.15b) is 162 × 162 ft (49.87 × 49.87 m) with an interior core measuring 58 ft-8 in. × 66 ft-8 in. (17.89 × 20.32 m). A five-level basement provided below the tower is of reinforced concrete construction. The foundation consists of shallow drilled piers bearing on rock.

The gravity load is primarily supported by 12 composite super columns. Four of these are located at the corner of the core, and eight at the perimeter, as shown in Figure 9.11b.

The core columns are braced on all four sides with diagonal bracing as shown schematically in Figure 9.11c. Since the braces are arranged to clear door openings in the core, their configuration



(a)



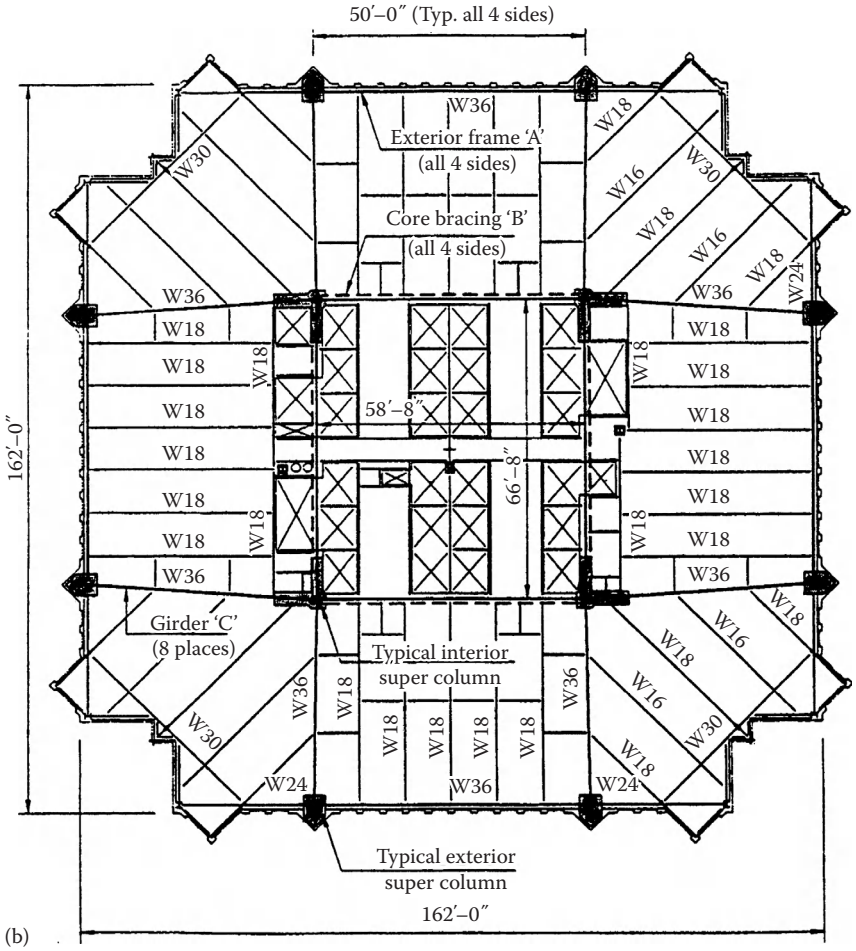
(b)

FIGURE 9.10 One-Ninety-One Peachtree, Atlanta: (a) building elevation; (b) typical floor framing plan.

is different on all four sides. Steel girders 36 in. (0.91 m) deep are moment-connected between the composite columns to transfer part of the overturning moment to the exterior columns. Because the girders are deeper than other gravity beams, openings have been provided in the girders to provide for the passage of mechanical ducts and pipes. A diagonal truss is used between levels 56 and 59 to tie the core columns to the perimeter super columns. These trusses transfer part of the overturning moment to the perimeter columns and also add considerable stiffness to the building. Above the 57th floor, the building tapers to form a 140 ft (42.68 m) tall cone head, which is used to house mechanical and telecommunication equipment. The structural design is by CBM Engineers, Inc., Houston, Texas.



(a)



(b)

FIGURE 9.11 Nations Bank Plaza, Atlanta, Georgia: (a) building elevation; (b) typical floor framing plan;

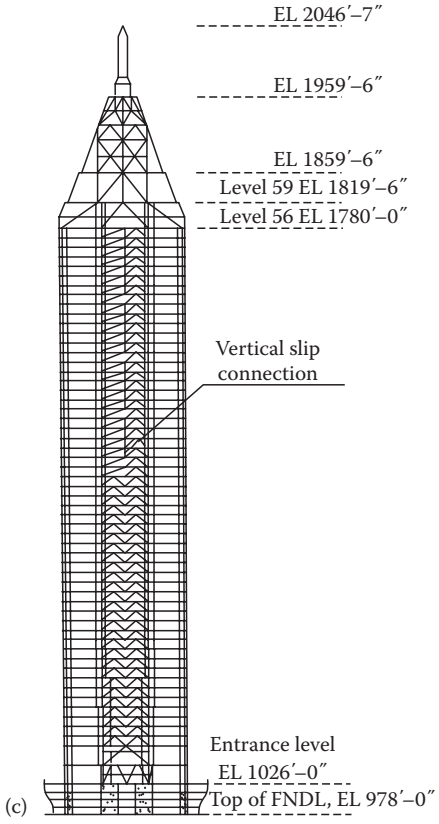


FIGURE 9.11 (continued) (c) section.

9.3.12 U.S. Bank Tower First Interstate World Center, Library Square, Los Angeles, California

This 75-story granite-clad building (Figure 9.12a) sports multiple step-backs. The structural system is a dual system consisting of an uninterrupted 73 ft 10 in. (22.5 m) square-braced spine (core) interacting with a perimeter ductile moment-resisting frame. The spine has a two-story tall chevron bracing core, as shown in Figure 9.12d.

The 55 ft (16.76 m) span for the floor beams coupled with the two-story tall free spanning core loads the corner core columns in such a way that the design is primarily governed by gravity design. To achieve overall economy and take advantage of the increase in allowable stresses then permitted under combined gravity and lateral loads, the columns are widely spaced to collect gravity loads with large tributary areas (Figure 9.12e). The column design is primarily for gravity loads with the additional loads due to seismic and wind resisted by the one-third increase in allowable stresses. As required in seismic design, the strong column weak/beam concept is maintained in the design of beam-column assemblies of the perimeter frame tube.

The sustained dead weight of the structures is 204,000 kip (972,272 kN), with the fundamental periods of vibration T_x = 7.46 s, T_y = 6.95 s, and $T_{z(s)}$ = 3.57 s. The interaction between the interior braced core and the perimeter ductile frame is typical of dual systems with the shear resistance of core increasing progressively from the top to the base of the building. Nearly 50% of the overturning moment is resisted by the core. The maximum calculated lateral deflection at the top under a 100 year wind is 23 in. (584 mm).

The structure is founded on shale rock with an allowable bearing capacity of 7.5 ton/ft^2 (720 kPa). The building core is supported on an 11.5 ft (3.5 m) thick concrete mat while a perimeter ring footing

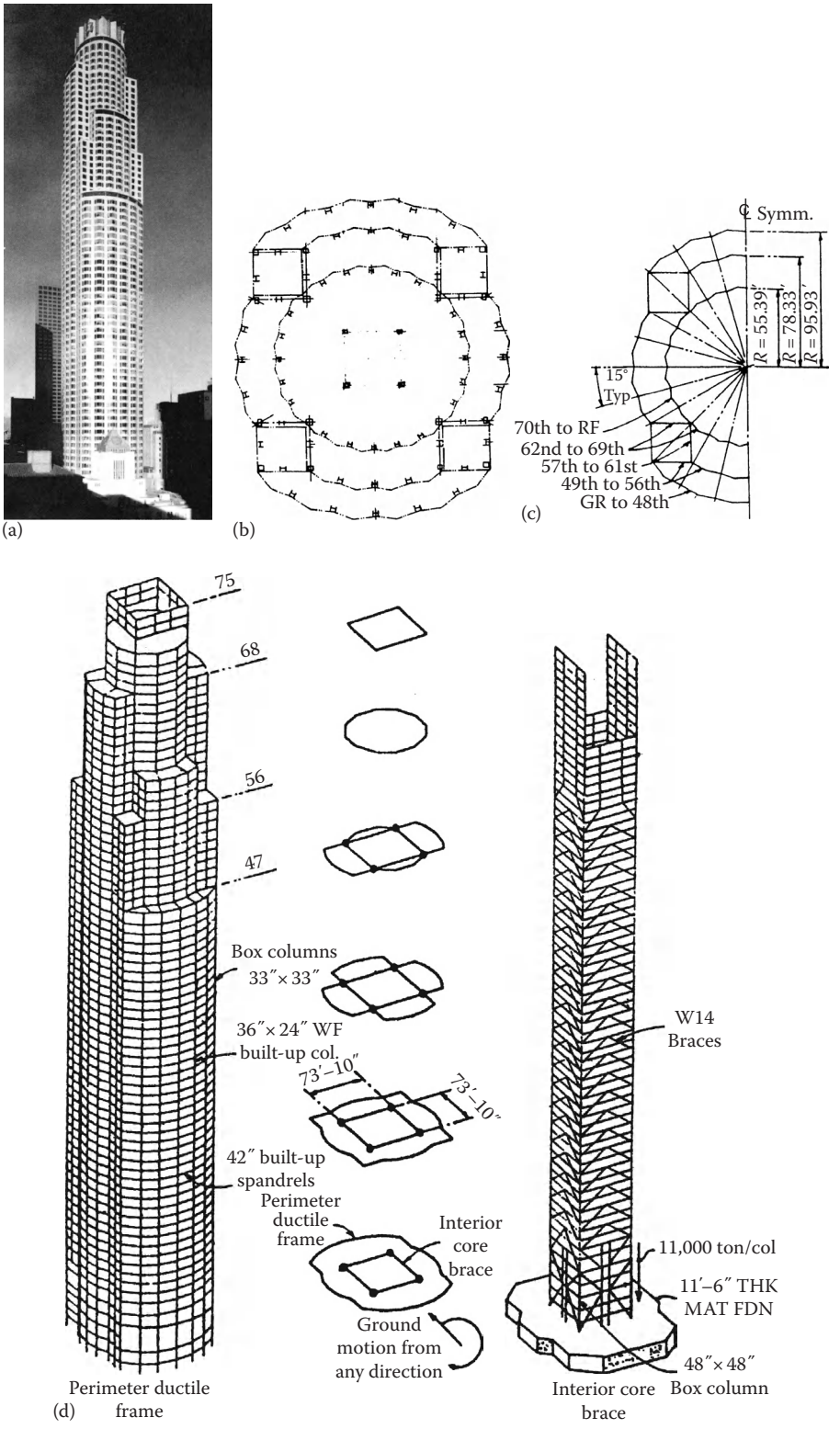


FIGURE 9.12 First Interstate World Center, Los Angeles, California: (a) elevation, (b) plan showing column transfers, (c) composite plan, (d) structural system,

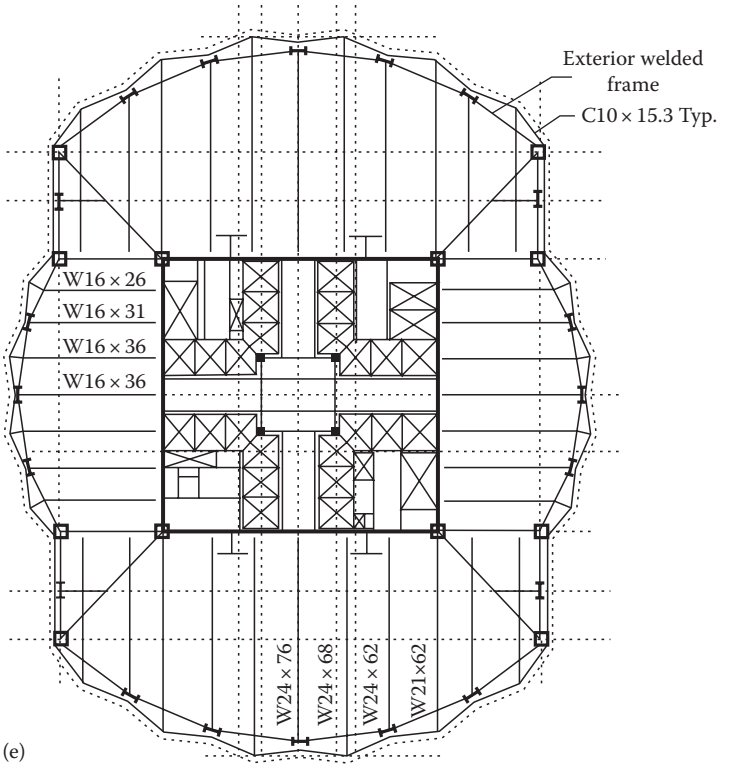


FIGURE 9.12 (continued) (e) framing plan.

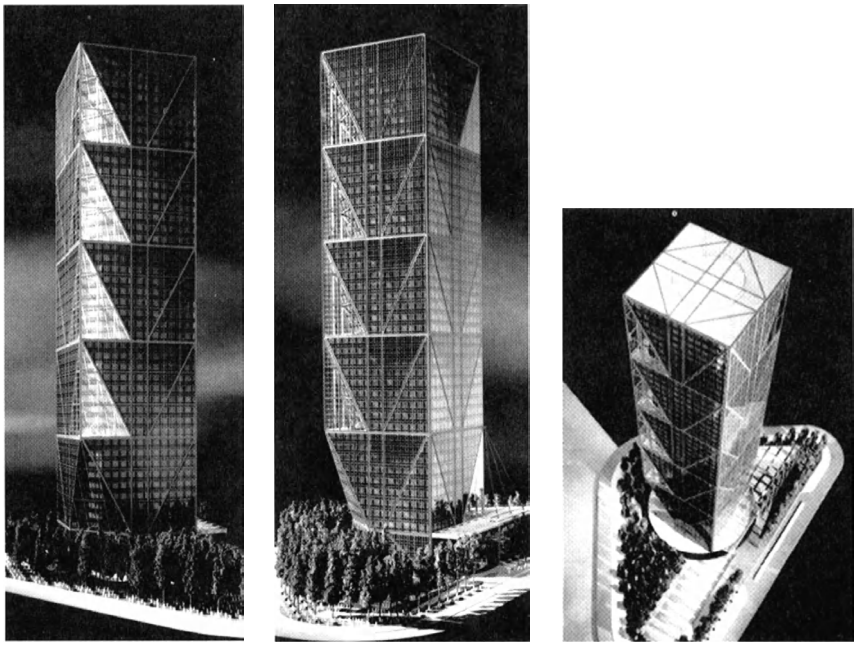
supports the ductile frame. Typical floor framing consists of W24 wide-flange composite beams spaced at 13 ft centers, spanning a maximum of 55 ft (16.76 m) from the core to the perimeter. The structural design is by CBM Engineers, Inc., Houston, Texas.

9.3.13 21st Century Tower, China

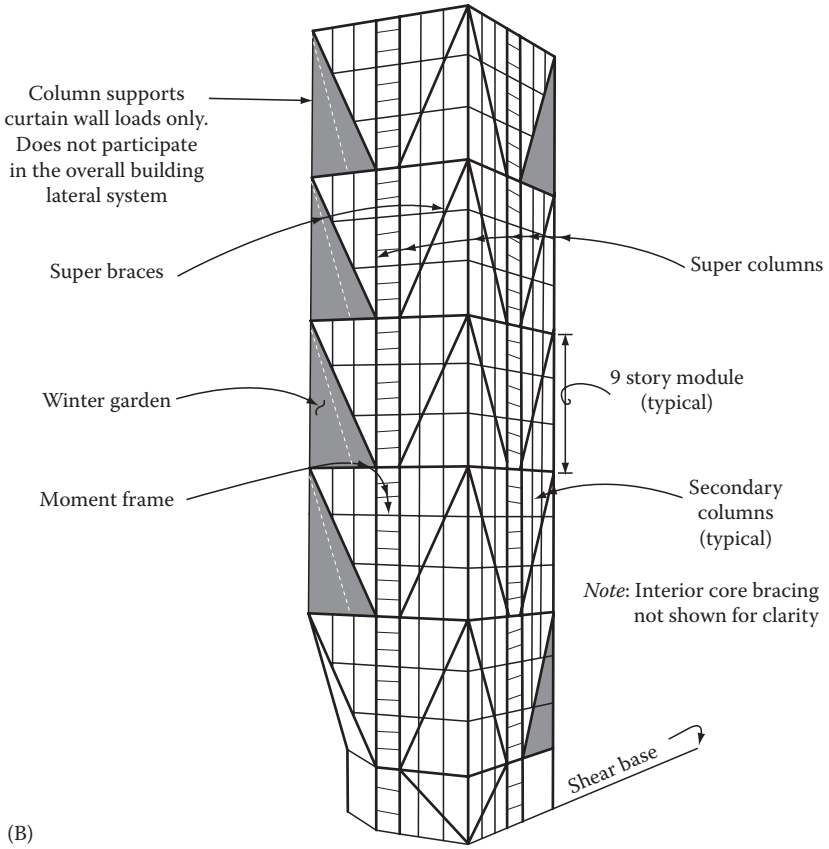
The exterior form of this proposed building in Shanghai, China, is that of a rectangular tower. The building however, has a setback base and a series of nine-story-high wedge-shaped atria or "winter gardens," which run the full height of the tower. These features effectively remove one corner column over the full height of the building and an opposite column at both the top and the bottom of the tower. The structure therefore takes the form of a stack of nine-story-high chevrons. Other architectural features include a cable-suspended skylight canopy roof over a podium, exposed rod-truss curtain wall supports at the winter gardens, and exposed truss-stringer stairs. Structural elements, most notably the nine-story-high super braces on each face of the tower, are boldly expressed in red, while blue and green solar glazing covers the office spaces. The winter gardens and the podium are enclosed in clear glass.

Although the building is expressed as a square, it is punctuated by a series of four nine-story-high wedge-shaped winter gardens cut into the northeast corner and two more at the southwest corner (Figure 9.13a and b). The winter gardens have the effect of dividing the tower, both visually and structurally, into a stack of five modules outlined by the super-braces. At the lowest module, the northeast corner column is eliminated entirely. As a result, the tower has only a single axis of symmetry, which passes at a 45° angle through the corners; in addition, nine different floor plans are required within each module. Plan at a representative floor is shown in Figure 9.13c.

The design of the tower is driven by the unique architectural treatment to the building envelope, by the arrangement of the winter gardens, and by the high wind speeds of up to 115 mph (185 km/h) resulting in design pressures as high as 136 psf (6.5 kPa).



(A) (a) (b) (c)

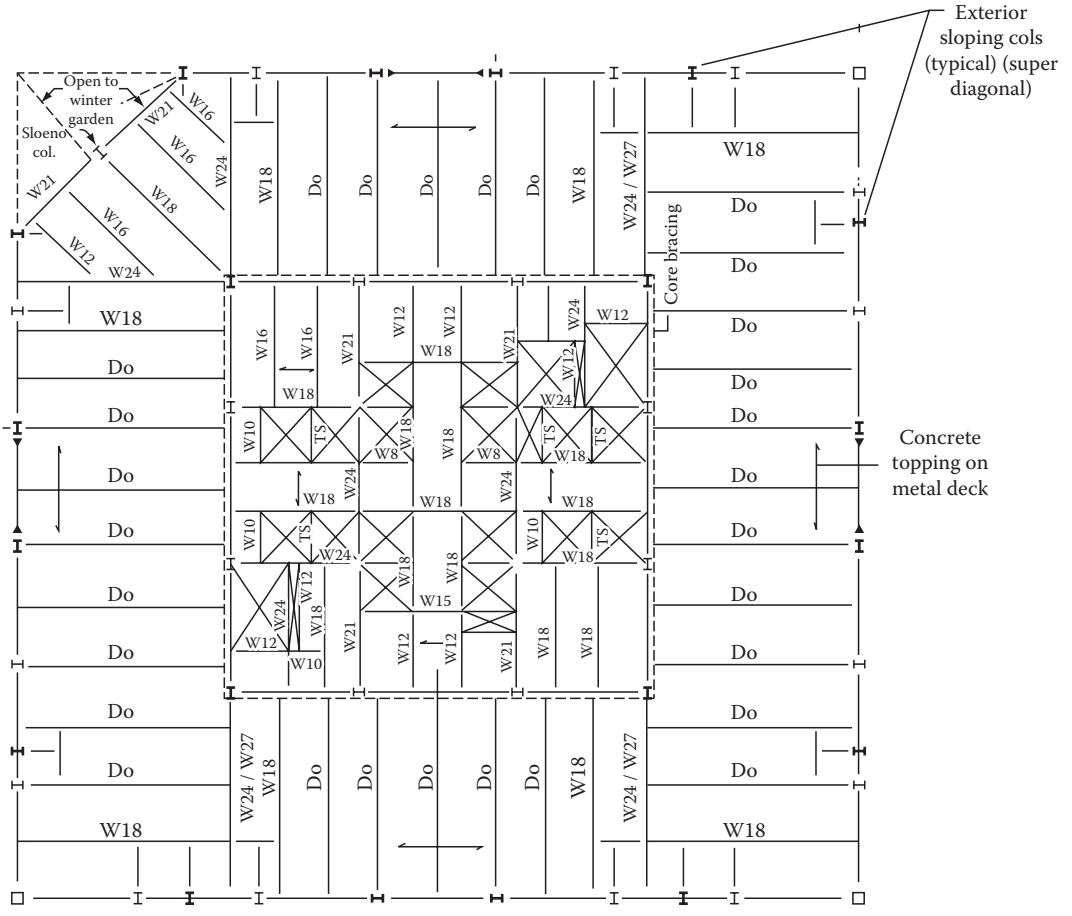


(B)

FIGURE 9.13 21st Century Tower, China: (A) model photographs (a), (b), and (c); (B) bracing system;

The structural solution for the tower consists of a super brace system on the exterior skin supplemented by an eccentrically braced interior service core. The super brace system extends for the full width of the tower to achieve maximum resistance to wind loading. Schematic interior core bracing is shown in Figure 9.13e. Structural action in the primary columns and braces at the base due to lateral loads is shown in Figure 9.13d.

The braces generally consist of heavy W14 sections, field-spliced every three floors and connected at their ends to square steel box columns. The braces also act at inclined columns and carry vertical loads from the secondary columns above them. This arrangement maximizes the vertical load carried by the corner columns and minimizes uplift. Columns vary in size from 20 to 24 in. (520 to 610 mm) with plate thickness up to 5 in. (130 mm). The braces are arranged in five 9-story-high vees with one-bay gap in the middle of each building face; stiffness in the gap is provided by a one-bay-wide rigid frame. Panel points for the super braces occur at the ground, 5th, 15th, 24th, 33rd, 42nd, and roof levels. Horizontal members and diaphragms at these levels are stiffened to transfer horizontal brace forces. Service core bracing provides additional overall stiffness and gives lateral support to floors between the super brace panel points. For architectural reasons, braces at the center bay of each core-face are eccentric (Figure 9.13e). The numerous corner cutouts of the tower structure effectively rotate the principal axes of the structure by 45°, to pass through the corner columns. The lowest two modes of vibration of the tower are single-curvature ending through



(C)

FIGURE 9.13 (continued) (C) framing plan, level 19;

(*continued*)

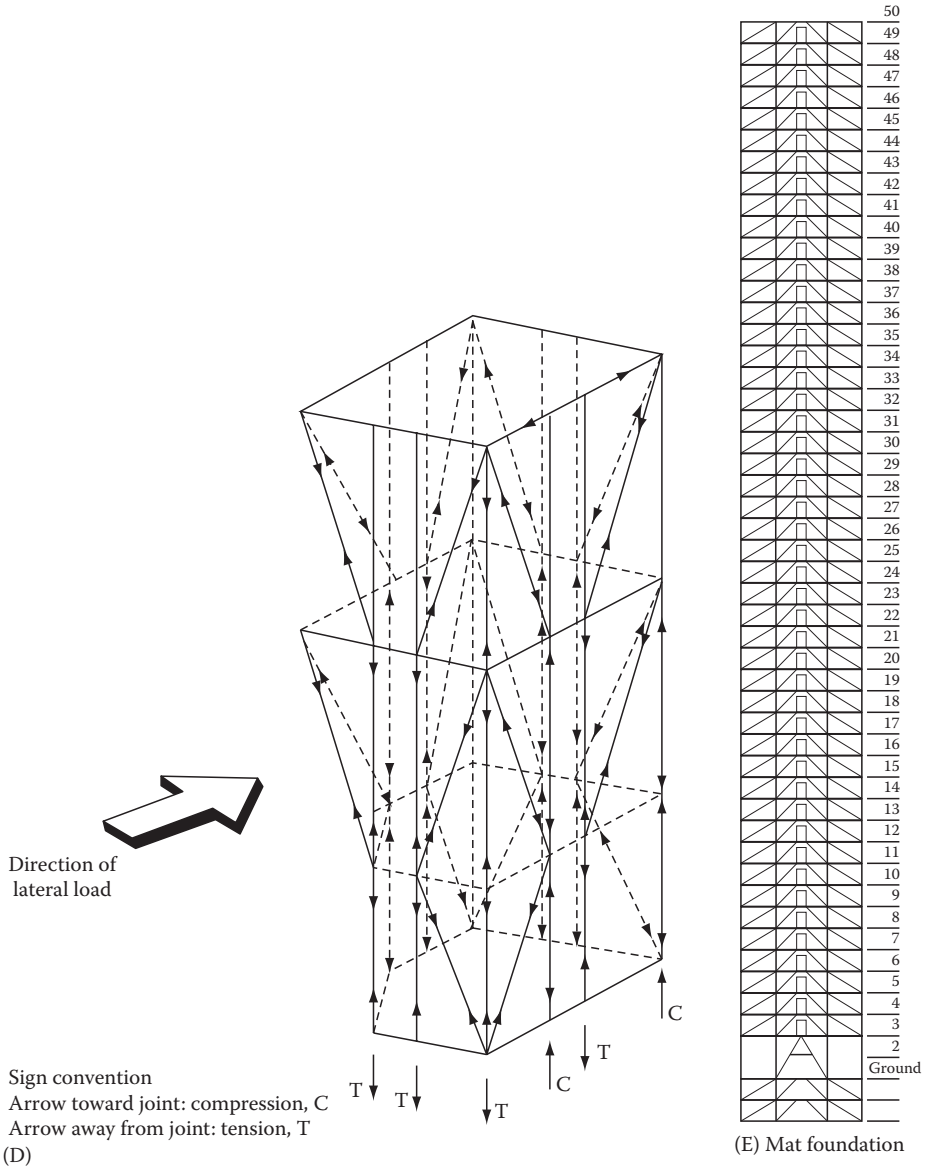


FIGURE 9.13 (continued) (D) structural action in primary columns and braces; (E) typical interior core bracing.

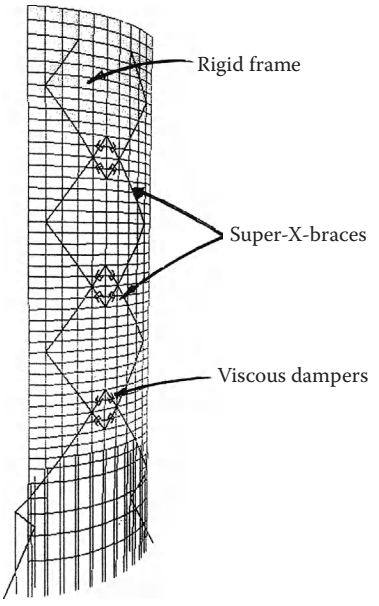
these axes; the third mode is torsional. Periods of the first three models are 4.98, 4.62, and 2.12 s, respectively. The building's average steel weight is 29 psf (142 kg/in.^{2}). The preliminary design is by John A. Martin & Associates and working drawings are by Martin & Huang, International, both of Los Angeles, California.

9.3.14 Torre Mayor Office Building, Mexico City

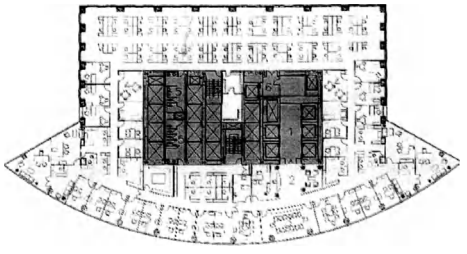
Constructed in one of the most seismically active regions in the world, this 738 ft (225 m) tall building, shown in Figure 9.14a and b, consists of 57 stories including a 13-story parking garage—four stories below ground and nine stories above. It rises from a 262.5 × 262.5 ft (80 × 80 m) footprint at the base, decreasing to 262.5 × 213.3 ft (80 × 65 m) in levels 4 through 10, and to 157.5 × 118 ft



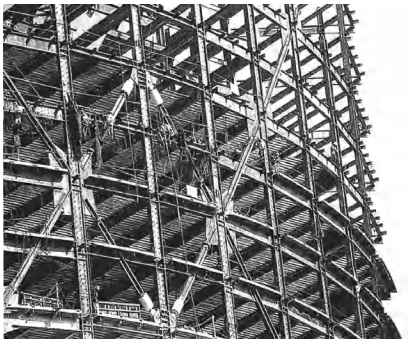
(a)



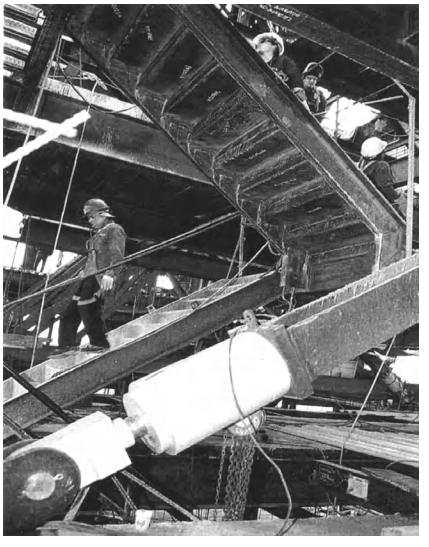
(c)



(b)



(d)



(e)

FIGURE 9.14 Torre Mayor Office Building, Mexico City: (a) building photograph; (b) plan; (c) schematic elevation showing exterior viscous dampers; (d) photograph showing bracing and dampers; (e) close-up view of dampers.

(48 × 36 m) in levels 11 through 53. The rectangular tower is juxtaposed with a curved façade. The tower is constructed on Mexico City's dry central lake bed or "bowl of Jello" consisting of high water tables and poor alluvial soils. The site is prone to very high seismic activity that can measure 8.2 on the Richter scale.

The design performance criterion established for the structure is that it remains operational immediately following a large seismic event. To fulfill this requirement, fluid viscous dampers have been installed in a structural system consisting of a perimeter moment frame interacting with an interior braced frame. A composite super brace frame acting in conjunction with a steel tube is present at the perimeter. A braced core at the building interior completes the lateral resisting system. The perimeter and core columns are encased in reinforced concrete up to the 30th floor. Viscous dampers, 74 of them in the core and 24 the perimeter framing, are installed to absorb and dissipate seismic energy.

Fluid viscous dampers have been installed in diamond-shaped super-diagonal bracing architecturally expressed on the building's perimeter moment frame (Figure 9.14c). All four elevations of the building contain super-diagonals configured as diamonds rather than Xs. Broad south and north faces contain dampers that resist seismic loads in the east-west direction. Each of these elevations has four steel diamonds with 137.8 ft (42 m) legs. The diamonds overlap each other at their peaks and valleys to form three smaller diamonds. Each small diamond has four 1200-kip-capacity dampers, one on each leg near the apex or valley. The building has a total of 98 dampers, 12 on each of the broad faces and 74 in the building core. The incorporation of dampers limits quake-induced damage. Dampers of 600-kip capacity are used in the core in the north-south direction. Core dampers are located conventionally on diagonals of the vertical trusses that transverse the core, two in the end walls and two in between. Photographs of dampers are shown in Figure 9.14d and e.

The columns in floors 1 through 10 are composite with structural steel columns encased in concrete, which limits the size of steel members. Damper clusters begin at the eleventh floor. Rigid floor diaphragms that connect the perimeter frame to a 90.55 × 49.22 ft. (27.6 × 15 m) structural steel core provide in-plane stiffness that ensures all structural elements respond simultaneously to a seismic event.

The building has a four-story basement that extends 49.2 ft (15 m) below ground. The foundation consists of 3.93 ft (1.2 m) diameter caissons that extend 164 ft (50 m) to hard rock below the alluvial deposits. A reinforced concrete mat ranging in thickness from 3.28 to 8.2 ft (1 to 2.5 m) links the caissons. The structural engineering is by Cantor Seinuk Group, New York City, and Enrique Martinez Romero, Mexico City.

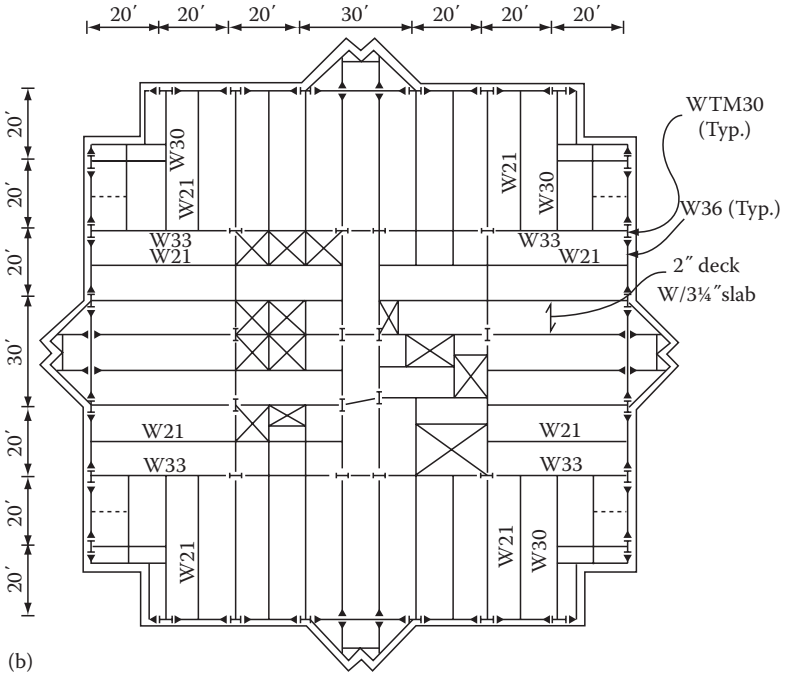
9.3.15 Fox Plaza, Los Angeles, California

The structural system for resisting lateral loads for this 35-story building consists of special moment-resisting frames located at the building perimeter. The floor framing consists of W21 wide-flange composite beams spanning 40 ft (12.2 in.) between the core and the perimeters. A 2 in. (51 mm) deep 18-guage composite metal deck with a 3¼ in. (83 mm) lightweight concrete topping is used for typical floor construction. A framing plan with sizes for typical members and a photograph of the building are shown in Figure 9.15. Architects are Johnson, Fain, and Pereira, Inc; the structural design is by John A. Martin & Associates, Los Angeles.

9.3.16 Figueroa at Wilshire, Los Angeles, California

Floor framing plans at various step-backs and notches for the 53-story tower (Figure 9.16a) are shown in Figure 9.16d. The structural system, designed by CBM Engineers, Inc., Houston, Texas, consists of eight steel super columns at the perimeter interconnected in a crisscross manner to





(a) (b)

FIGURE 9.15 Fox Plaza, Los Angeles, California: (a) building photograph; (b) floor framing plan.

an interior-braced core with moment-connected beams acting as outriggers at each floor (Figure 9.16b and c). The floor framing is structured such that the main columns participating in the lateral loading system are heavily loaded by gravity loads to compensate for the uplift forces due to overturning.

The structural system consists of three major components:

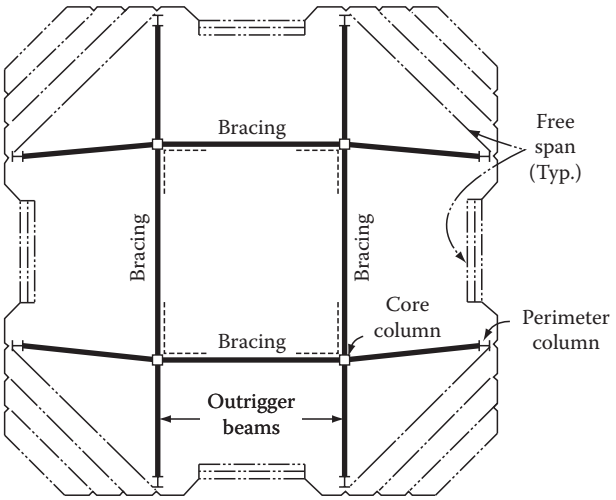
1. Interior concentrically braced core.
2. Outrigger beams spanning approximately 40 ft from the core to the building perimeter. The beams perform three distinct functions. First, they support gravity loads. Second, they act as ductile moment-resisting beams between the core and exterior frame columns. Third, they enhance the overturning resistance of the building by engaging the perimeter columns to the core columns. To reduce the additional floor-to-floor height that might otherwise be required, these beams are notched at the center, and offset into the floor framing, as shown in Figure 9.16e and f, to allow for mechanical duct work.
3. Exterior super columns loaded heavily by gravity loads to counteract the uplift effect of overturning moments.

9.3.17 California Plaza, Los Angeles, California

The project consists of a 52-story office tower rising above a base consisting of lobby and retail levels, and six levels of subterranean parking (Figure 9.17a). A structural steel system consisting of a ductile moment-resisting frame at the perimeter resists the lateral loads. The parking areas outside the tower consist of a cast-in-place concrete system with waffle slab and concrete columns. Figure 9.17b shows a typical mid-rise floor plan for the tower with sizes for typical framing elements. Architects are Arthur Erickson, Inc., and structural design for the building by John A. Martin & Associates, Inc., Los Angeles, California.

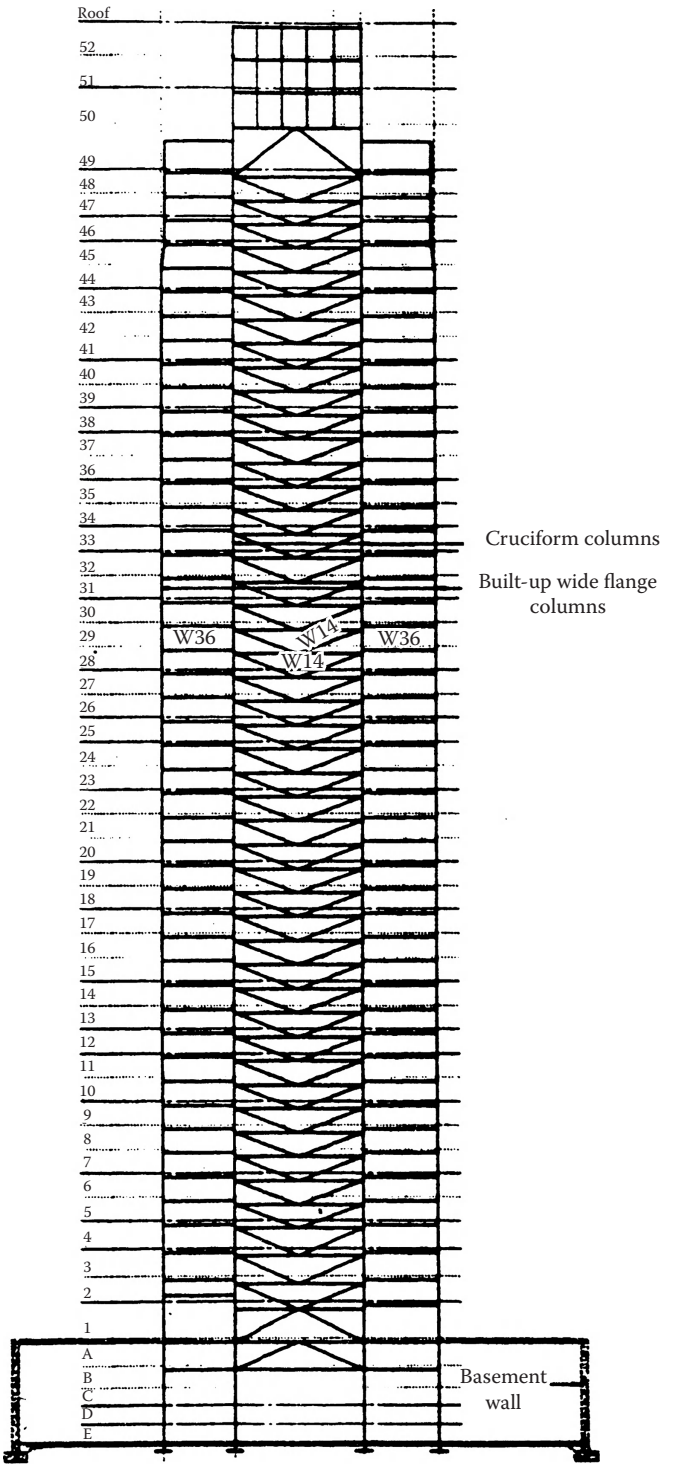


(a)



(b) Spine structure

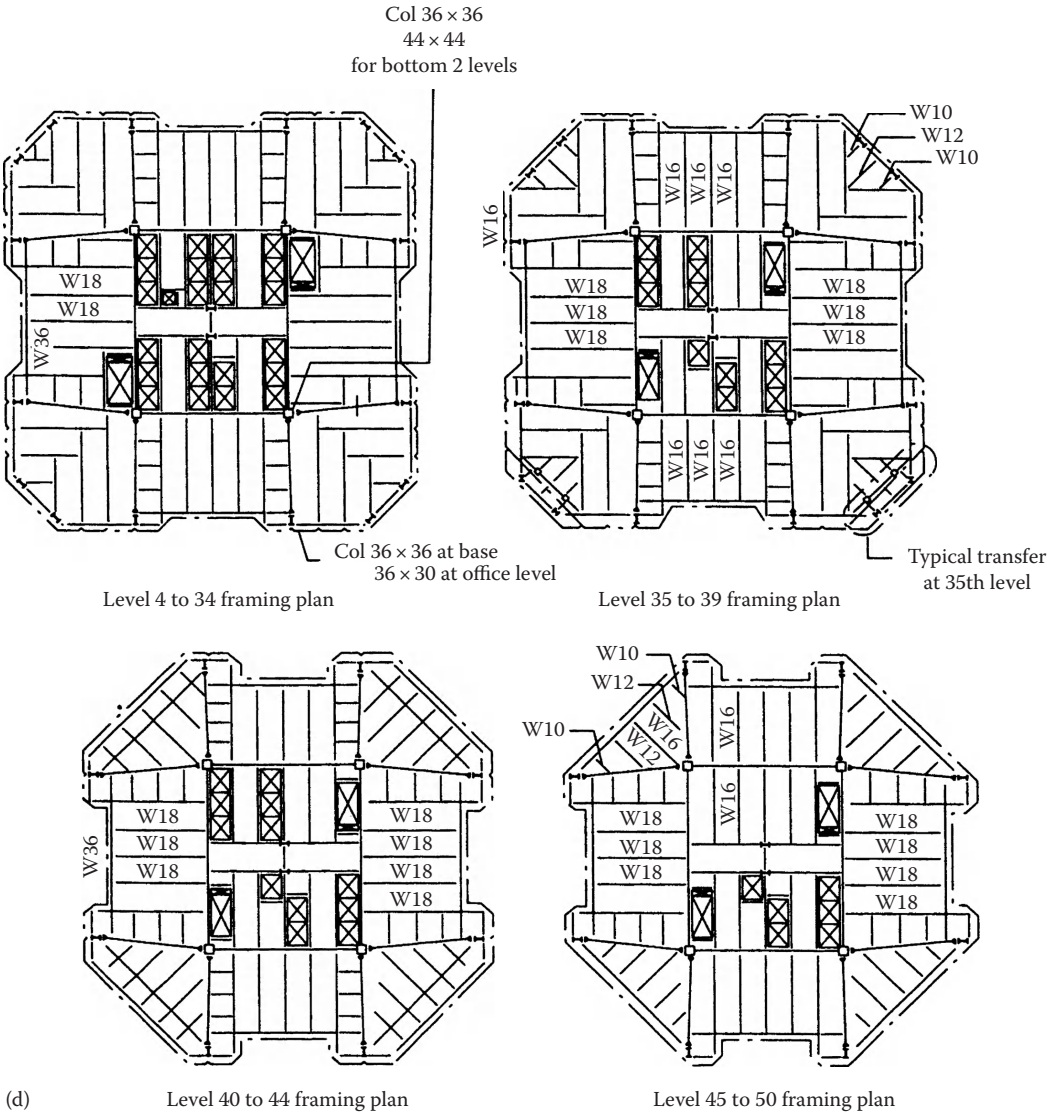
FIGURE 9.16 Figueroa at Wilshire, Los Angeles, California: (a) building elevation; (b) lateral system;



(c)

FIGURE 9.16 (continued) (c) section;

(*continued*)



(d)

FIGURE 9.16 (continued) (d) framing plans;

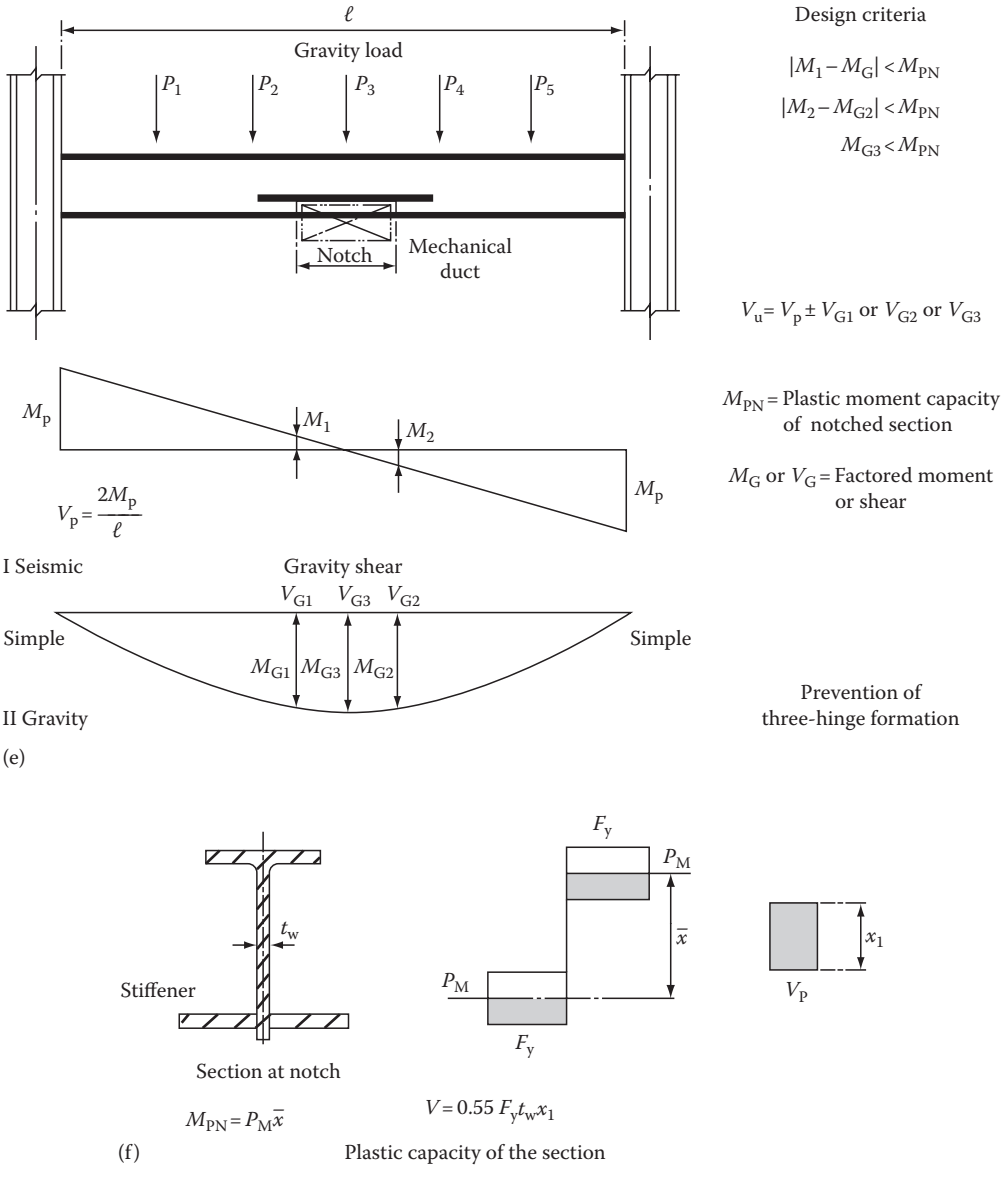
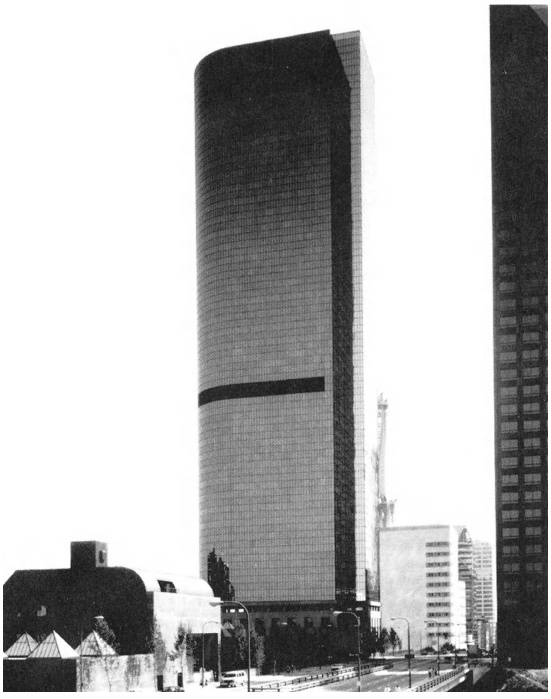


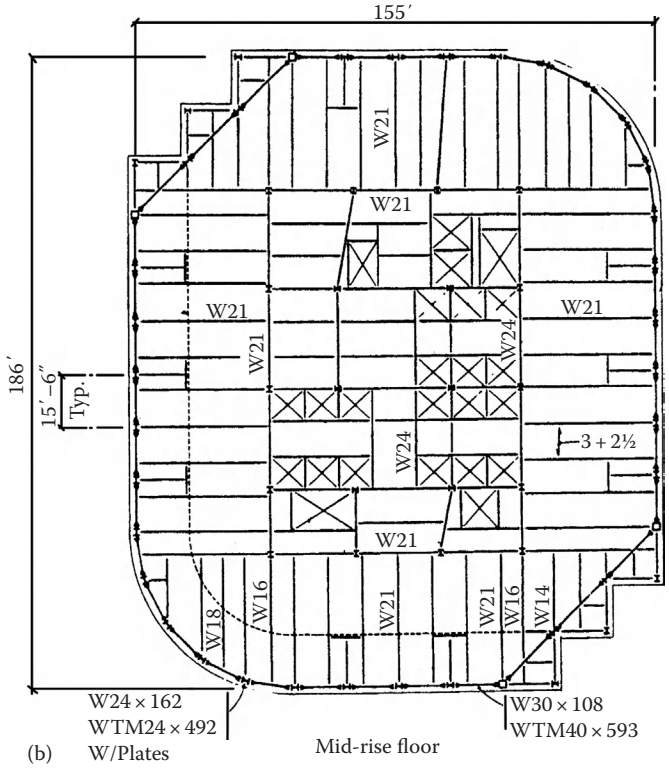
FIGURE 9.16 (continued) (e) design concept; (f) reinforcing at beam notches.

9.3.18 Citicorp Tower, Los Angeles, California

This 54-story tower rises to a height of 720 ft (219.50 m) above ground level and has a height-to-width ratio of 5.88:1 (Figure 9.18a). It has two vertical setbacks of approximately 10 ft (3.05 m) depth at the 36th and 46th floors, as shown in the composite floor plan (Figure 9.18a). As is common to most tall buildings in seismic zone 4, this building was designed for site-specific maximum probable and maximum credible response spectrums, which represent peak accelerations of 0.28*g* and 0.35*g*, respectively. The corresponding critical damping ratios are 5% and 7.5%. The structural system consists of a steel perimeter tube with WTM24 columns spaced at 10 ft (3.05 m) centers and



(a)



(b) Mid-rise floor

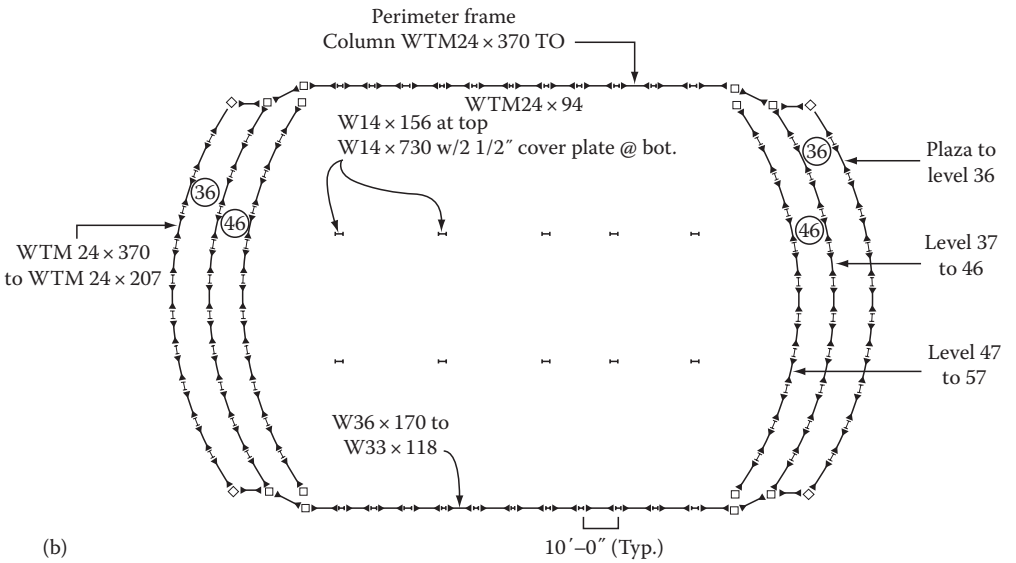
FIGURE 9.17 Cal Plaza, Los Angeles, California; (a) building elevation; (b) mid-rise floor framing plan.

36 in. (0.91 m) deep spandrels. The columns at the setback levels are carried by 48 in. (1.22 m) deep transfer girders and by Vierendeel action of the perimeter frame. Typical floor plans at the setback levels are shown in Figure 9.18c and d.

The foundation for the tower consists of a 7 ft (2.14 m) deep mat below a four-story basement. The structural design is by John A. Martin & Associates, Inc., Los Angeles, California.



(a)



(b)

FIGURE 9.18 Citycorp Tower, Los Angeles, California: (a) building photograph; (b) composite plan; (*continued*)

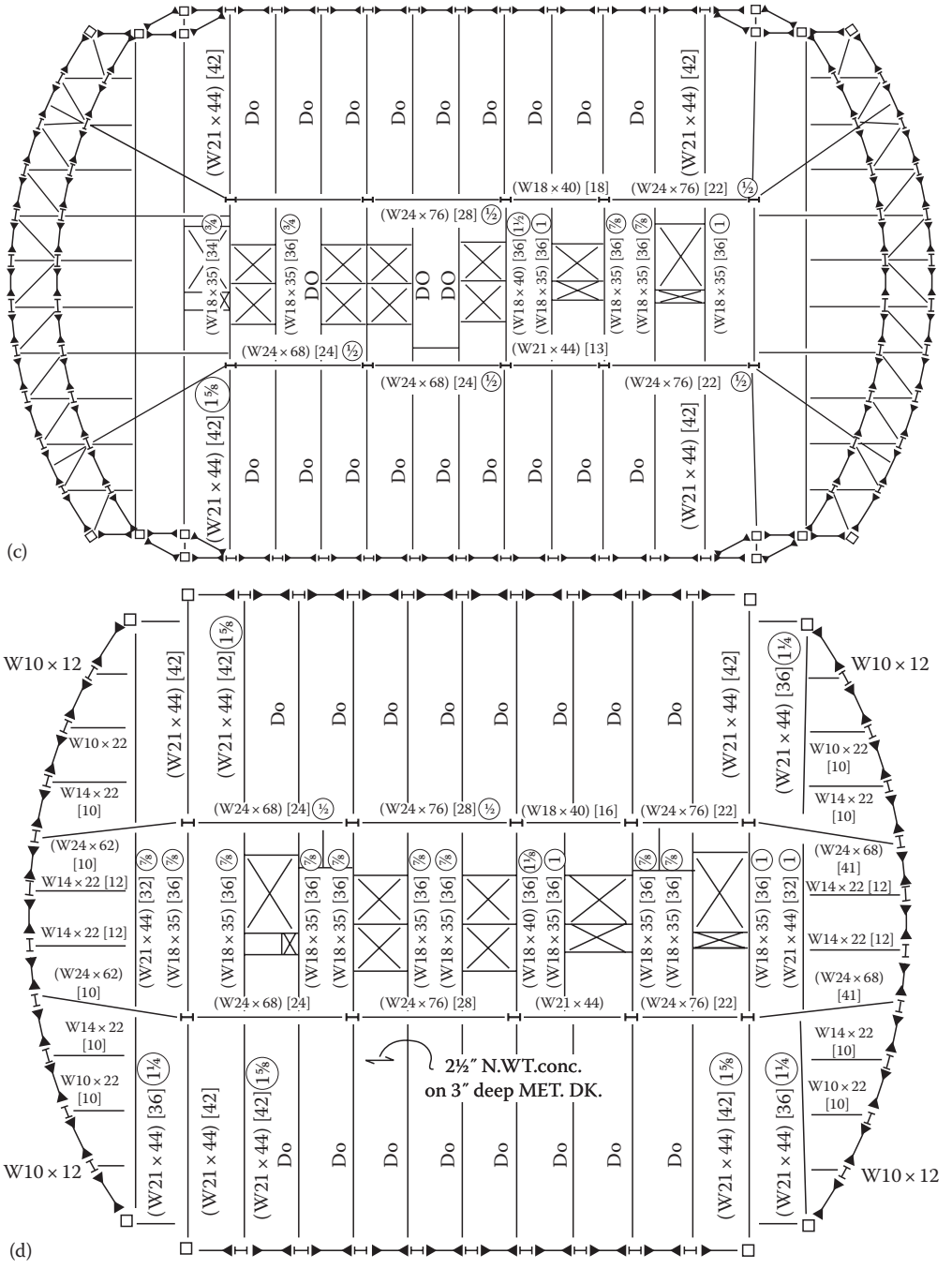
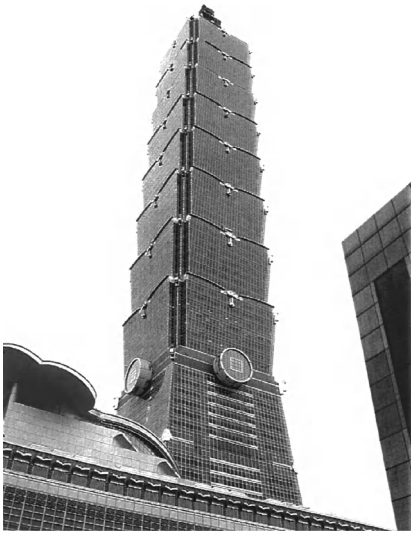


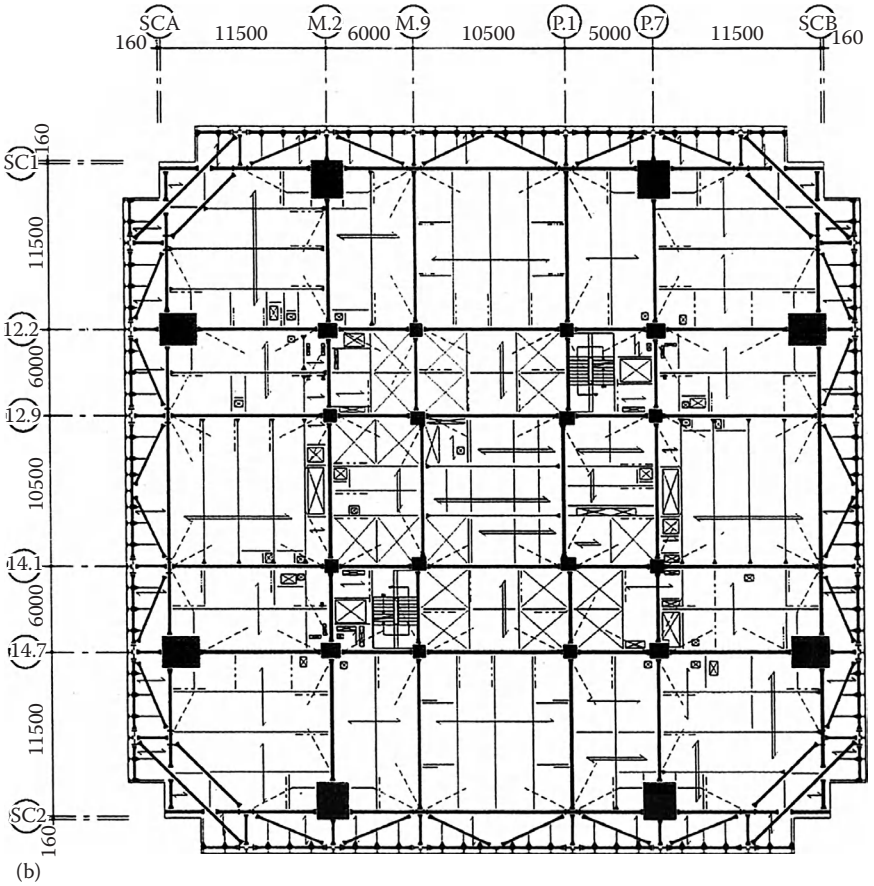
FIGURE 9.18 (continued) (c) 36th floor framing plan; (d) 47th–52nd floor framing plan.

9.3.19 Taipei Financial Center, Taiwan

The 101-story Taipei Financial Center in Taipei, Taiwan, shown in Figure 9.19a rises to a height of 1668 ft (508 m). The lowest 25 floors of the building taper gradually inward, forming a truncated pyramid. Above is a stack of eight 8-story-high modules with outward sloping wall, creating a "waist" at the 26th floor and setbacks at floors 34, 42, etc. (see Figure 9.19b). The modules also have



(a)



(b)

FIGURE 9.19 (a) Taipei Financial Center, photograph. (b) Framing plan level 50. The structure consists of a dual system of braced core connecting to a perimeter sloping frame at each face. The core diagonal and chevron braces are interconnected to vertical super columns via outriggers and belt trusses. The super columns at the base are 2.4 m × 3 m (approximately 8 ft × 10 ft).

(*continued*)

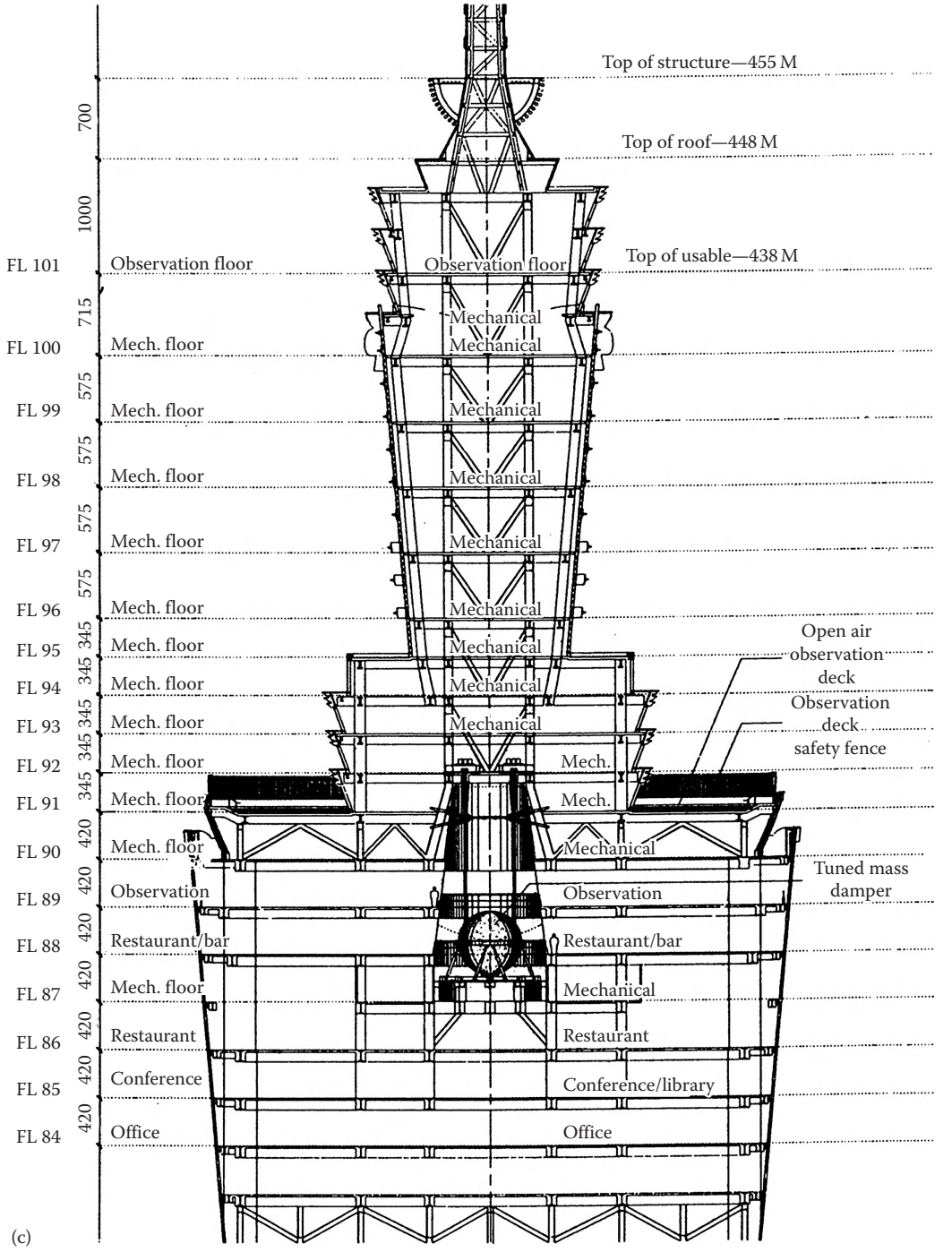


FIGURE 9.19 (continued) (c) Tuned Mass Damper (TMD). The 730 ton TMD, consisting of a steel sphere, is suspended by steel cables from level 92. In addition to the TMD for the tower itself, two additional TMDs are installed for the 197 ft (60 m) spire.

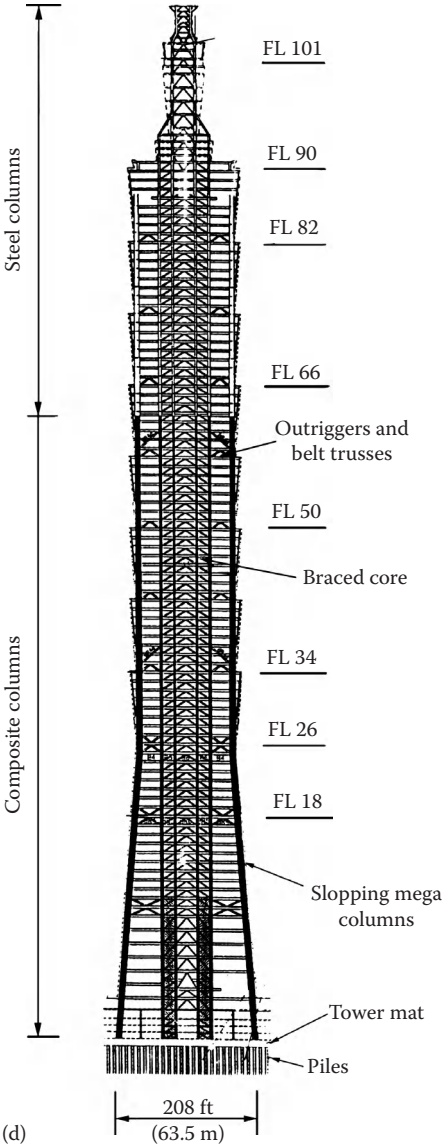


FIGURE 9.19 (continued) (d) Taipei Financial Center, schematic cross section.

double-notched corners. A narrower tower segment and an architectural pinnacle top the eighth module. Because façade slopes and setbacks interrupt vertical continuity of columns, and doubly notched floor plans reduce the efficiency of an exterior moment frame around corners, a perimeter tubular-frame system was not used for this project.

Instead, the lateral bracing consists of a dual system comprising a braced core interconnected to a planar moment frame at each sloping face, through a system of outriggers and belt trusses. The braced core offers high-shear stiffness with chevron and diagonal braces of I-shaped sections in four planes in each direction. A mix of single-, double-, and triple-story outriggers is distributed every 8–10 floors along the building height (see Figure 9.19c). Typically, on each building face, they engage two vertical super columns. Below the 26th floor, additional outriggers engage two more columns on each face with the belt trusses engaging corner columns as well. Steel box-core columns and perimeter super columns are filled with concrete to provide additional stiffness. The size of super columns' steel shell at the base is 8 × 10 ft (2.4 × 3.0 m).

The core is designed as a concentric braced frame (CBF). To clear architecturally required openings, some work points for adjacent braces are spread apart, creating eccentric links. But the system is not designed as an eccentric braced frame (EBF). The design is as for a CBF with the braces, not links, controlling the systems strength. Reduced beam section (RBS) also referred to as dogbone connection is used at locations where the analyses showed plastic rotation demand in excess of 0.005 rad in a 950 year return-period earthquake. For added strength, beam-to-column connections within the braced core system are detailed as moment connections.

Wind engineering studies indicated that accelerations of the building's upper floors would be 30%–40% higher than desired for this office building. Therefore, to improve the structure's ability to dissipate dynamic energy, a passive damping system consisting of a 730 ton tuned mass damper, TMD, has been installed near the top of the tower. The TMD consists of a massive steel sphere suspended by steel cables.

The design of TMD is by Motioneering, Inc., Ontario, Canada. The building structural design is by Evergreen Consulting Engineering, Taipei, and Thornton-Tomasetti Engineers, New York City.

9.3.20 Caja Madrid Tower, Spain*

Caja Madrid Tower, at 820 ft tall consists of five below-grade parking levels and 34 office floors that are divided into three distinct office blocks of 11, 12, and 11 floors. Below each office block is a two-story space dedicated to mechanical equipment that serves the floors above. Within the spaces are two pairs of perpendicular two-story steel trusses, one pair spanning between two end cores and supporting a transverse pair that aligns with the steel columns above. This truss system transfers all the gravity loads to the reinforced concrete cores at the building ends, which are the only vertical-load-carrying elements that extend to the foundation. This results in channeling of sufficient gravity loads on the concrete cores to counter tensions due to overturning resulting from lateral loads. Additionally, a much sought after column-free lobby exists at the ground floor.

The typical office floor plate, about 14,500 ft^2, is framed in structural steel allowing the four corners of the floor to cantilever approximately 27½ ft from the core and 23 ft from the nearest exterior column. This is accomplished by the use of a steel Vierendeel frame that wraps the perimeter of the building.

The typical office floors have a 3 in. composite deck with 3 in. of light weight concrete topping. They are supported at each level by the two reinforced concrete cores and by four interior and four exterior W14 steel columns. The floor slabs corresponding to the top and bottom chords of the two-story truss system at the transfer levels, are 3 in. composite deck with 6 in. of normal weight concrete topping. The thicker slabs are used primarily to minimize sound transmission from the mechanical equipment located at the transfer levels.

The north-south lateral loads are resisted by the cantilever action of the two cores. Because the gravity load for the entire building is supported only by the cores, this is no net tension or uplift in the cores even though their aspect ratio is quite large at 11 to 1. For east-west lateral loads, the cores are too narrow to provide adequate strength and stiffness and, therefore, are linked together at the three transverse levels by trusses. Thus, the end cores and truss system behave like a giant moment frame to resist lateral loads.

At each of the three transfer levels, the truss system consists of the following: two "primary" trusses that span 105 ft between the cores and two "secondary" trusses that cantilever 33 ft to the north and south past the primary trusses. These secondary trusses transfer the four exterior columns back to the primary trusses, which in turn, along with the four interior columns are eventually transferred to the cores.

Because the primary trusses are an integral part of lateral-load-resisting giant frame in the east-west direction, the truss chords are rigidly connected to the cores. However, to minimize the gravity load negative moments at the ends, bolts in these connections were pretensioned only after the dead load of the office blocks had been applied to the truss.

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* The author would like to acknowledge his gratitude to Bob Halvorson, founder of Halvorson and Partners, Chicago, for providing the illustrations for the project.

To minimize the thrust against the cores resulting from elongation of trusses bottom chords due to gravity load, post-tensioning tendons are provided along the bottom chord of the primary truss and anchored to an embedded steel column in the core.

At each level within the core, where the truss top and bottom chords attach, a 6.2 ft. thick slab with two-way system of post-tensioning is provided to allow for a positive means of engaging the entire cross section of the core to resist the truss chord forces. Schematic plans and elevations are shown in Figure 9.20a and b.

Halvorson and Partners of Chicago are the structural engineers for the project.

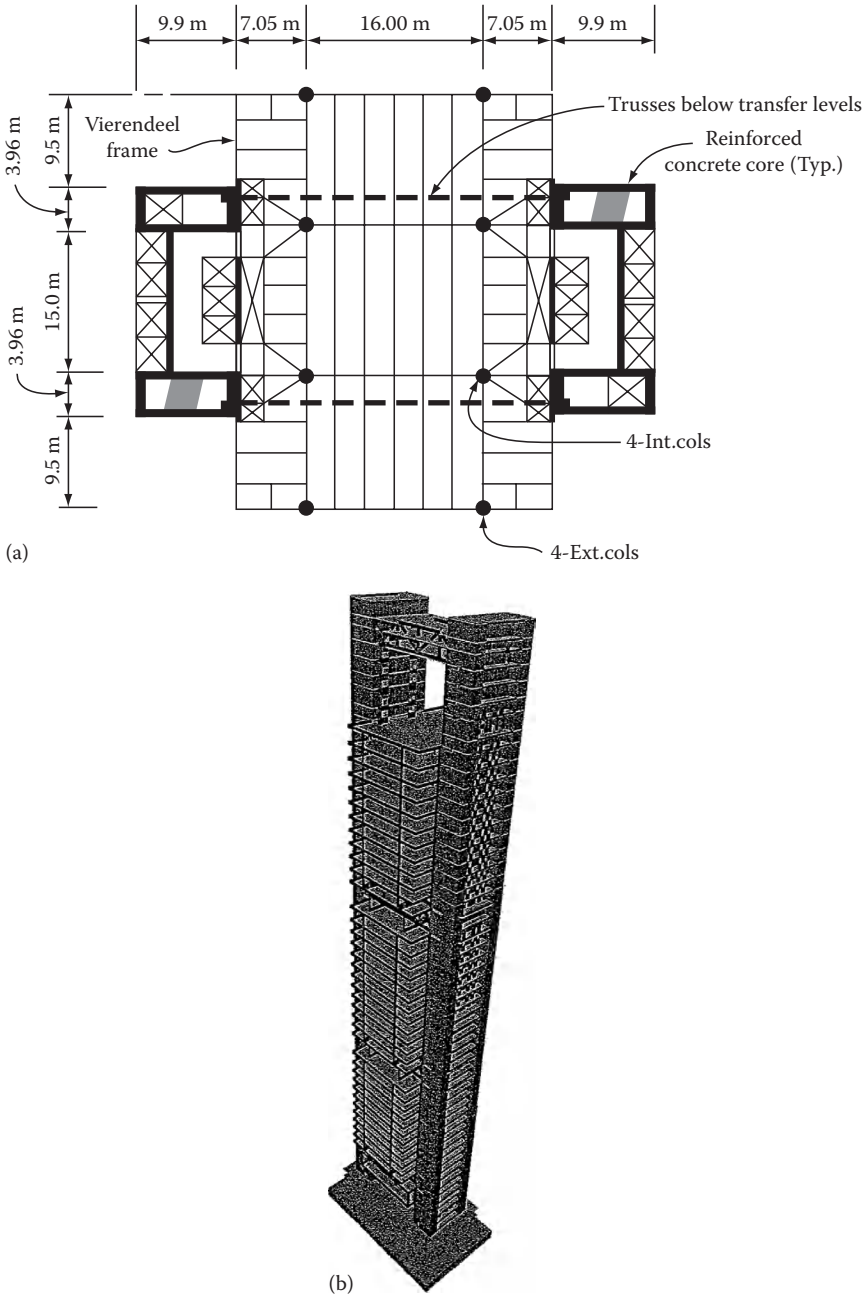


FIGURE 9.20 Caja Madrid, Spain; (a) typical floor framing plan; (b) structural schematics.

9.3.21 Federation Tower, Moscow, Russia Tower A

This is the tallest building in Europe at 1181 ft, with its architectural spire reaching 1.470 ft. The 93-story, Tower A is mainly of concrete construction, but has major steel framing at multiple areas of the tower, thus earning its place as one of the tallest buildings using composite construction.

Structural steel outrigger and belt truss systems are placed at one-third and two-thirds height of the tower (levels 61 and 33). The complex geometry of the tower with perimeter angle changes made these areas more conducive to steel framing. The sixty-first floor truss system is approximately 43 ft deep and doubles as a transfer truss for half of the perimeter columns. Steel sections comprising of W14 wide flanges and built-up shapes from 5 in. thick plates, 30–34 in. wide were used. The steel trusses were embedded in the concrete core walls, and the columns within height of trusses were extended one floor up and one floor down into the core walls. Stubbing of steel columns up and down allows for a smooth transmission of loads between the steel and concrete members. Structural engineering for the project is by Thornton Tomasetti, New York.

9.3.22 The New York Times Building, New York

Located a few blocks away from the Times Square, this 54-story office building stands 744 ft from the sidewalk to the roof. The façade extends above the roof to hide roof top mechanical equipment. A 300 ft high steel mast extends above the roof, topping the overall structure out at 1048 ft.

The footprint of the building resembles an elongated cruciform shape in plan.

The lateral-load-resisting system consists of a steel-braced frame core, 65 ft in the east-west direction by 90 ft in the north-south direction. At the mid-level and upper level mechanical floors, double-story diagonal braces extend from the core to the perimeter columns engaging the perimeter columns to participate in the lateral load-resistance of the building.

The cruciform shape results in four notches of the tower. The two columns on the north and south notch faces are placed outside the building envelope and consist of 30 in. by 30 in. exposed steel columns built from flange plates varying from 4 in. at the base of the base of the building to 2 in. at the top. The thickness of web plates also vary and are inset 3 in. from the toes of column flanges to crate as aesthetic reveal.

In addition to the core bracing and the two-story outriggers, the building also has exposed bracing at the exterior consisting of a two-story-high X-braced system. The bracing consists of pretensioned high-strength (64 ksi) steel rods 4 in. in diameter at the base and 2.5 in. at the upper levels. The rods are pretensioned to overcome any compression due to differential axial shortening of columns due to gravity load imbalance, differential temperatures in exposed steel members, and wind loads. Because the rods themselves are not fire proofed (to keep the exposed bracing light and elegant), their participation in resting the lateral loads is considered only in evaluating the serviceability criteria for the building that typically consists of limiting the accelerations of the building to levels deemed acceptable for human comfort (typically 20 milli-g for office buildings for a 10-year return period wind).

An interesting feature of the exposed braces is that in lieu of single diagonals intersecting at a center note, two pairs of steel rods are used, one set aligned vertically and the other horizontally. This arrangement allows the diagonals to bypass each other without requiring a common node at the intersection.

One of the other structural features is the cantilevered floor systems that form the north and south stems of cruciform in plan. At the east and west faces of the bay, the structure is supported by built-up cantilevered beams tapering 22 in. deep at the column to 18 in. at the tip. The beams are welded directly into the box columns without back-spans as is typically done in cantilever construction. However, a single 2.25 in. diameter diagonal rod is used to control the deflection of the cantilevered beam. The beam is sized from strength consideration only and, therefore, not fire proofed similar to the exposed diagonal rod bracing, which serve only serviceability role.

Like other sites in midtown Manhattan, the building site contains bedrock close to the level of surrounding sidewalk. A majority of tower columns, 27 to 30, are founded on rock that has an allowable bearing capacity of 40 ton/ft^2. The other seven tower columns at the southeastern corner of the site are supported on drilled caissons consisting of 24 in. diameter steel casings filled with 6000 psi concrete to achieve a capacity of 2.400 kip. The reason for using the caissons was the rock at the southeastern corner had an allowable bearing capacity of only 8 ton/ft^2. The structural engineering for the building is by Thornton Tomasetti, Inc., of New York City.

9.3.23 Pacific First Center, Seattle, Washington

This is a 44-story building of composite construction with eight 7.5 ft (2.3 m) diameter pipe columns in the core and sixteen 2.5 ft (0.76 m) diameter columns at the perimeter. All pipe columns are filled with 19 ksi (131 MPa) concrete.

A schematic plan is shown in Figure 9.21.

9.3.24 Gate Way Center

This is a 62-story office building with 9 ft (2.7 m) diameter columns filled with 19 ksi concrete and tied together to form 10-story-high X-braces.

A schematic plan is shown in Figure 9.22.

9.3.25 Two Union Square, Seattle, Washington

This 58-story office building (see Figure 9.23 for a schematic plan) is another example of composite construction that uses large diameter pipe columns filled with concrete. The columns are 10 ft (3 m) diameter pipe columns filled with concrete. To achieve composite action, steel studs

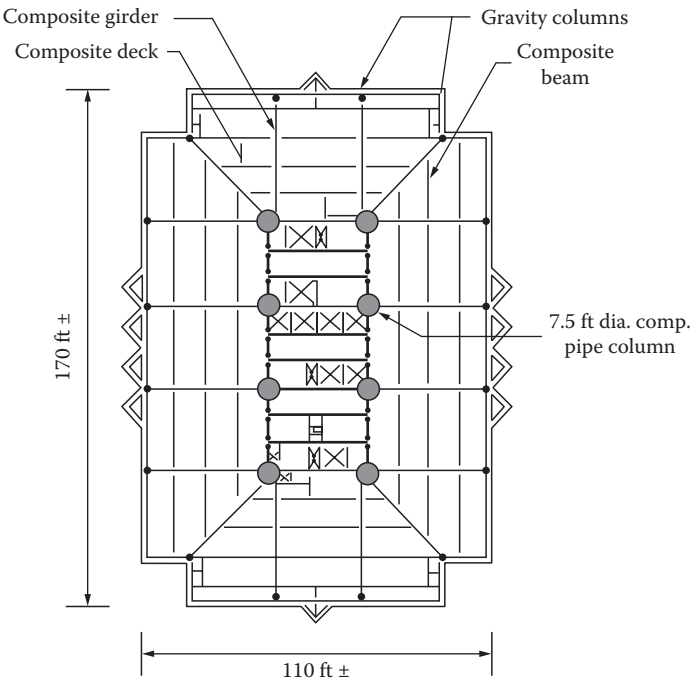


FIGURE 9.21 Pacific First Center; 44-story building.

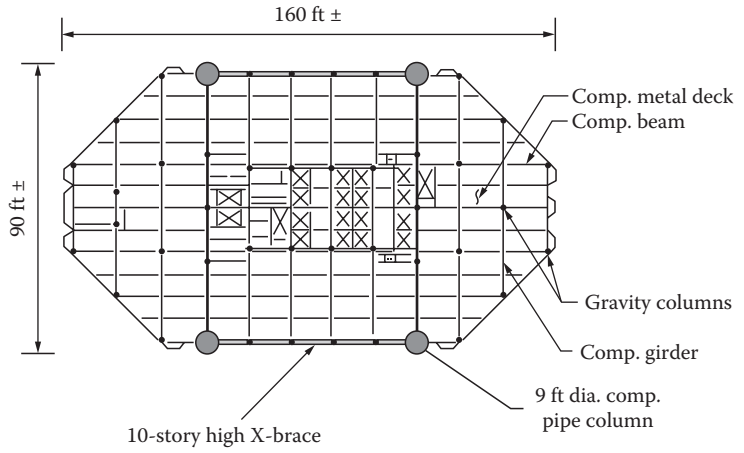


FIGURE 9.22 Gateway Center; 62-story building.

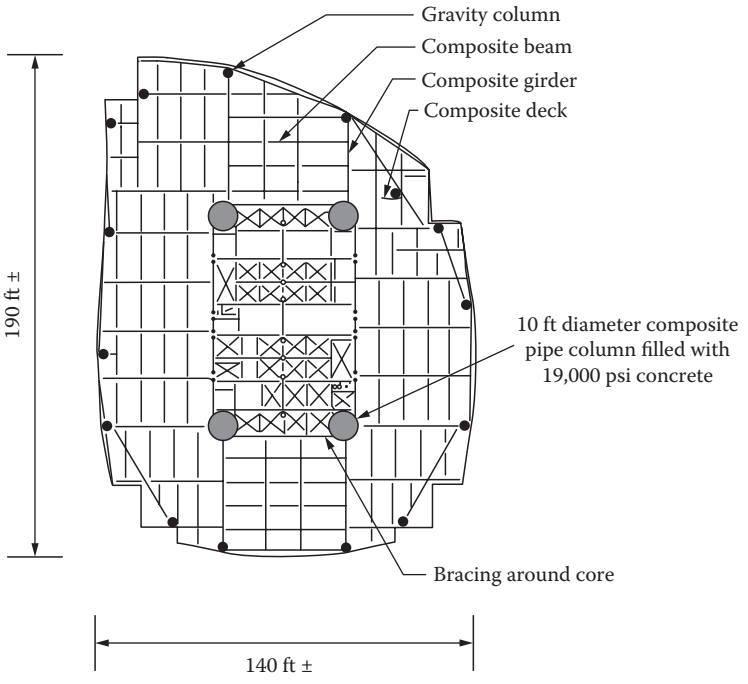


FIGURE 9.23 Two Union Square; 58-story building.

are welded to the interior surface of pipe columns, a method that was also used for the other two buildings described earlier.

9.3.26 InterFirst Plaza, Dallas, Texas

An example of a building that uses formed composite super columns is shown in Figure 9.24. In this 73-story, 921 ft (281 m) tall building, the entire weight of the building is supported on 16 composite columns located up to 20 ft (6 m) inboard from the building exterior. The lateral loads are resisted by

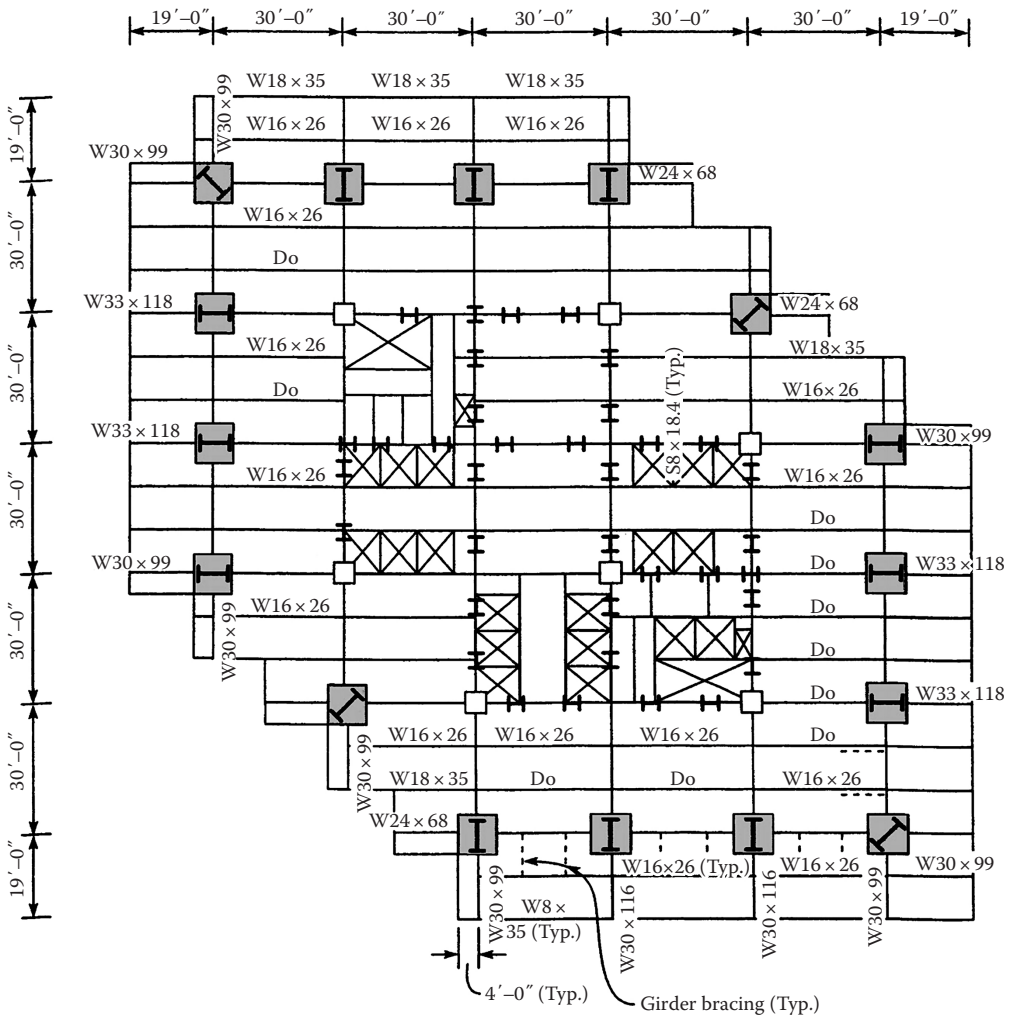


FIGURE 9.24 Dallas Main Center; Interfirst Plaza, Dallas Texas; 26th–43rd floor framing plan.

the composite columns interconnected to a system of 7-story two-way Vierendeel trusses. The composite columns vary in size from 7 ft × 7 ft (2.1 m × 2.1 m) at the base to 5 ft × 5 ft (1.5 m × 1.5 m) at the top; 36 in. (0.30 m) deep steel shapes are encased in 10 ksi (69 MPa) concrete columns, reinforced with 75 ksi (517 MPa) mild steel reinforcement.

9.3.27 Bank of China Tower, Hong Kong

This prism-shaped building (shown in Figure 9.25a and b) designed by the architectural firm of I. M. Pei and Partners and structural engineer Leslie E. Robertson, is a 76-story building consisting of four quadrants. Each of the quadrants rises to a different height, and only one reaches the full 76 stories. The lateral bracing consists of a space truss spanning between the four corner columns. From the top quadrant down, the gravity loads are systematically transferred out to the building corner columns by truss action. Transverse trusses wrap around the building at selected levels. At the 25th floor, the center column is transferred to the corners by the space truss, providing for an uninterrupted 158 ft (48 m) clear span at the lobby. At the fourth floor,

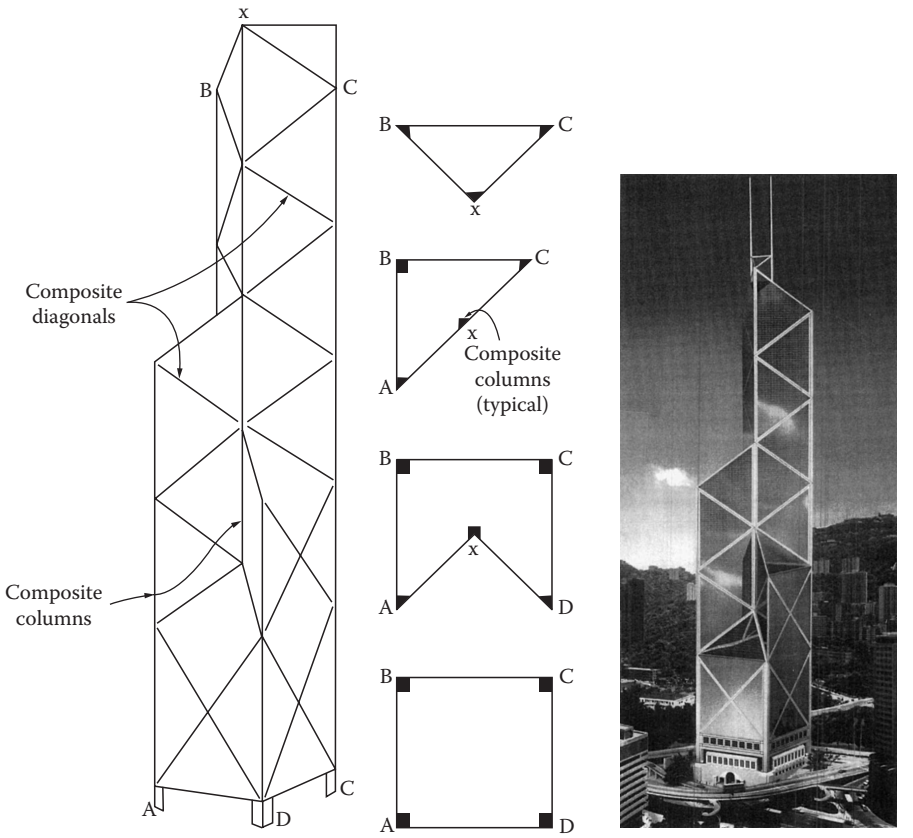


FIGURE 9.25 Bank of China, Hong Kong.

the horizontal shear forces are transferred from the space truss to the interior composite core walls through ½ in. (12 mm) thick steel plate diaphragms acting compositely with the floor slab. The foundation for the building consists of caissons as large as 30 ft (9.1 m) in diameter hand-dug to bedrock.

The Bank of China Tower is one of the most recognized skyscrapers in central Hong Kong. The building is 315 m tall with two masts reaching 1209 ft (36.9 m) high. The structural expression adapted in the design of the building resembles growing bamboo shoots, symbolizing lively hood and prosperity. The entire structure is supported by four columns at the corner of the building, with the triangular frameworks transferring the weight of the structure to these columns.

9.3.28 Bank of Southwest Tower, Houston, Texas

The Bank of Southwest Tower, an 82-story, 1220 ft (372 m) building proposed for, and not built in, downtown Houston, Texas, uses the unique concept of composite columns with interior steel diagonal bracings. The diagonals transfer both the gravity and the lateral loads into eight composite super columns. The building has a base of only 165 ft (50.32 in.), giving it a height-to-width ratio of 7 to 4. The characteristic feature of the design consists of a system of internal braces that extend through the service core and span the entire width of the building in two directions.

A typical bracing consists of an inverted K-type brace rising for nine floors, there being two such braces in each direction. Eight of these 9-story trusses are placed one on top of another within the tower. All of the gravity loads are transferred to eight composite columns located at the building perimeter. The structural engineering is by LeMessurier Consultants, Inc., and Walter P. Moore & Associates, Inc.

See Figure 9.26a–d for building schematics and photograph of the building model.

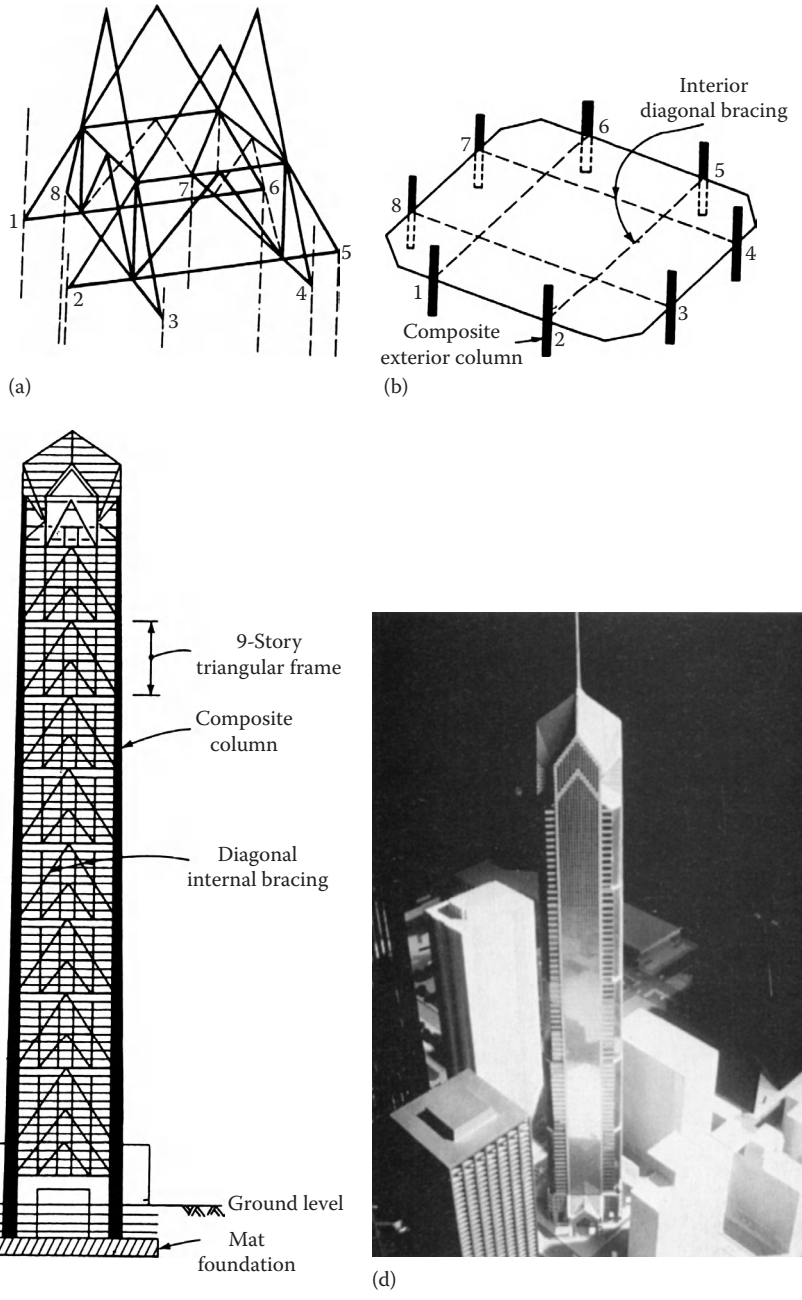


FIGURE 9.26 Bank of Southwest Tower, Houston, Texas: (a) schematics of interior diagonal bracing; (b) plan; (c) section; (d) photograph of model.

9.3.29 First City Tower, Houston, Texas

Designed by structural engineers Walter P. Moore & Associates, Inc., this 49-story tower comprises a number of distinctly different composite elements (Figure 9.27A–D). Shown in Figure 9.27B are the details of the composite columns. Typically, the embedded steel columns vary from a W14 × 370 at the bottom to a W14 × 68 at the top. The vertical reinforcement in the columns varies from #18 bars (57 mm) at the bottom to #7 bars (22 mm) at the top. Open ties permitted in low seismic zones are used throughout.

Figure 9.27C shows the arrangement of reinforcement around a W10 × 72 erection column embedded in the shear walls. Ties are used where the vertical reinforcement ratio is more than 1% or where the reinforcement is required for resisting compression. This requirement has been in the ACI 318 code for the past 25 years. See Section 14.3.6 of ACI 318-02.

Figure 9.27D shows the connection detail between the concrete shear wall and a typical steel link beam. The moment capacity of the beam is developed through a shear transfer mechanism by steel studs welded to the top and bottom flanges of the beam. The stiffener plate, set flush with the wall face, has no structural purpose but helps in simplifying the form work around the beam. The construction sequence generally used in a composite section is shown in Figure 9.27C.

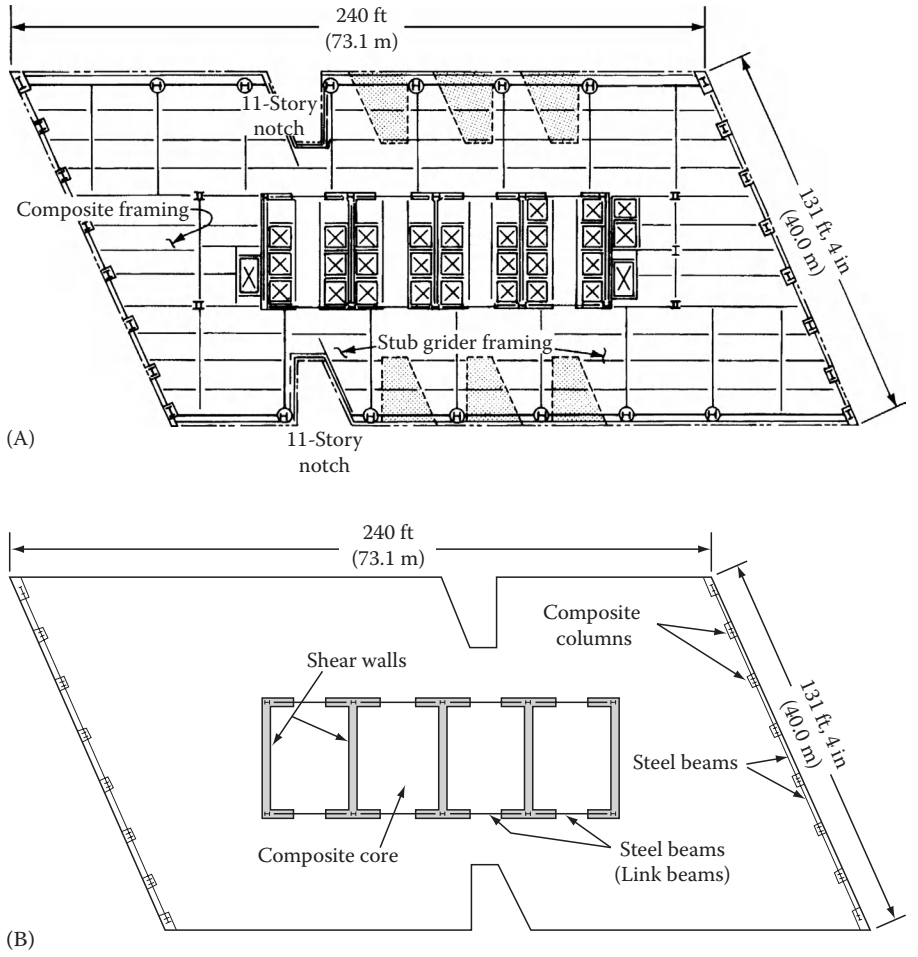
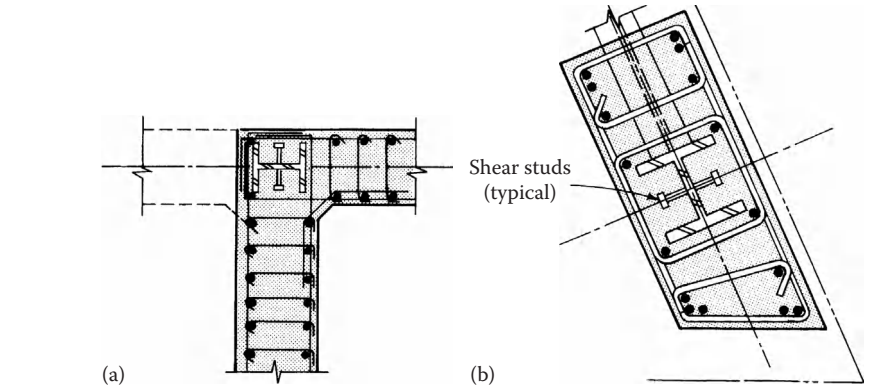
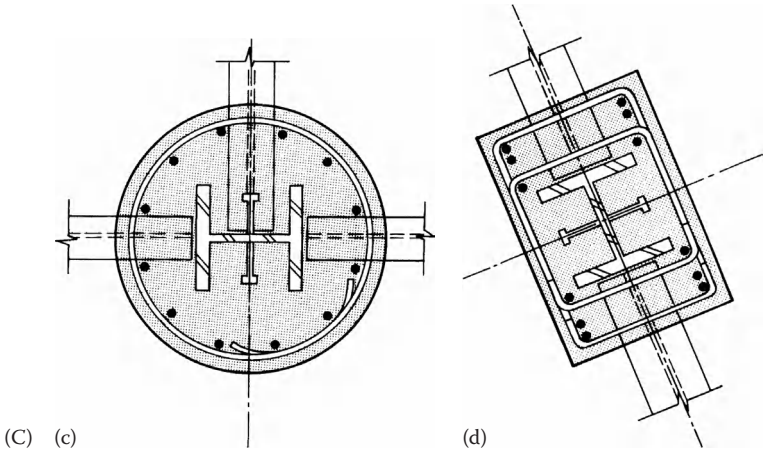


FIGURE 9.27 (A) First City Tower, Composite floor framing plan. (B) Composite elements.





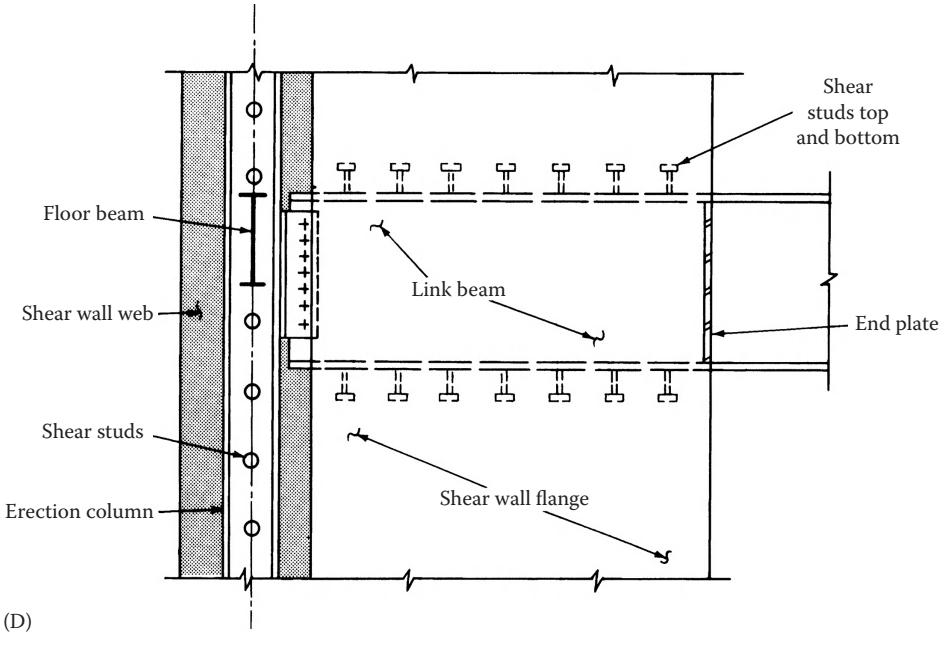


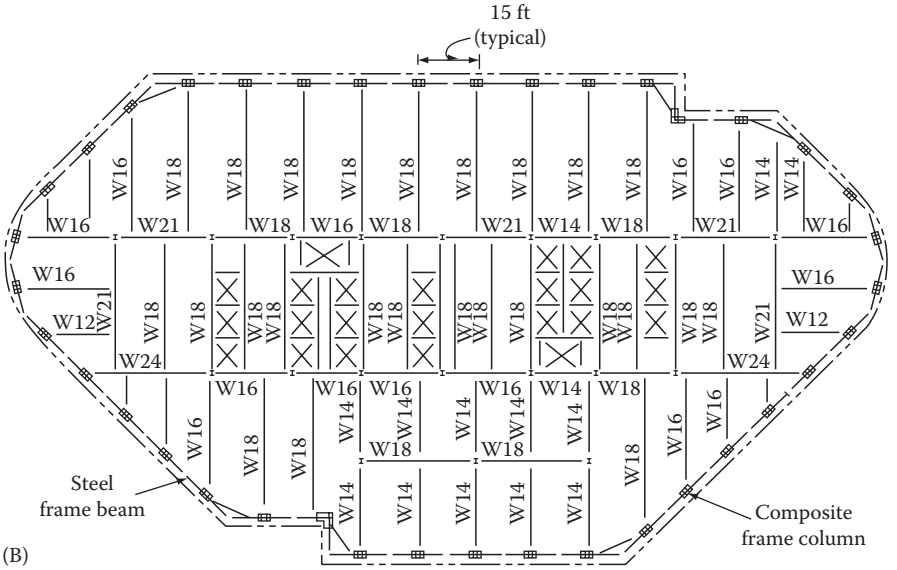
FIGURE 9.27 (continued) (C) Composite vertical elements: (a) composite shear wall; (b) composite corner column; (c) typical circular composite column on long faces; (d) typical exterior composite column on short faces. (D) Steel link beam-to-shear wall connection.

9.3.30 America Tower, Houston, Texas

Shown in Figure 9.28A and B are the details for a 42-story office building called America Tower, designed by structural engineers Walter P. Moore and Associates, Houston, Texas. This building uses a hybrid tubular perimeter frame consisting of composite columns above the third floor and structural steel columns below (see Figure 9.28C). This integration of steel columns and composite columns eliminated form work for columns at nontypical lower levels.

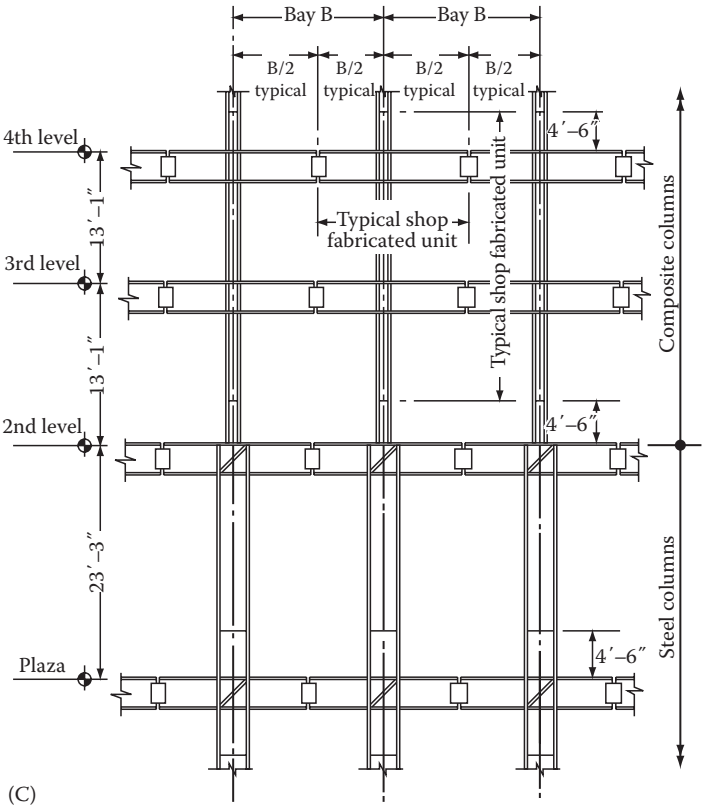


(A)



(B)

FIGURE 9.28 (A) America Tower, Houston, Texas, a 42-story composite tube system. (B) Floor framing plan.



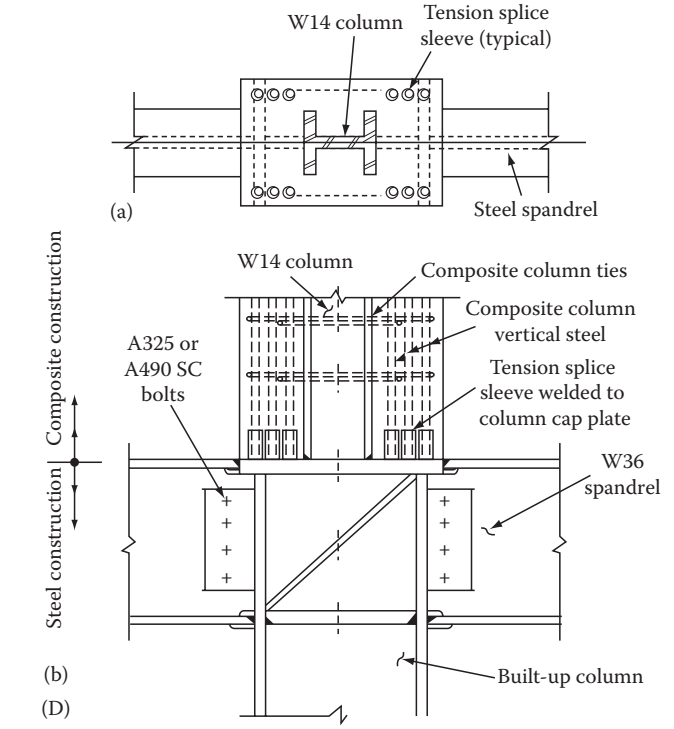


FIGURE 9.28 (continued) (C) America Tower, composite column-to-steel column connection. (D) Composite column-to-steel column connection detail: (a) plan; (b) elevation.

9.3.31 The Bow Tower, Calgary, Alberta, Canada

Scheduled for completion in 2011, the 59-story, 780 ft (238 m) tall Bow Tower consists of a perimeter diagonal gird structure coupled with interconnecting moment frames acting as primary lateral load system. The tower foot print measures approximately 320 ft (97.5 m) by 190 ft (57.5 m) and will include a gross area of over 2.1 million square feet above grade and 1 million square feet, six-level below grade construction for parking spaces, loading docks, and other service areas.

The five parking levels below grade as well as the roadway consist of a post-tensioned, precast hybrid system. The post-tensioned beams span 60 ft (18 m) across the roadway and over the loading dock while precast planks spanning 28 ft (8.5 m) are used between the post-tensioned beams.

To facilitate the erection of the steel tower ahead of the completion of the parking slabs below grade, a *top-down* construction technique is used. This method allows for construction of the super structure and substructure simultaneously.

The majority of the tower columns below the ground level consist of large diameter steel pipes filled with concrete. These are designed for an unsupported length of 60 ft (18 m) with a maximum diameter of 48 in. (1.2 m).

The foundation for the tower consists of a 10 ft (3 m) thick reinforced concrete mat. The entire 18,300 yd^3 (14,000 m^3) of 7,250 psi (50 MPa) concrete for the mat was placed in a single pour. The structural design for the project is by Halcrow Yolles, Inc. Toronto, Canada.

9.3.32 Shard Tower, London, United Kingdom

When completed in 2012, the Shard Tower will rise to a height of 1017 ft (310 m). It is designed to accommodate retail and office usages at the lower levels, a hotel at mid-height levels, and residences at the high-rise levels of the tower. A tapered geometry along the height is used to efficiently accommodate the various usages.

Key features of the structural design include use of *top-down* construction for the tower. Top–down construction is a method of construction where, the basement floors are constructed as the excavation progresses from grade level down to the lowest basement level. Typically in this type of construction, structural steel members are embedded in freshly poured concrete piles installed from the grade level. The steel columns are designed to work as structural columns for the tower. Shard Tower has a three-level basement designed as reinforced concrete flat slab with drop panels.

The floor system consists of a combination of composite steel beams and post-tensioned concrete slabs. At the lower retail and office levels above ground, composite steel framing with spans up to 50 ft (15 m) are used. This system occurs in levels 3–39. At the upper hotel and residential levels, levels 40–72, 8 in. (200 mm) thick post-tensioned slabs are designed for spans up to 30 ft (9 m).

Typical composite floor beams at the office levels are 20 in. (200 mm) deep built-up members. The relatively shallow built-up steel beams are used to minimize the ceiling depth. The depth of spandrel beams is also minimized to allow for maximum daylight penetration into the building interior.

The first four floors of the hotel will consist of a steel floor system with a Vierendeel truss system to support the realized upper floor columns.

The tapering top 197 ft (60 m) of the tower above the residential units will consist of a central braced steel mast supporting access floors to the mechanical units. To reduce the visual impact of structural members, the bracing members for the mast will use 1 in. (25 mm) diameter rods.

The perimeter columns of the tower vary in size and spacing along the building height. The spacings are as follows: 19.7 ft (6 m) at the base and office floors, 9.8 ft (3 m) at the hotel and apartment levels, and 4.9 ft (1.5 m) at the mast levels. The column transitions are made by sets of Vierendeel trusses at selected locations along the height of the tower.

The lateral load resistance for the tower is provided primarily by the reinforced concrete core supplemented by additional shear walls at the hotel and residential levels, and an outrigger hat truss at the top mechanical level.

Structural engineering for the project is by WSP Cantor Seinuk, Inc., New York.

9.3.33 Hearst Tower, New York

Standing nearly 600 ft tall, the Hearst Tower is a 46-story, 456,000 ft^2. office building in New York, a city of iconic skyscrapers. At the base, it is attached to the art deco façade of an existing six-story land mark building, built in Cirea 1928. Thus, the 46-story combines 40 stories of new construction and the original six story Hearst Corporation's headquarters.

The tower has two distinct zones. The lower zone, which rises to the 10th floor, contains street level entrance, a lobby, cafeteria, auditorium, and an open space approximately 80 ft in height. The upper office zone above the 10th level rises all the way up to the building top.

The structural system consists of a perimeter tube composed of triangulated trusses at all four faces of the tower. Because the nodes of the diagonal grid occur at every fourth level, a secondary system of bracing consisting of braced frames is used in the service core area placed asymmetrically toward one side of the tower. The floor system consists of composite steel beams with spans up to 40 ft between the core and the building perimeter.

Typically, wide-flange rolled steel sections are used for the exterior diagram members with bolted connections.

A series of perimeter columns support the diagonals below the 10th floor. At this level, the lateral loads are transferred to the building core consisting of composite shear walls comprised of steel braced frames encased in reinforced concrete walls. The perimeter columns are primarily built-up steel tube sections filled with concrete. Two of the tower's interior columns are also transferred out to the perimeter columns using diagonals below the 10th floor. The horizontal component of the forces in the diagonals is resisted by a system of ring beams and horizontal trusses located in the plane of the third floor diaphragm.

Civer Steel Company, Gouverneur, New York, performed the projects connection design. Of particular interest is the fabrication of typical nodes connecting the diagrid columns and the spandrel beams. The building consists of 84 similar nodes fabricated from 10 in. thick machined steel plates. The diagonal columns bear on the nodes; paddle plates coming off the nodes are bolted to the columns through plates that are welded on the toes of columns. The 10 in. plates keep the diagonal columns away from the centroid of the nodes allowing the beams to connect into the center of nodes. Both standard and oversized holes with slip-critical bolts are used to allow for fit-up tolerance in the field. A stainless steel cladding system covers the diagrid columns and spandrels.

The structural engineering for the project is by WSP Cantor Seinuk, Inc., New York.

9.3.34 Standard Oil of Indiana Building, Chicago, Illinois

The building project consists of a base structure extending five levels below a plaza, covering the entire site measuring 350 ft by 385 ft, and a tower structure extending through and 82 stories above the plaza.

There are about 3 million square feet of floor area in the tower and an additional half-a-million square feet below the plaza surrounding the tower. The tower height above the plaza is 1123 ft and the height above the lowest basement is 1179 ft.

The lateral-load-resisting system for the tower is a tube system achieved by an arrangement of steel plate walls with openings only large enough to accommodate architecturally acceptable vision panels. Thus, the steel for the tube system providing lateral resistance is distributed continuously around the perimeter walls, rather than in intermittent exterior columns and spandrels.

The plates in the exterior wall are arranged in a patter as shown in Figure 9.29a–d. The vertical columns are V-shaped consisting of two plates and are welded to horizontal plates that serve as

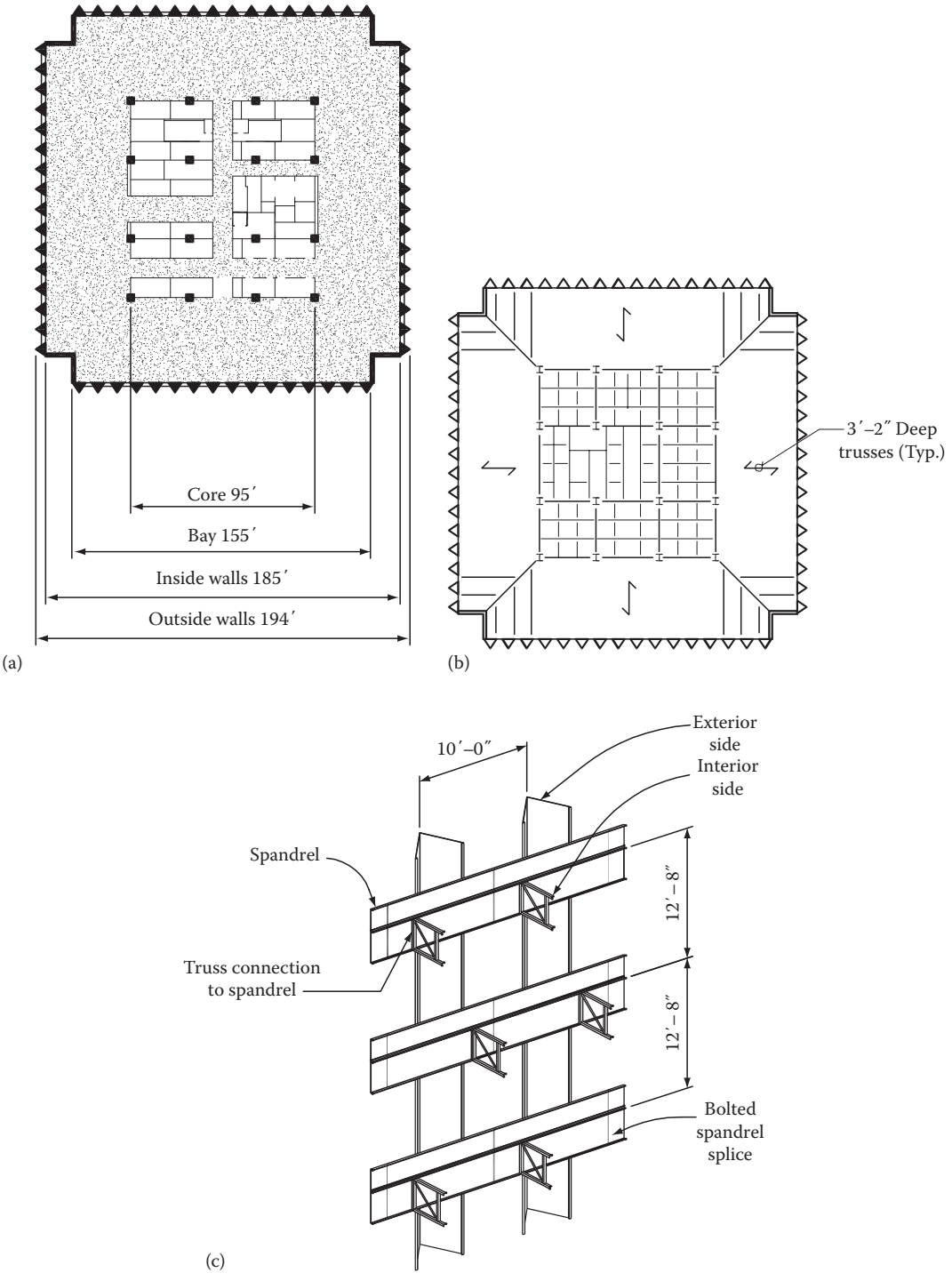


FIGURE 9.29 Standard Oil Building of Indiana, Chicago; (a) building floor plan; (b) floor framing plan, levels 3–19; (c) steel plate arrangement of exterior wall;

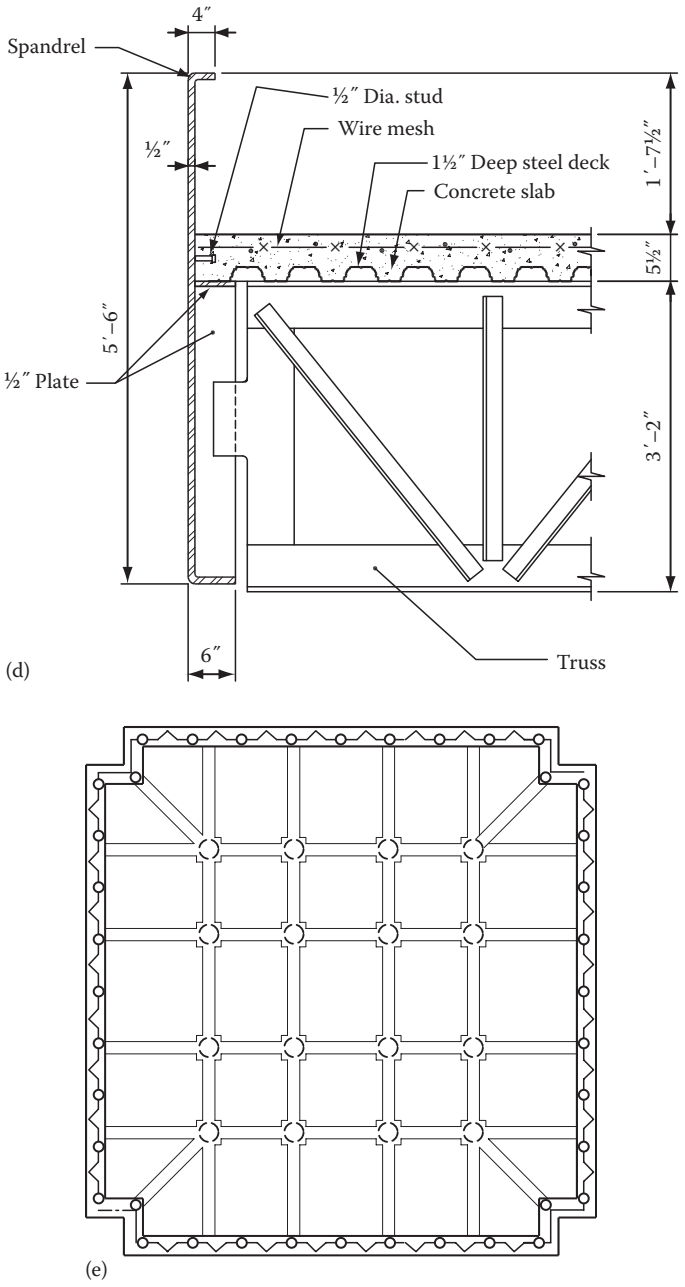


FIGURE 9.29 (continued) (d) cross section of spandrel beam, floor trusses, and deck; (e) plan at top of caissons.

spandrel beams. The balance of the structural system includes the framing for the central core and the floor system. Figure 9.29b shows the floor plan for floors 3 through 19, with 16 interior columns and 64 exterior V-shaped columns.

The reentrant corners made of stiffened steel plates serve to transfer the column axial loads from one face of the building to the other, under the action of the lateral loads.

The main floor span is 45 ft from the core to the exterior and is framed by steel trusses 38 in. deep overall, spaced 10 ft apart. The variable span corner floor beams are wide-flange sections, as are

the four diagonals from the corner columns of the core to the center of the reentrant corners. The floors framing is a blended metal deck system consisting of 3 ft wide panels of 1½ in. deep, 18 gage deck alternating with 2 ft wide panels of 18/20 gage cellular deck. Topping consists of 5000 psi light weight concrete, 4 in. thick for a total diaphragm thickness of 5½ in.

The tower is founded on caissons extending to rock. There are 40 perimeter caissons supporting a transfer girder that carries 64 V-shaped columns, see Figure 9.29e. The caissons vary from 5 ft to 6 ft-9 in. in diameter. The 16 core columns are supported on caissons varying from 8 ft-9 in. to 10 ft-3 in. in diameter. All caissons are constructed of 6000 psi concrete.

One of the main objectives in developing tubular behavior of building structures is to minimize the steel tonnage used in the structural frame. The steel weight per square foot of floor area for the building is 33 psf. The building completed as the Standard Oil Building in 1973 was renamed Amoco Building in 1985 and then as the Aon Center in 1999.

When completed in 1973, the entire building was sheathed with slabs of Italian marble. In 1974, just a year after completion, one of the marble slabs detached from the façade and penetrated the roof of a nearby building. Further inspection found numerous cracks and bowing in the marble cladding. To prevent the problem, stainless steel straps were added to hold the marble in place. Later, from 1990 to 1992, the entire building was resurfaced with white granite.

The project architects and structural engineers were Edward Durell Stone Associates and The Perkins & Will Corporation.

9.3.35 The Renaissance Project, San Diego, California

An example of composite construction in a high-seismic zone is the Renaissance project, a residential development of downtown San Diego, California. It is constructed primarily of cost-in-place conventional and post-tensioned concrete, but has certain unique design features including the use of steel link beams embedded in reinforced concrete shear walls and story-high composite outrigger beams coupling the core to exterior composite columns. The development consists of two 24-story towers placed within a three-story-high podium, and has residential and retail facilities. Additionally, a two-level below-grade parking structures is present under the entire podium. Structural engineering for the project is by KPFF Consulting Engineers, San Diego, California.

Typical framing for residential floors (see Figure 9.30a) consists of an 8 in. (203 mm) thick two-way flat plate. Stud-rail reinforcement is used to resist punching shear column heads. A 12 in. (305 mm) thick slab is used at the third level to resist a relatively heavy landscape loading. The slab also acts as a diaphragm by distributing the lateral loads to the podium shear walls. A 12 in. (305 mm) thick slab also occurs at the 22nd floor to support a two-story steel structure. The entire structure is founded on a variable thickness mat up to 7 ft (2.13 m) thick under the towers. Structural steel framing is used for a two-story structure atop the 22nd level, and inside the tower core walls.

The lateral system consists of a composite core with story-high composite outriggers interconnected to exterior composite columns at the mid-height of the building in the slender direction of the core. Steel beams, W14 × 311 (W360 × 196), are used as flanges of the outriggers with plates up to 1½ in. (38 mm) thick as webs (see Figure 9.30b).

The use of diagonally reinforced concrete coupling beams as required by ACI 318-05, was judged by the design engineers to be impractical. Therefore, steel wide-flange beams up to W18 × 258 (W460 × 383) are used as link beams as shown in Figure 9.30. The core walls are 24 in. (0.60 m) thick from the base to the 16th floor, and then step down to 16 in. thick. The embedment length of the link beams is sufficiently long to develop the full plastic capacity of the steel beam. Typically, this is the plastic shear capacity, $V_p = 0.6F_y(d - 2t_f)t_w$, as the beams are designed to function similar in manner to a ductile link of an eccentric braced system.

Ascertaining satisfactory performance of the gravity system—particularly the flat slab system—subjected to deformations due to seismic lateral loads was a concern. The slab system, together with the columns, behaves as a flat slab-frame and is thus subjected to additional punching

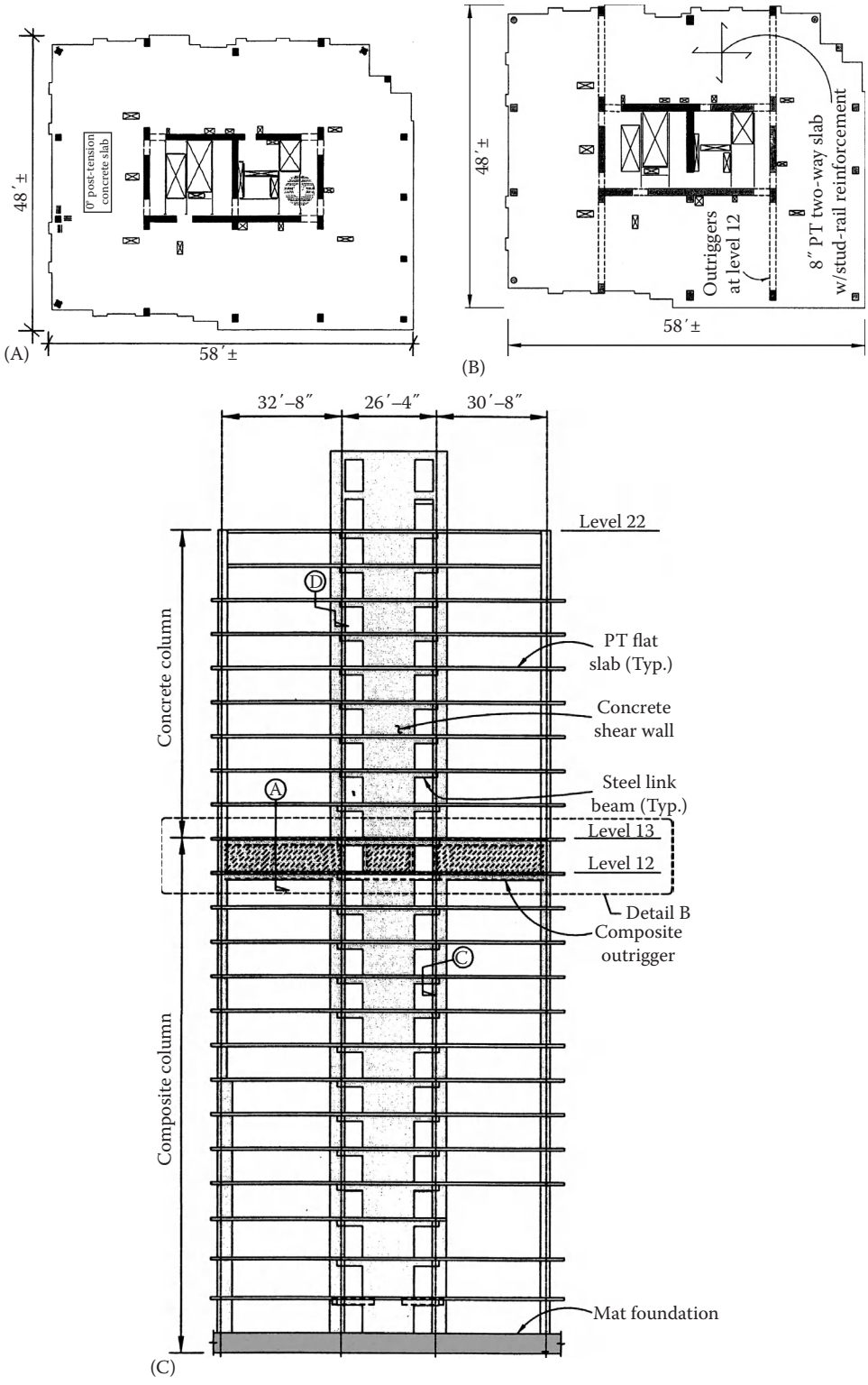
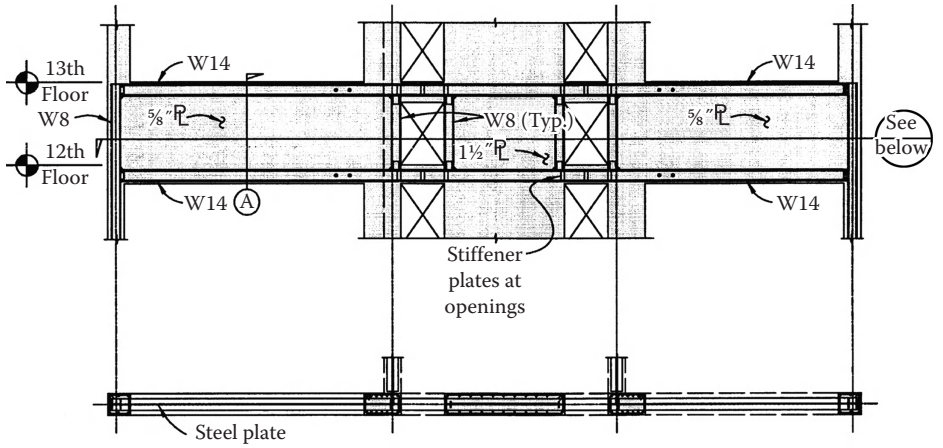
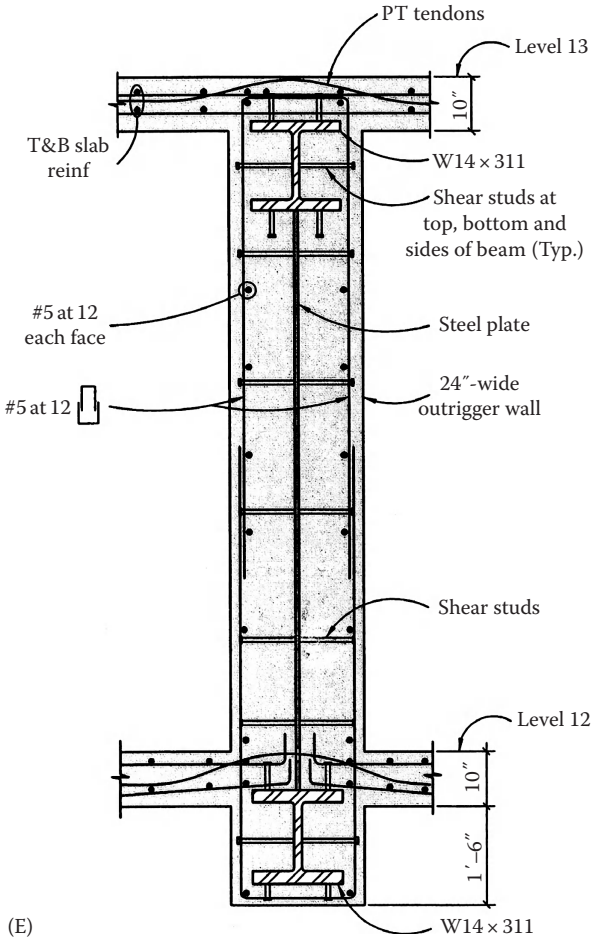


FIGURE 9.30 Renaissance Project, San Diego, California: (A) typical floor framing plan; (B) plan at outrigger level; (C) transverse section showing composite outriggers;

(*continued*)

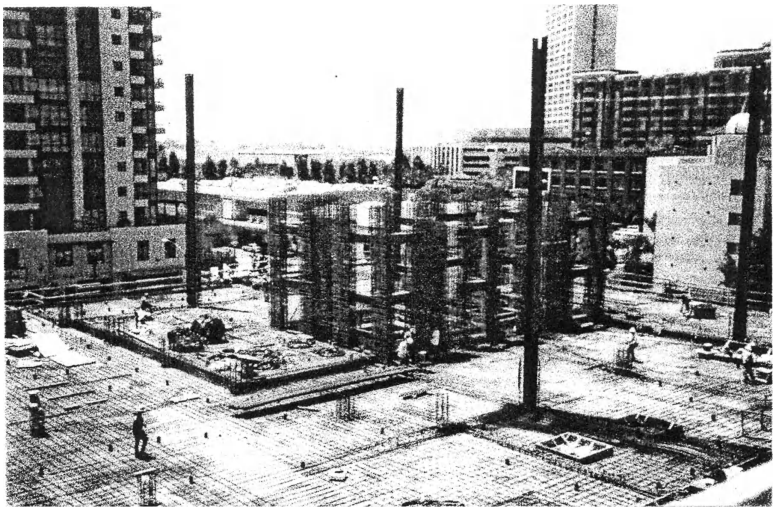


(D)

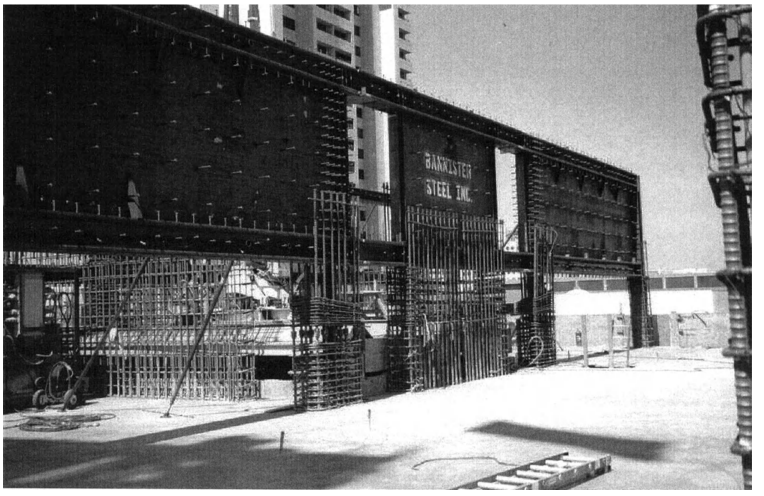


(E)

FIGURE 9.30 (continued) (D) detail B; (E) section A;



(a)

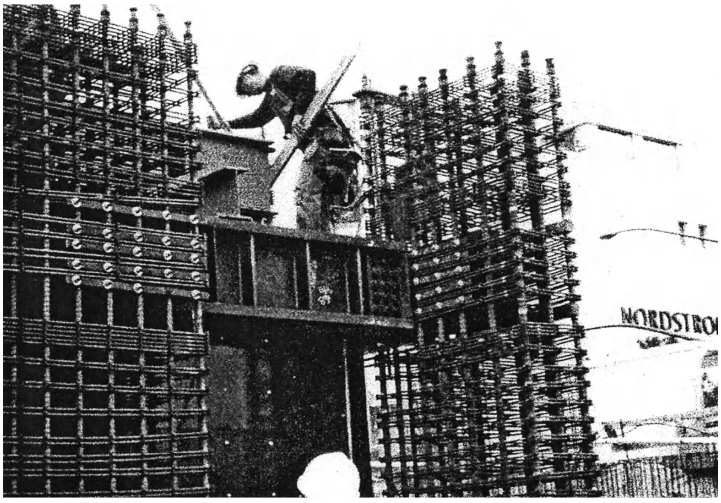


(b)

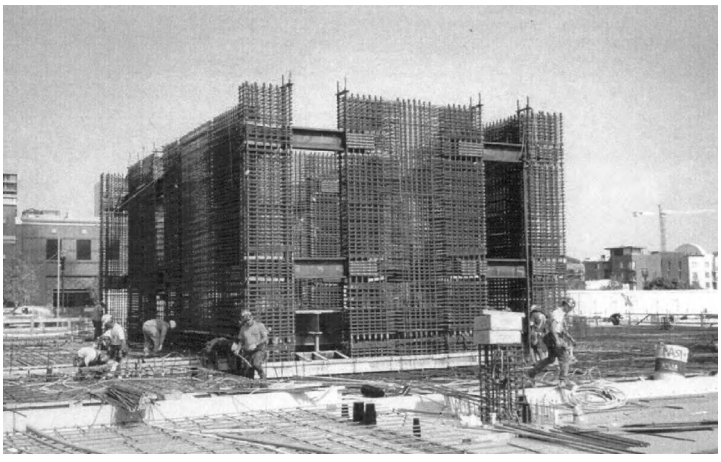
(F)

FIGURE 9.30 (continued) (F) (a), (b) photographs;

(*continued*)



(a)



(b)

(G)

FIGURE 9.30 (continued) (G) (a), (b) photographs.

shears by virtue of the fact that the slab-frame experiences the same lateral deformations at the lateral-load-resisting elements. To determine the shears in the slab system, a 2D model of an equivalent frame with slabs and columns was analyzed by applying the calculated building's drift to the model.

9.3.36 Tokyo City Hall, Tower 1, Japan

Tower 1 of this complex is a high-rise building with a height of 800 ft (243.4 m) consisting of 48-stories above ground and 3-stories underground. The basic structural element in the vertical direction consists of a 21 ft (6.4 m) square super column. The super column is made up of four 40 × 40 in. (1020 × 1020 mm) steel box columns linked by K-braces. Eight such columns run through the building from foundation to the top. These columns are connected by an orthogonal system of beams at each floor level. The super columns are interconnected by a system of 1-story deep belt trusses at the 9th, 33rd, and 44th floors.

A large column-free space of 63 × 357 ft (19.2 × 108.8 m) is established at every floor with the use of deep beams. The typical floor-to-floor height is 13.13 ft (4.0 m). See Figure 9.31 for schematics.

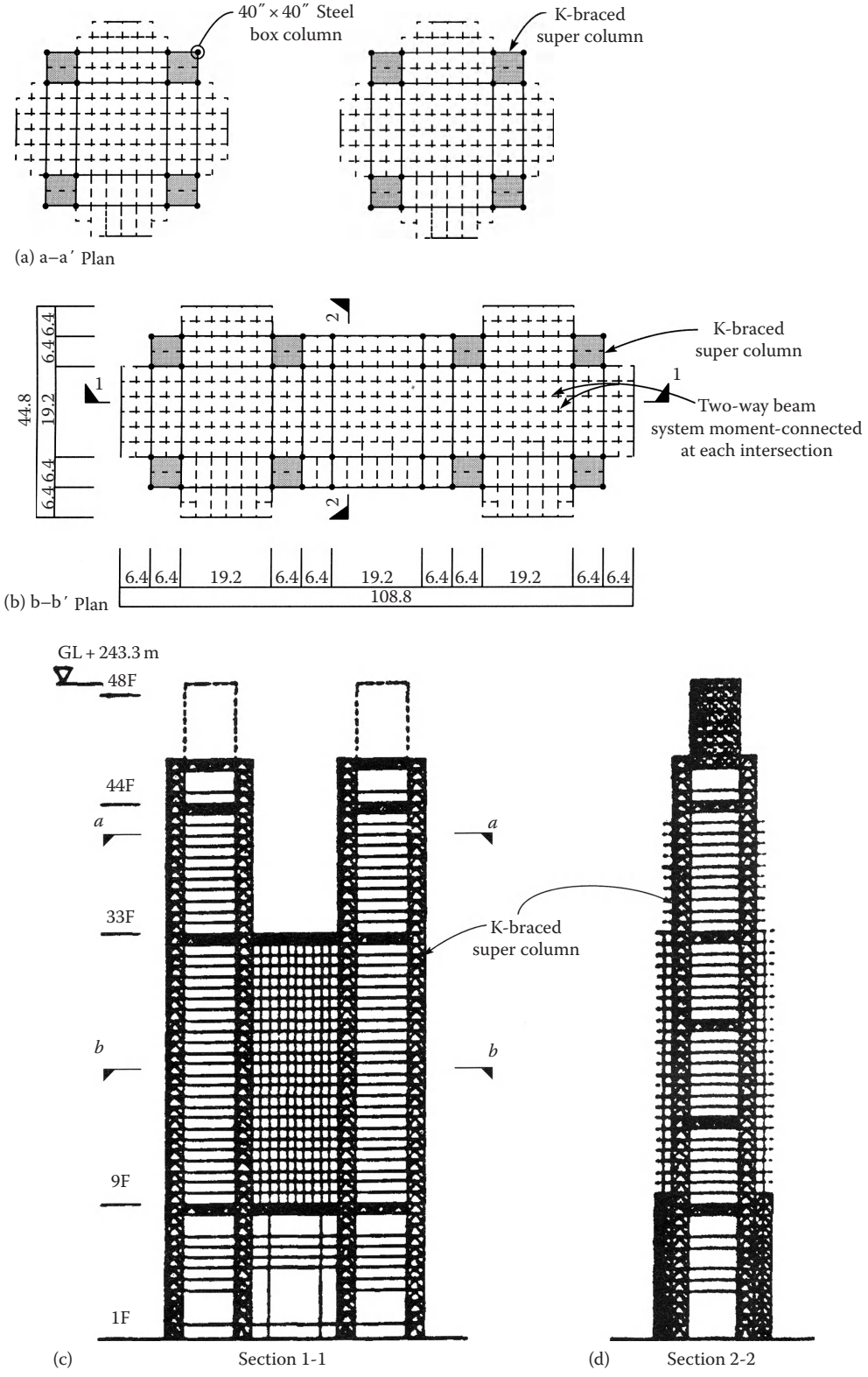


FIGURE 9.31 Tokyo City Hall: (a and b) schematic plans; (c and d) schematic sections.

9.3.37 Bell Atlantic Tower, Philadelphia, Pennsylvania

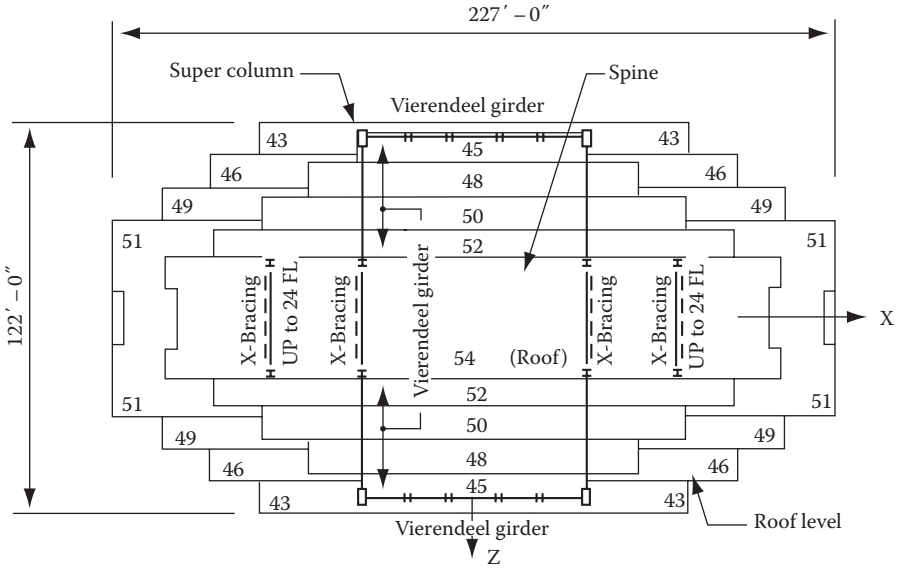
This is a 53-story steel building consisting of a braced core linked to four super columns via a shear-resisting system consisting of two-story Vierendeel girders placed alternately at each side of the core. The varying configurations of the floor plates shown in Figure 9.32a and b, would have required multiple levels of perimeter column-transfer in a conventional tube system making the structural system uneconomical. The braced core system working in concert with the exterior columns, on the other hand, maintains the structural efficiency close to that of a conventional tube system, with the added benefits of providing column-free corner offices. (See Figure 9.32c.)

The building perimeter consists of four major built-up steel columns measuring 54 × 30 in. (1.37 × 0.76 m). These box columns are linked by a series of vertically stacked five-story Vierendeel frames. (See Figure 9.32d.) Each stack of Vierendeels is linked to the one below by a series of hinges designed to transfer horizontal shears only. This detail prevents the build-up of gravity loads in the Vierendeel frame by systematically shedding the loads to the box columns,

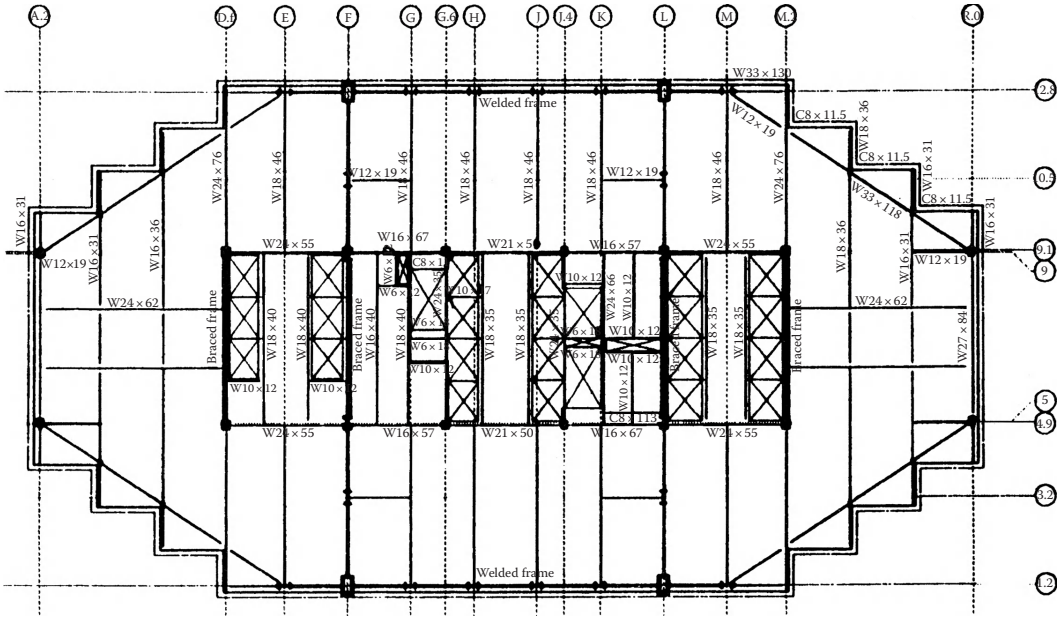


(a)

FIGURE 9.32 Bell Atlantic Tower: (a) building elevation;



(b) Bell Atlantic tower



(c) Bell Atlantic tower

FIGURE 9.32 (continued) (b) composite floor plan; (c) floor framing plan;

(*continued*)

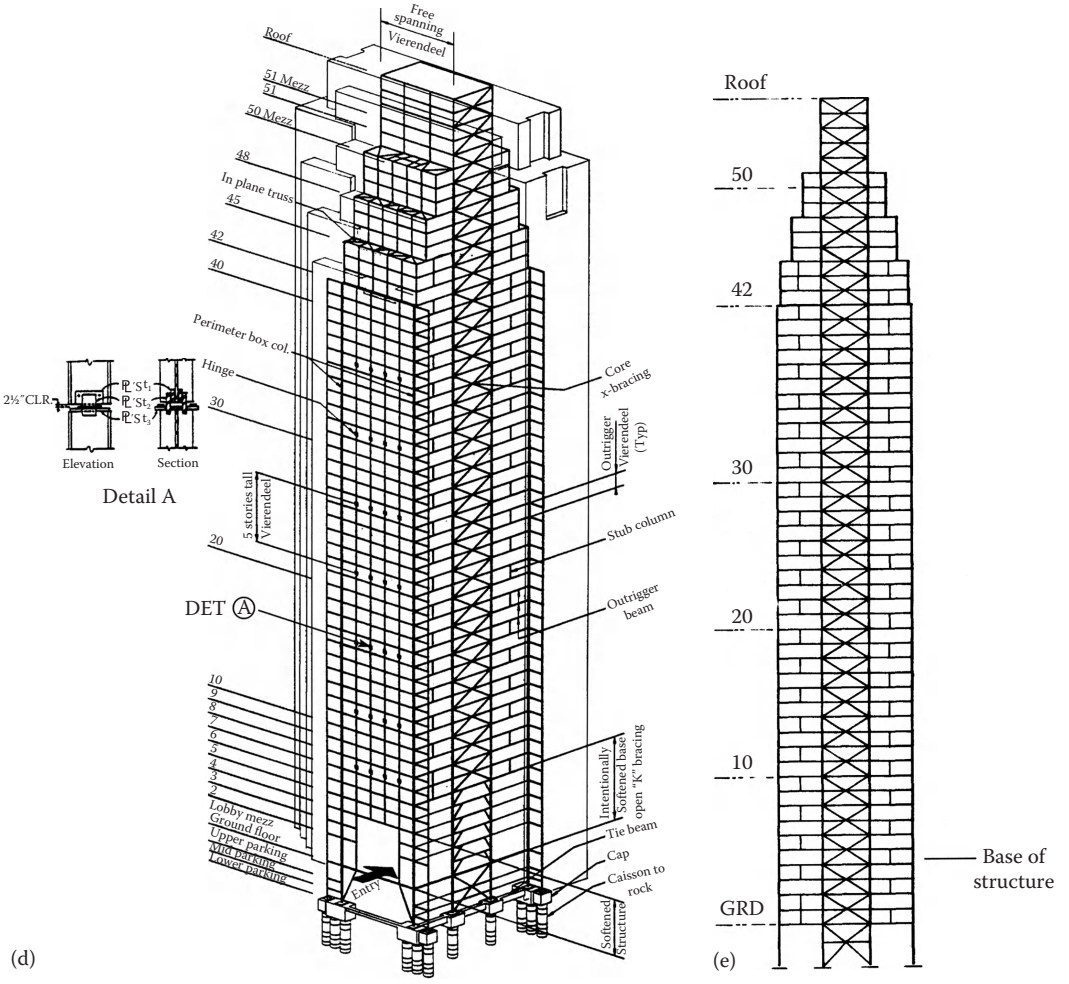
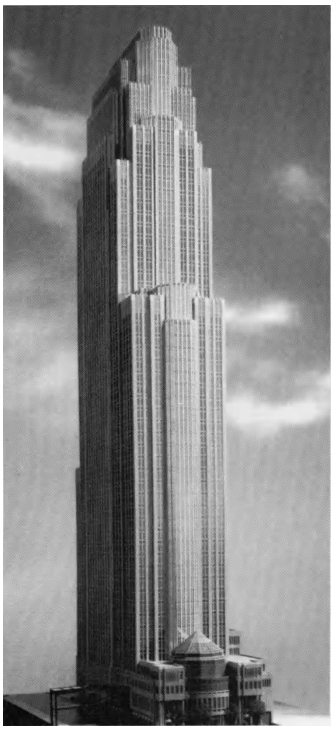


FIGURE 9.32 (continued) (d) lateral system; and (e) section.

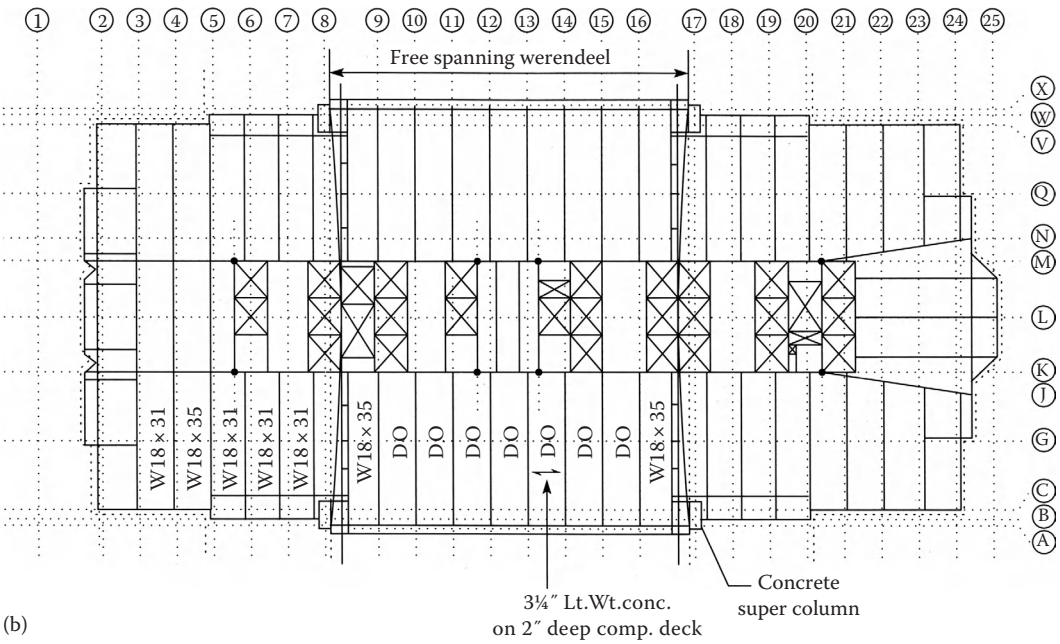
thereby increasing their efficiency in resisting the overturning moments. The lateral resistance in the transverse direction is provided by the braced core linked to the box columns by the two-story Vierendeel girders (Figure 9.32e). The structural design is by CBM Engineers, Inc. Houston, Texas.

9.3.38 Norwest Center, Minneapolis, Minnesota

The structural system for the 56-story bank building (Figure 9.33a) is similar to the Bell Atlantic Tower. The only difference is that composite super-columns are used instead of built-up all-steel box columns Figure 9.33b and c. High strength, 10,000 psi (68.94 MPa) concrete is used in the composite columns. This type of construction has been estimated in the North American construction market to be five to six times less expensive than steel columns of equivalent strength and stiffness. A representative detail of the connection between the steel beam and composite column, and the structural actions associated with moment transfer between the two are shown in Figure 9.33d. Details of hinge used in the Vierendeel frame are shown in Figure 9.33e. The structural design is by CBM Engineers, Inc., Houston, Texas.



(a)



(b)

FIGURE 9.33 Norwest Center: (a) building elevation; (b) typical floor framing plan;

(*continued*)

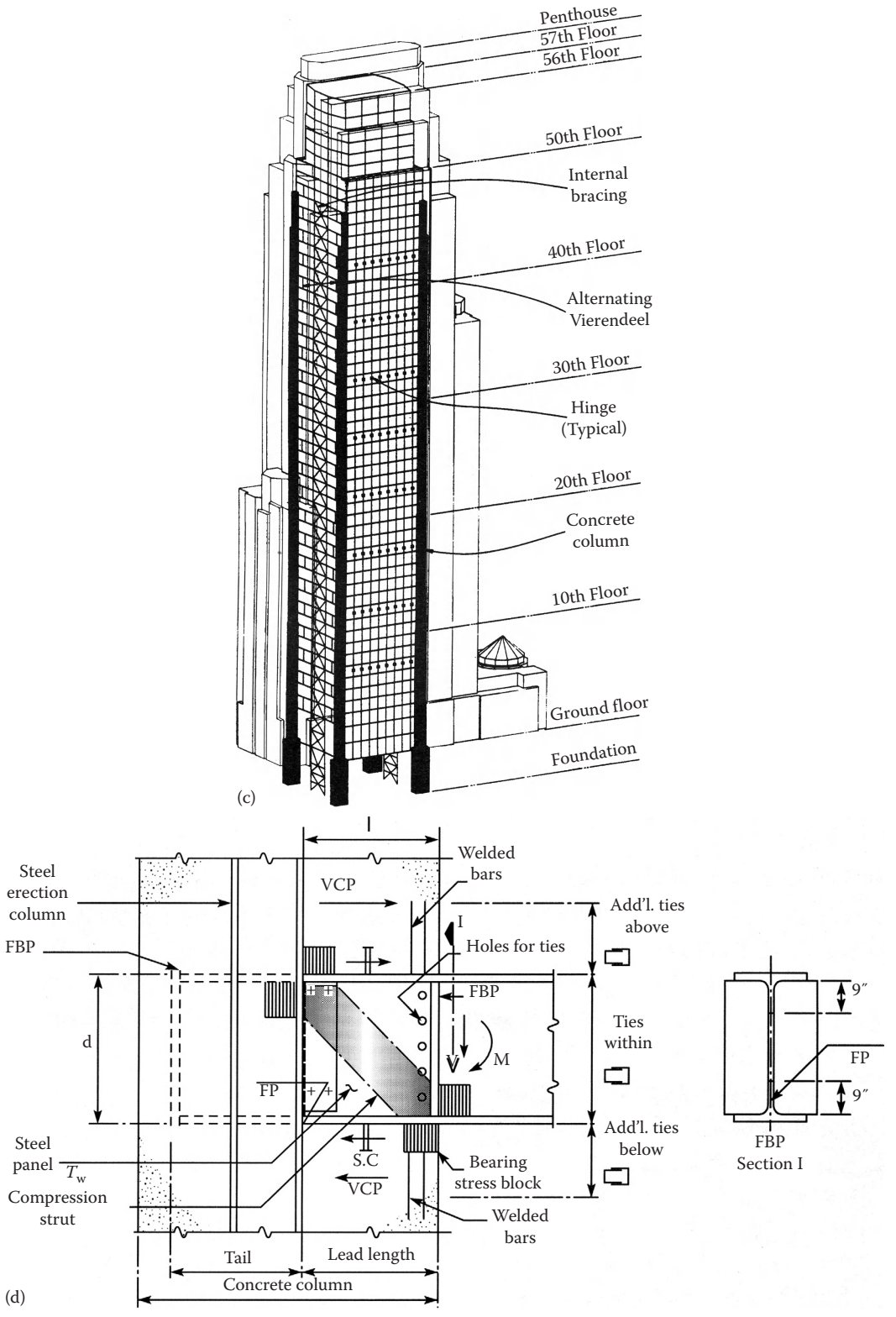


FIGURE 9.33 (continued) (c) structural systems, isometric; (d) structural details, steel beam to concrete column connection; and

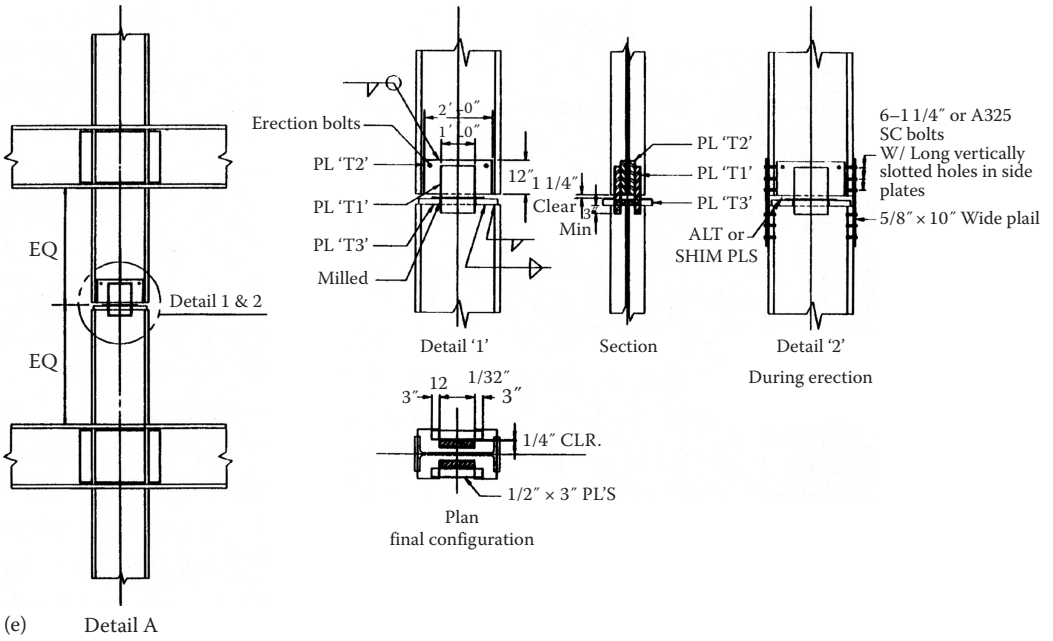


FIGURE 9.33 (continued) (e) hinge details.

9.3.39 First Bank Place, Minneapolis, Minnesota

This is a 56-story, granite-clad building (Figure 9.34a) comprising an array of changing floor plans (Figure 9.34b) with an added architectural stipulation that the north-east corner of the building should have a minimum of structural columns to provide unobstructed views of the city. The response was to come up with an uninterrupted cruciform-shape structural spine as shown. A combination of steel bracing and moment frames consisted with interior space planning is provided along the two spines, AA^1 and BB^1 in Figure 9.34b, to act as shear membranes. Composite columns with concrete strengths up to 10,000 psi (68.44 MPa), and varying in size from 74 ft^2 (7m^2) at the base, to 50 ft^2 (4.65 m^2) at top, are used at the spine extremities as shown in Figure 9.34c.

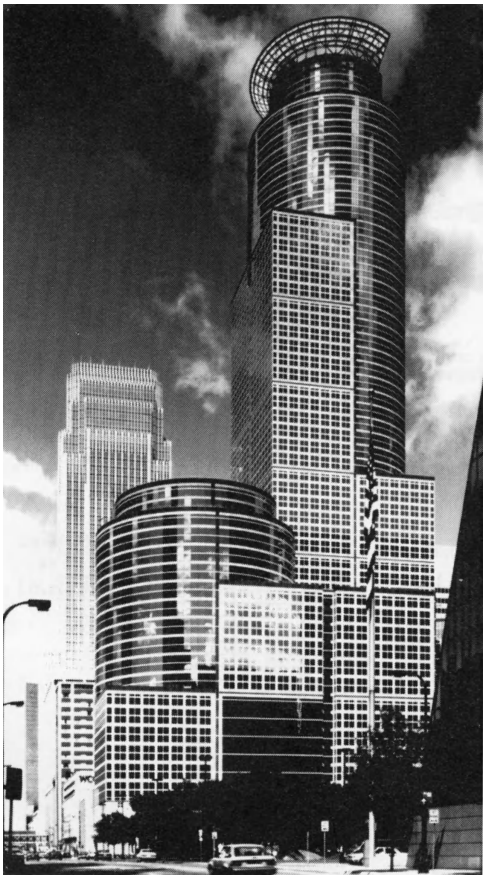
Since the cruciform shape is torsionally unstable, the spine is stabilized by providing braced frames along the perimeter BC and B^1C^1 and by a series of three-story tall Vierendeel trusses along lines CC^1 and B^1D^1. A 12-story tall Vierendeel girder along A^1D^1 links the composite concrete supercolumn at A^1 to the perimeter steel column D^1 (Figure 9.34d). It also supports the perimeter circular Vierendeel along BAB^1E above the 45th floor. This circular Vierendeel provides both torsional and lateral resistance to the entire frame. The entire system is an outstanding example of strategic structural response to complex building geometry and leasing demands without having to pay an undue premium for the structural system. The structural design is by CBM Engineers, Inc., Houston, Texas.

9.3.40 Allied Bank Tower, Dallas, Texas

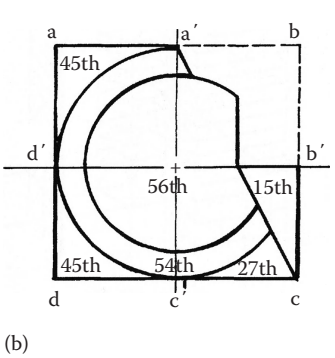
This is a 60-story building with an unconventional geometrical shape rising to 726 ft (221.3 m) above a tree-studded plaza in downtown Dallas (Figure 9.35a). The geometrical composition of the tower shown in Figure 9.35b consists of (1) a rectangular block 192 ft (58.53 m) square in plan raising to 54 ft (16.46 m) above the plaza; (2) a 480 ft (146.3 m) tall geometric shape that wedges gradually from a square at the bottom to a 96 × 192 ft (29.3 × 58.5 m) rhombus; (3) a 16-story-high skewed triangular prism capping the building top. The combined 672 ft (204.83 m) high building is raised on a four-story-high pedestal that matches the rhombus shape at the top as shown schematically

in Figure 9.35c. The resulting unconventional shape serves an eye-catcher with a distinct exterior façade and a unique sloping top.

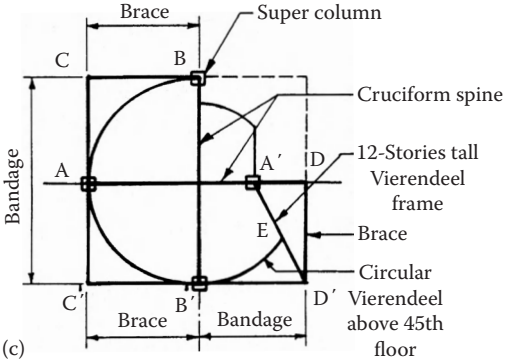
The structural system shown in Figure 9.35b and c consists of a perimeter trussed frame that performs a dual function by providing the required lateral resistance, and an architecturally desired free-span at the base. This system has a 40-story deep megatruss on each of the 156 ft (47.55 m) side, and an eight-story deep truss on each of the 96 ft side as shown in 9.35d. The trussed exterior frames are set back 3 ft (0.92 m) from the building skin to clear the curtain wall. Also the geometry of the diagonal



(a)



(b)



(c)

FIGURE 9.34 First Bank Place: (a) building elevation; (b) composite floor plans; (c) plan of structural system; and

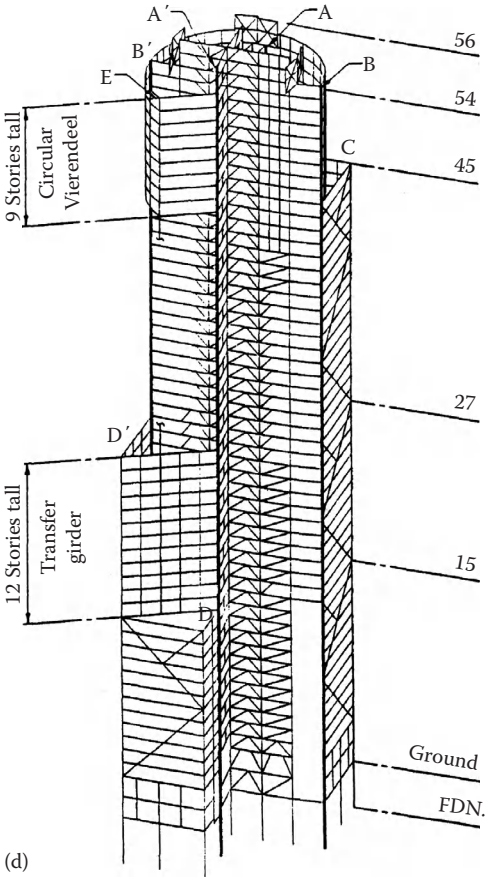


FIGURE 9.34 (continued) (d) isometric of structural system.

dictated by the architectural and leasing requirements resulted in the intersection of the diagonal at mid-height of columns (Figure 9.35d). Therefore, a conventional truss action in which the truss diagonal, the column, and the floor beam, all meet at a common node, could not be provided to transfer the unbalanced horizontal components of axial forces in the diagonals, directly to the floor members. As an alternative solution, a story-deep Vierendeel truss was designed to resist the moments caused by the unbalanced horizontal components of the forces in the diagonals.

Above the 45th level, moment frames are used to resist the lateral loads in both directions. To facilitate a smooth floor of forces between the trussed and moment-connect regions, an overlap zone was created by extending the moment frames nine floors below the apex of the trussed floor. A schematic of the structural system is shown in Figure 9.35c.

To eliminate the temporary shoring, otherwise required for erecting steel four levels above the open plaza, a one-story deep truss is used all around the perimeter between the fifth and sixth levels. The truss is designed to support the erection load of eight floors above the fifth level. The eight-story deep subtruss (Figure 9.35c) together with the Vierendeel truss at the 12th level and the truss at the fifth level supports the construction loads of the next eight floors. This sequence of supporting the construction loads is carried through, for the entire structure.

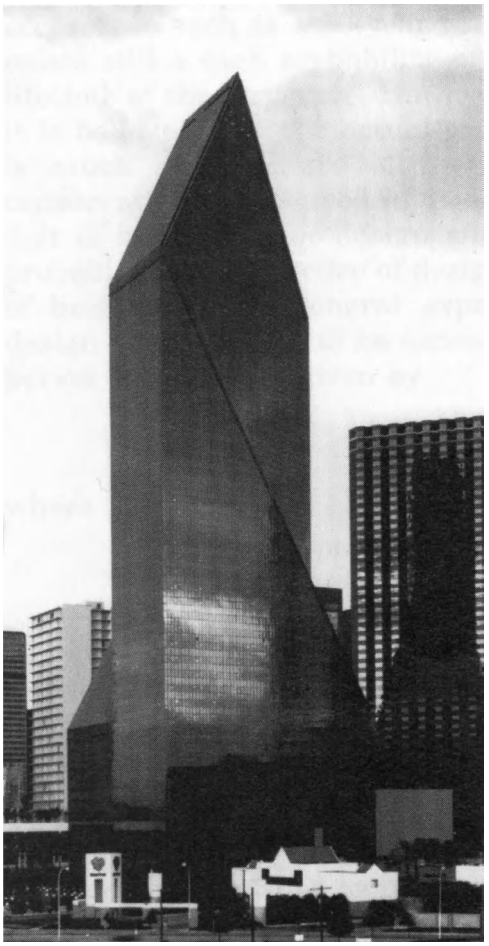
Below the fifth level, two 30 ft wide (9.15 m) pylons located on opposite sides of the building are the only elements for transferring the entire north-south wind shear from above. Heavy built-up columns with W14 wide flanges as cross-bracing resist shear and overturning without significant uplift in the columns.

The floor spandrels between the intersection of the diagonals are moment connected to the diagonals to serve three purposes: (1) to act as a secondary lateral system to transmit wind loads applied at each level to truss panel points, (2) to provide additional lateral restraint to the system, and (3) to provide lateral bracing for the compression diagonals of the truss.

Both 36 and 50 ksi steel are used for the project. All diagonals of the mega truss are W14 wide-flange shapes. The maximum weight of built-up trapezoidal column with 8 in. (203 mm) thick plates is 2450 lb/ft (35 kN/m). Two-sided gusset plates are used at the intersection of diagonals with the box columns, while single-gusset plate with stiffeners matching the wide-flange diagonals and columns are provided at the intersection of the diagonals (Figure 9.35d).

The typical floor construction consists of 50 ksi (344.74 MPa) composite floor beams with a 3 in. deep (76 mm) metal deck and 2½ in. thick (63.5 mm) normal weight concrete topping. Instead of welded wire fabric, fiber-reinforced concrete is used in the slab as reinforcement. Additional mild steel reinforcement is used at Vierendeel floors and at levels 13 and 45, where the shape of the exterior façade changes.

The building consists of three basements below the plaza level. Shallow drilled piers bearing on 50 ton capacity rock support the building. The unit weight of the structural steel is 22 psf (1053.4 Pa). The floor-to-floor height is 12 ft (3.66 m) as compared to 13 ft (3.96 m) normally used for this type



(a)

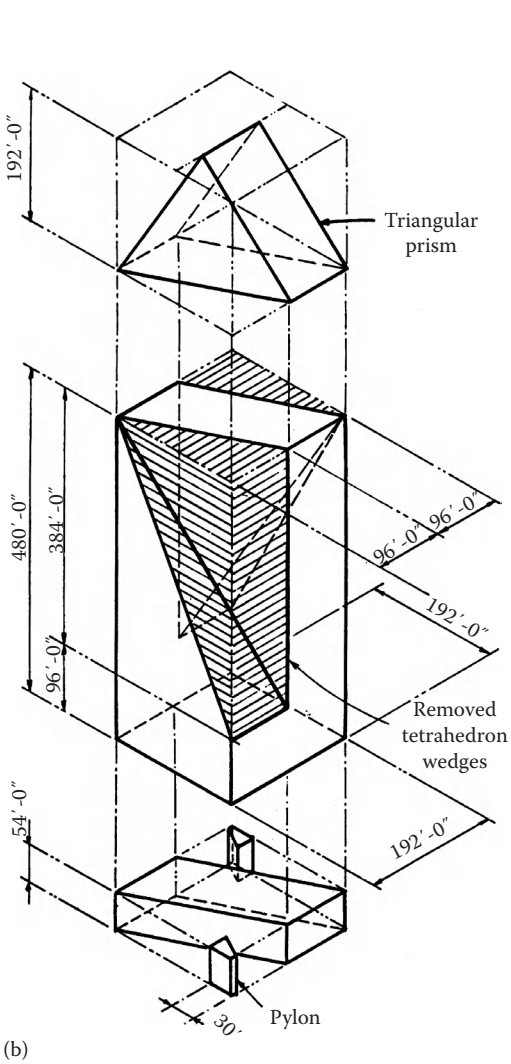
FIGURE 9.35 Allied Bank Tower, Dallas, Texas: (a) building elevation;

of office buildings. The exterior column spacing of 48 ft (14.63 m) offers generous views from the building interior even with the presence of diagonals. The structural design is by CBM Engineers, Inc., Houston, Texas.

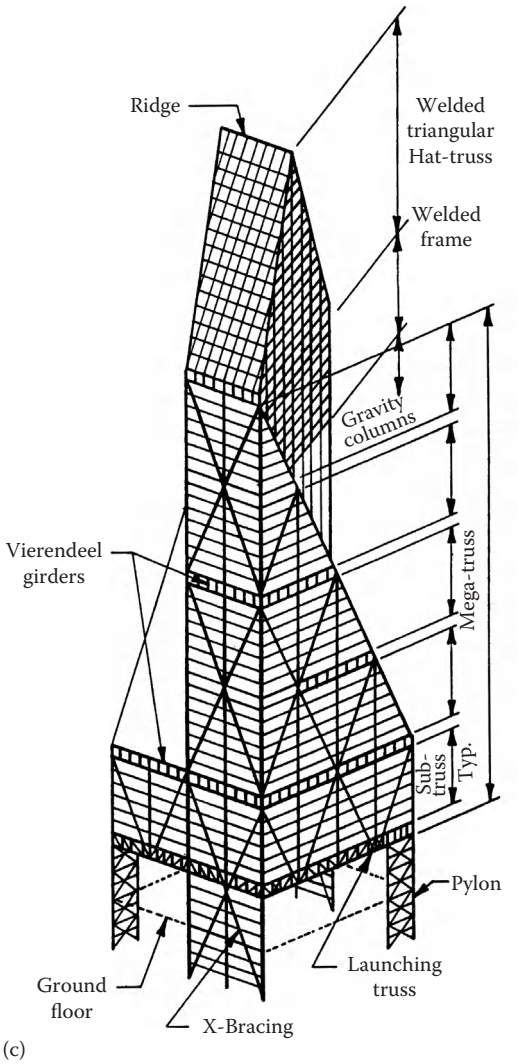
9.3.41 Future of Tall Buildings

September 11, 2001 did not mark the end of the skyscraper era. Instead, the race for tallness continues unabated. But the race is happening not in America, but in the Far East, making this past 10 years the tall building era of the Far East. The world's tallest building is now in Dubai.

What is the motivation behind the race? To be candid, the reasons are the same today as they were some 70 years ago; height now, as then, is an exhibition of technology and power. Nothing is more expressive than an upright symbol, particularly the one with high-tech items such as pressurized double-decker elevators, external damping devises to reduce sway caused by windstorms,



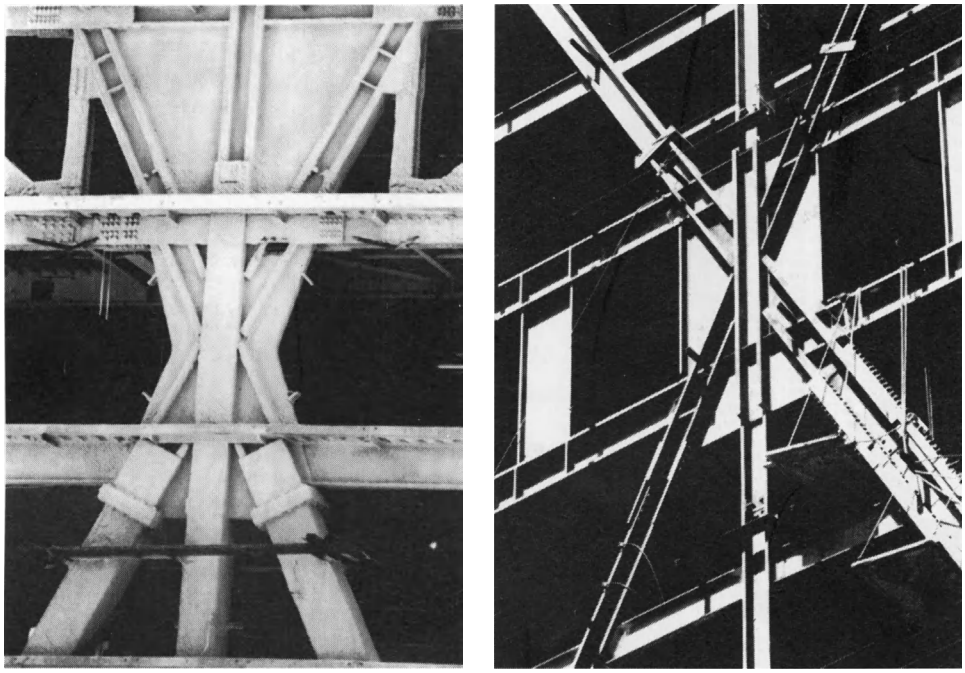
(b)



(c)

FIGURE 9.35 (continued) (b) building geometry; (c) schematic view of structural system; and

(*continued*)



(d) 1 2

FIGURE 9.35 (continued) (d) intersection of diagonals, detail 1, detail 2.

and fiber-optics incorporated into curtain walls that transform buildings into giant billboards. Tall buildings become instant icons, putting their cities on the world map.

Given humanity's competitive nature, it is hard to believe that Burj Dubai at 2717 ft (818 m) will wear the crown long. The quest for the title of world's tallest building is alive and well. This begs the question, how tall can buildings go? Answer: No limits are in sight, at least from structural considerations. Humanity has an obsession with super-tall structures, particularly when humans can live and work in them. While there are indeed lessons to be learned from the WTC catastrophe, the skyscraper will remain viable well into the foreseeable future.

9.4 BUILDING MOTION PERCEPTION

A constantly buffeting wind will exert a fluctuating pressure on a building, which can be approximated by a mean pressure. Such is not the case for wind *gusts* that buffet the building surface with fluctuating pressures. Wind gustiness is dynamic by nature and its effects, in a broad sense, may be considered similar to those resulting from earthquakes. Just as a building experiences accelerations due to sudden ground motions, so it does under gusty winds, particularly at top floors. Wind-induced oscillations of the Tacoma Narrows Bridge are a vivid example of dynamic nature of wind.

Human tolerance to wind-induced motions has become increasingly important as building structures are built to ever increasing heights. It is known for quite some time that prediction of dynamic properties of buildings such as building fundamental period, mode shapes, and damping, particularly for those with an aspect ratio greater than 8:1, have a great effect on the predicted accelerations. In spite of the great strides made in wind engineering over the past two decades, the subject of occupant tolerance to building motion is still not an exact science. It is rather subjective being a highly complex mix of engineering, psychology, and physiology and not well understood, even by wind engineering specialists.

Because acceleration limits have been developed from subjective criteria, it is perhaps prudent to have building owners and stake-holders participate in the selection of the acceleration design

criteria. Having the owners experience building accelerations in a motion simulator such as the one at the Hong Kong University of Science, may be something worth considering at the very beginning of a tall building project.

9.5 STRUCTURAL DAMPING

The estimation of structural damping has been of great interest to structural engineers. This is especially the case in more recent years, since the number of tall buildings and structures throughout the world has increased dramatically. Many model and full-scale studies of these structures have been performed, and are still being done today. The motivation behind this interest is usually the comfort of the occupants of the building, as well as peak loads and deflections. As buildings become more slender, the damping characteristics often become the most important variable. For these structures, the dynamic response tends to be the dominant structural action, and excessive vibrations could lead to unhappy owners and occupants alike. The majority of the structures, however, have sufficient inherent damping to control excessive vibrations with respect to the ultimate strength limit state. The additional criteria that are of more concern are the serviceability limit state. The serviceability limit state relates to the occupant comfort, or the usability of the building when subjected to design loading rather than its behavior in extreme events.

The definition of loads and deflections in the strength limit state are defined by the physical properties of the construction materials and the structural system that have been in existence for quite some time. They are well understood and used without much disagreement in practice. The serviceability limit state with respect to accelerations, however, is a more recent development. As the development of the skyscraper progressed, engineers recognized the difficult nature of describing the effect of motion on the occupants of buildings. In the early 1930s, engineers began to report on the dynamic performance of tall buildings. Heavy and stiff buildings were noted to inspire the occupants with confidence. The success of commerce throughout the world and the progression in construction methods and materials led to higher and lighter structures. The importance of defining the dynamic characteristics was noted for these taller buildings. The researchers realized that the perception of motion was subjective, and depended on the individual experiencing building motions. This led to moving room experiments with people, to determine thresholds for tolerance of building accelerations without undue discomfort. These experiments consisted of a moving "room," with occupants either sitting, standing, or lying down. Various recommendations resulted from the experiments. These experiments, however, do not fully define the experience of the occupants of a building. There are various other factors that have not been included, such as audio cues (creaking of building) and visual cues (swinging of the horizon). Further research on the combined effects of the physiological response to motion and additional audio/visual cues is continuing.

It is of interest to note that in most early skyscrapers, the dynamic effects due to the effect of wind were not investigated. This did not lead to any severe problems, as the construction materials and methods of the day, made for very stiff and massive structures. The Empire State Building is a classical example. The amount of steel and masonry that it was built with is several times that of current buildings. For example, the unit quantity of the structural steel in the New York's Empire State Building is 42 psf as compared to the 33 psf for Sears Tower (now called Willie's Tower), and 29 psf for the John Hancock Center, both in Chicago, Illinois. This large quantity of material used in the earlier skyscrapers led to a higher stiffness, as well as contribution to the damping from masonry infills and nonload bearing partitions. This reduced the dynamic amplification of the wind-induced response and as a result, occupant comport was achieved without much effort from engineers. As buildings became more flexible and lighter, however, more attention to the dynamic amplification was required.

If a structure is prone to excessive vibrations resulting in peak accelerations that are higher than the recommended threshold, the most effective way of reducing the vibration is by adding damping. Although there are other ways of modifying the dynamic characteristics such as by changing the

mass and stiffness, increasing the damping is generally the only economically viable alternative. In present-day wind-sensitive tall buildings, due to the method of construction and/or design, there is not enough inherent damping in the structural system itself. This has led to the notion of increasing the total damping by additional damping.

There are many different systems of adding damping to a structure. They can be grouped in two classes: Active systems and passive systems. Active systems rely on the input of energy from a mechanical and electrical system to effectively add damping. A tuned-mass-damper such as was installed in the First National City Corporation headquarters in New York City, is an example of such an active system. This system requires several thousand kilowatts of electricity to operate the damper. Passive systems on the other hand, depend only on the dynamic characteristics of the structure and the damper–structure interaction to provide additional damping. The grouping of passive systems can be broken into two further classes. The first subclass is those systems that change the nature of the response of the structures. One example of this type would be sloshing water damper, which modifies the mechanical characteristics of the structural system. The second subclass would be those systems that remove energy from the system by the adding friction. A viscoelastic damper is an example of a damper that removes energy by the shearing action of a layer(s) of rubberlike material sandwiched in a mechanism attached to the structure at various locations.

Various buildings and structures throughout the world have added damping system. The First National City Corporation building in New York City, is a 910 ft building that is elevated above a plaza with four 112 ft columns. Early in the design stage, it was realized that additional damping would be required. A tuned mass damper was used to reduce vibrations to an acceptable level. The now nonexistent World Trade Center Towers in New York City had added dampers as well. Nearly 10,000 viscoelastic dampers had been installed in each tower of the World Trade Center.

The effect of damping is less dominant for seismic response than it is for dynamic wind response. As mentioned previously, the choice of damping parameter is not easy, if only because there have been very few measurements on tall buildings. The current recommendations are based on measurements made over many years on relatively less tall buildings.

Based on a study of Japanese database of published and unpublished measurements, it appears that the inherent damping of buildings has high variability with a trend to reduce with building height.

Typical structures possess 1%–5% inherent damping contributed by nonstructural elements. Engineered damping system can be built into new construction or used for retrofit of existing structures to significantly increase the structural damping. Many damped structures have critical damping of 15%–35%. The four major types of dampers used in the United States are fluid viscous dampers, viscoelastic dampers, friction dampers, and metallic-yielding dampers. Each type has specific characteristics, advantages and disadvantages for structural applications, and each type is made by several different manufacturers. To design with dampers, it is critical to capture the dynamic characteristics and the nonlinear behavior of the structure. This permits the designer to determine the amount of additional damping and stiffness that is required to achieve the desired performance. Engineered damping systems are, therefore, designed on the basis of performance-based design.

9.6 PERFORMANCE-BASED DESIGN

The purpose of PBD is to give a realistic assessment of how a structure will perform when subjected to either particular or generalized earthquake ground motion. While the code design provides a pseudo-capacity to resist a prescribed lateral force, this force level is substantially less than that to which a building may be subjected during a postulated major earthquake. It is assumed that the structure will be able to withstand the major earthquake ground motion by components yielding into the inelastic range, absorbing energy, and acting in a ductile manner as well as by a multitude of other actions and effects not explicitly considered in code applications. Although the code requires special ductile detailing, it does not provide a means to determine how the structure will actually perform under severe earthquake conditions. This is the role of PBD.

PBD in a manner of speaking is not new. Historically, its use can be traced back to the 1960s, when the now nonexisting New York World Trade Center Towers, Chicago's John Hancock Building, and Sears Towers (now Willi's Tower) were designed using PBD principals to wind engineering.

When these buildings were designed, wind engineering was in its early childhood. The definition of appropriate demand levels and corresponding acceptance criteria were nonexistent. At the time, they were developed from scratch by leading industry experts. Definitions of recurrence intervals of wind events for strength and serviceability limits and commensurate acceptance criteria for thresholds of occupant comfort were developed for the first time. These have been refined over the years and are now well established and accepted generally by practicing professionals. The basic framework for performance-based seismic design (PBSD) may thus be traced back to the 1960s. However, introduction of PBD to seismic community is relatively new. It was introduced in FEMA 273/274, published in October 1997, which was then reissued in November 2000 as FEMA 356—*Prestandard and commentary for the seismic rehabilitation of buildings.* ASCE 41-06—*Seismic Rehabilitation of Existing Buildings* has since superseded both versions of the FEMA standard.

The reasons for the emergence of PBDs (also noted in some publications as PBSD) are many fold. Based on experience gained from about 20 tall buildings built in San Francisco thus far, it seems owners like them because they cost less; architects go for it because they offer more design freedom; contractors love them because they are easier to build. And engineers being thrifty like them because they can result in higher quality structures with the least amount of materials and labor to accomplish the required task in a safe and efficient manner.

9.6.1 Alternative Design Criteria: 2008 LATBSDC

The acronym LATBSDC stands for Los Angeles Tall Buildings Structural Design Council. The council has put forth a document "Alternate Design Criteria" in which a performance-based approach for seismic analysis and design of tall buildings is provided.

For the purpose of this document, tall buildings are defined as those with height h_n greater than 160 ft above average adjacent ground surface, roughly corresponding to a 12-story office building or an 18-story apartment building.

The PBD design methodology is based on capacity design principles followed by a series of performance-based design evaluations. Thus, to qualify under this procedure, a structure must necessarily have a suitable ductile yielding mechanism capable of experiencing nonlinear lateral deformations. For example, a special moment resisting frame, SMRF, must have inelastic zones typified by (1) flexural yielding of beam ends, (2) shear yielding in column-beam panel zones, (3) yielding at column bases.

The adequacy of the design is then demonstrated by analytically verifying the building performance using two distinct levels of earthquake ground motions due to

1. *Extremely rare earthquake* defined as an event having a 2% probability of exceedance in 50 years (approximated to a 2500 return period). This earthquake is defined as the maximum considered earthquake, MCE, in ASCE 7-05/10. The purpose of this evaluation is to demonstrate that collapse does not occur when the building is subjected to ground motions corresponding to extremely rare seismic events. Performance of all structural members including those not considered as part of lateral-load-resisting systems is evaluated. Cladding and their connections to the structure must accommodate displacements corresponding to MCE without failure.
2. *Frequent earthquakes* defined as an event having a 50% probability of being exceeded in 30 years (43 year return period). The purpose of this evaluation is to verify that the structural and nonstructural components of the building remain essentially elastic thus retaining their functionality during and after the postulated seismic event. Repairs, if necessary, are expected to be minor and could be carried out without unduly affecting the normal functionality of the building. This criteria is deemed satisfied if the analysis shows only minor yielding of ductile yielding of primary structural system.

Buildings designed according to the Alternate Design Criteria of LATBSDC shall be designed for a minimum base shear equal to 3% of the buildings seismic weight *W*.

$$V_{\min} = 0.030W$$

The base shear may be distributed along the height of the building according to the equivalent lateral procedure of ASCE 7-05 Section 12.8.3. As an alternate, an elastic response spectrum analysis may be performed using a scaled design spectrum, and a complete quadratic combination, CQC, procedure for modal response combination. The requirement of design for a minimum base shear of 3% of the building weight is not a performance-based design provisions but has been retained to continue design tradition of Los Angeles tall buildings.

The analysis shall be evaluated for the following load combination

$$1.0 + L_{\text{exp}} + 1.0E$$

where

D is the service dead load
L_{exp} is the expected service live load
E is the earthquake load

The provisions of ASCE 41 and ASCE 7-05, Section 16.1.3 may be used for verifying the acceptability criteria for both collapse prevention (strength limit) state and serviceability limit state.

Independent project-specific peer review is a requirement under the provisions of the alternative procedure. The reader is referred to the original document for further details. Another document of interest for designers of tall buildings in seismically active regions anywhere in the world is the "Recommendations for the seismic design of high-rose buildings," 2008, published by the Council on Tall Buildings and Urban Habitat, CTBUH.

9.6.2 Recommended Administrative Bulletin on the Seismic Design and Review of Tall Buildings Using Nonprescriptive Procedures AB-083

This document known as AB-083 for short, was published in 2007 by the Structural Engineers Association of Northern California (SEAONC), and adapted by the San Francisco Department of Building Inspection (SFDBI). While both AB-083 and 2008 LATBSDC share the same philosophy of PBD for the seismic analysis and design of tall buildings, their approach is somewhat different.

For example, AB-083 permits code approaches with certain exceptions while 2008 LATBSD bases its provisions on Capacity Design Procedures only. However, it should be noted that 2008 LATBC requirement of 3% of the building weight as the minimum base shear is in fact, a prescriptive equivalent lateral force procedures permitted in most seismic codes.

The reader is referred to publication AB-083 for further details.

9.6.3 Pushover Analysis

A pushover analysis is basically a step-by-step inelastic analysis for which the lateral loads of constant relative magnitude are applied to a given structure and progressively increased until a target displacement is reached, while gravity loads are kept constant. Thus, as the name implies, the structure is analytically pushed sideways (or pushed over) to determine its ultimate lateral-load resistance as well as the sequence of yielding events needed to reach that goal, or the magnitude of plastic deformations at the target displacement. Typically, this is accomplished by repeatedly

running linear elastic structural analysis computer programs, modifying the model of a structure as necessary to account for the progressive appearance of plastification at finite locations throughout the structure. The pushover analysis method is relatively accessible and has been used in addition to conventional analyses to determine the ultimate capacity of important existing structures, to validate proposed retrofit or design solutions, and to compare the ultimate capacity and, to some extent, the ductility of various design alternatives.

Although, in principle, nothing precludes the extension of this concept to conduct cyclic pushover analysis for a limited number of cycles, the method rapidly becomes excessively arduous without the help of special purpose computer programs. Consequently, nearly all practical applications of the pushover analysis so far have considered monotonically increasing lateral loads.

It must be recognized that the information acquired from a pushover analysis is highly dependent on the lateral load distribution adopted. Therefore, whenever the chosen lateral load distribution is intended to capture the possible effects of dynamic excitation, it may be wise to consider multiple lateral-load distribution patterns.

9.6.4 Concluding Remarks

Performance-based design can be used for a variety of purposes such as design verification for new construction, evaluation of an existing structure to identify damage states, and correlation of damage scenarios to various amplitudes of ground motion.

Simply stated, the procedure compares the capacity of the structure, typically determined using a pushover analysis with the demands imposed on the structures as postulated by using a demand spectra. It should be noted that in order to account for nonlinear behavior of the structural system, effective viscous damping is sometimes applied to linear elastic spectra.

The purpose of PBD is to give a realistic assessment of how a structure will perform when subjected to either particular or generalized earthquake ground motion. While the code design provides a pseudo-capacity to resist a prescribed lateral force, this force level is substantially less than that to which a building may be subjected during a postulated major earthquake. As stated previously, it is assumed that the structure will be able to withstand the major earthquake ground motion by components yielding into the inelastic range, absorbing energy, and acting in a ductile manner as well as by a multitude of other actions and effects not explicitly considered in code application. Although the code requires special ductile detailing, it does not provide a means to determine how the structure will actually perform under severe earthquake conditions. This is the role of PBD.

It is of interest to watch as groups of researchers hyperbolize PBD as an exact solution and compare other *approximate* solutions to this *exact* solution. In their opinion, a nonlinear dynamic analysis is often deemed to be the exact solution. It is however, difficult to understand how such a claim can be made because every computer analysis requires development of a computer model of the building's structural system. In the computer, the model is then subjected to a digitized version of each of the ground motions. At best, the computer model is an idealized mathematical model that is based on a mixed bag of assumptions regarding strength, behavior, and configuration of the component structural materials and assemblages. Therefore, the computer analysis is, at best, able to generate an exact numerical solution to a reasonable but inexact set of assumptions.

The insight we gain by a nonlinear time history evaluation of how a building has actually performed in a past earthquake may not be all that is required to give us total confidence in its use. This is because there is always a nagging doubt that such an *after the fact* analysis is more than likely to benefit from prior knowledge of the building's actual performance and more often than not liberally tweaked to match the analysis to the actual observed performance. Therefore, it may not be prudent to rely entirely on the results of an after the fact analysis of an existing building to predict equally well as *before the fact* seismic performance of a building to a yet-to-occur earthquake with unknown characteristics.

To accurately predict how a building will perform, one must have accurate data on both demand and capacity; however, the accuracy of the answers cannot be greater than the accuracy of the input data. This is not to say that nonlinear time history analysis does not provide important information about structural response; but it is not "exact" as claimed by its promoters.

Over the years, PBD procedures have evolved significantly from their beginnings; however, there has been a push to develop increasingly complex, codified, PBD procedures. In the author's opinion, by codifying PBD, the very essence of PBD, that is, the focus on the attributes and behavior of an individual building, is destroyed. Engineers must be given sufficient latitude to arrive at the best estimate of a building's capacity.

PBD is a useful tool for design and to estimate the performance characteristics of buildings subjected to strong earthquake ground motion. It is not a *magic predict all* procedure. It takes a combination of analytical procedures, data evaluation, judgment, and experience to get a credible approximation of how a building works in the inelastic range of lateral motion.

Since the publication of the ASCE 31 and 41 standards, the PBD process that underlies these standards has been widely used. As popular as these first-generation procedures have become, they are of unknown reliability, present difficulties with regard to definition of performance intent, and may create significant liability for the design professional using them. Since 2001, FEMA has been sponsoring the ATC-58 project to develop next-generation performance-based design criteria. Intended to eventually replace the technology contained in the present ASCE standards, this new methodology permits engineers to characterize performance directly in terms of probably repair costs, occupancy interruption time, and casualties associated with building response to earthquakes. Slated for completion in 2011, the new procedures are anticipated to revolutionize the practice of performance-based earthquake engineering.

9.7 PRELIMINARY ANALYSIS TECHNIQUES

Building design today principally consists of presenting a 3D model of the building to the computer, and then modifying the model to satisfy strength and serviceability requirements dictated by the governing code. Thus, we execute, rather than create the design.

However, before the analytical model is handed over to the computer, it is our business to make sure that it satisfies the basic tenets of equilibrium, load path development, and compatibility. Computers help us in the analysis phase of the design process, but it is the engineer who manipulates the arrangement of the structural system components to resolve behavioral flaws in structural systems.

The importance of attaining this knowledge at a fundamental level, as opposed to a mathematically precise level, cannot be over emphasized. Preliminary designs using fundamental concepts give us this opportunity.

In the lateral load analysis of buildings, wind and earthquake forces are typically treated as equivalent static loads reduced to a series of horizontal loads applied at each floor level. Portal and cantilever methods offer quick ways of analysis of a rigid frame. The idea behind both these methods of analysis is based on the well-observed characteristics of portal frames, namely, that the points of contraflexure in beams and columns tend to form near the center of each column and girder segment. For purposes of analysis, the inflection points are assumed to occur exactly at the center of each member.

In the portal method, a moment frame is treated as a series of consecutive single-bay portal frames for determining axial stresses in the columns due to overturning. Interior columns are considered as part of two such portals. Therefore, compression arising from the overturning effect of the leeward column of one portal is offset by tension on the windward column of the adjacent portal. With the shears in each column known and the points of contraflexure preestablished, the moments in beams and columns are determined from simple statics.

In the cantilever methods, as the name implies, the building is analyzed as a cantilever standing on end fixed to the ground. The overturning moment is assumed to be resisted by axial compression

in columns on the leeward side of the neutral axis and tension in columns on the windward side. The neutral axis is determined as the centroid of the areas of columns, and the axial forces due to overturning are assumed to be proportional to their distances from the neutral axis. As in the portal method, the points of inflection are assumed to occur at mid-height of columns and mid-span of girders. From the known axial forces in columns and location of points of contraflexure, moments in columns and girders are obtained.

It is important to emphasize that in structural analysis field, as in many others, use of computers and commercially available computer programs rule the world. Hand calculations given here for frame analysis are not substitutes for such computer analysis. They are given here with an eye to stressing the basics and to serve as procedures for checking computer output. The goal is to assist young engineers who analyze complex structures using computer techniques to understand what computers are doing and why they are doing it.

Even in today's high-tech computer-oriented world with all its sophisticated design capability, there still is a need to perform approximate analysis of structures. First, the analysis provides a basis for selecting preliminary member sizes because the design of a structure, no matter how simple or complex, begins with a tentative selection of members. With the preliminary sizes, an analysis is made to determine if design criteria are met. If not, an analysis of the modified structure is made to improve its agreement with the requirements, and the process is continued until a design is obtained within the limits of acceptability. Starting the process with the best possible selections of members results in a rapid convergence of the iterative process to the desired solution.

Second, because of the ever-increasing cost of labor and building materials, it is almost mandatory for the structural engineer to compare several designs before choosing the one most likely to be the best from the points of view of structural economy and how well it minimizes the premium required by the mechanical, electrical, and curtain wall systems. Of the myriad structural systems that present themselves as possibilities, only two or three schemes may be worthy of further refinement requiring full-blown computer solutions. Approximate methods are all that may be required to logically arrive at cost figures and to sort out the few final contenders from among the innumerable possibilities. It is very time consuming, costly, and indeed unnecessary to undertake a complete sophisticated analysis for all the possible schemes. Preliminary designs are therefore very useful in weeding out the weak solutions.

Sophisticated computer analysis are indispensable in reducing the number of inaccuracies caused by hand analysis techniques and are being used routinely in everyday engineering practice. Although such computer analysis may intimidate the structural engineer by virtue of their unbelievable amount of documentation and output, the prudent engineer will always verify the reasonableness of the computer analysis by using approximate hand-calculated values for forces, moments, and deflections. Approximate analysis is, therefore, a powerful tool in providing the engineer with (1) a basis for preliminary sizing of members, (2) an orderly method for evaluating several schemes to select the most likely one for further study, and (3) methods for obtaining approximate value of forces, moments, and deflections to check on the validity of the computer solutions.

9.7.1 Portal Method

This method is the most commonly used approximate method for analyzing tall buildings. The following are the simplifying assumptions made in the portal method.

1. A point of contraflexure occurs at the center of each beam.
2. A point of contraflexure occurs at the center of each column.
3. The total horizontal shear at each story is distributed to columns of that story in such a way that each interior column carries twice the shear carried by each exterior column (other rational distributions such as column shear based on tributary widths of columns are also often used).

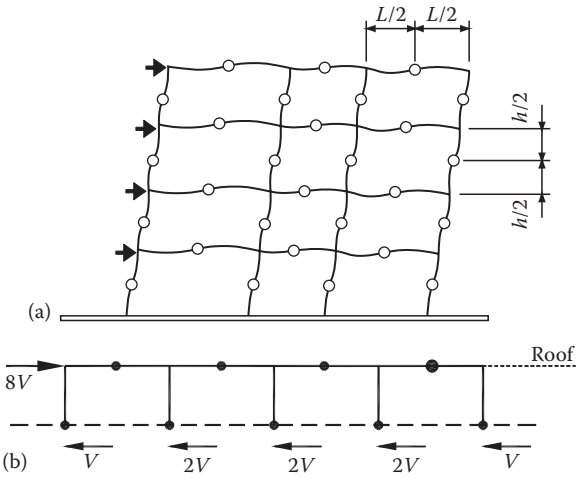


FIGURE 9.36 (a) Portal and cantilever methods of frame analysis; assumed locations of inflection points. (b) Portal method: horizontal shear distribution in top story columns.

The aforementioned assumptions convert the indeterminate multi-story frame to a determinate structure. The steps involved in the analysis of the frame are detailed as follows:

1. The horizontal shears on each level are distributed between the columns of that floor according to assumption (3). It should be noted, however, that it is quite rational to assume that shear is distributed in proportion to the tributary bay width. This is the assumption used in the examples given later in this section.
2. The moment in each column is equal to the column shear multiplied by half the column height according to assumption (2).
3. The girder moments are determined by applying moment equilibrium equation to the joints by noting that the sum of the girder moments at any joint equals the sum of the column moments at that joint. These calculations are easily made by starting at the upper left joint and working joint by joint across to the right end.
4. The shear in each girder is equal to its moment divided by half the girder length. This is according to assumption (1).
5. Finally, the column axial forces are determined by summing up the beam shears at each joint. These calculations again, are easily made by working from left to right and from the top floor down.

Figure 9.36 shows the assumptions of the portal and cantilever methods of analysis diagrammatically and Figure 9.36a shows the distribution of horizontal shear in top story columns. Figure 9.37 shows the schematic flow of forces resulting from portal analysis.

9.7.2 Cantilever Method

This method gives good results for high-narrow buildings compared to those from the portal method. It may be used satisfactorily for buildings of 25–35 stories tall. It is however not as popular as the portal method.

The simplifying assumptions made in the cantilever method are as follows:

1. A point of contraflexure occurs at the center of each beam.
2. A point of contraflexure occurs at the center of each column.
3. The axial force in each column of a story is proportional to the horizontal distance of the column from the center of gravity of all the columns of the story under consideration.

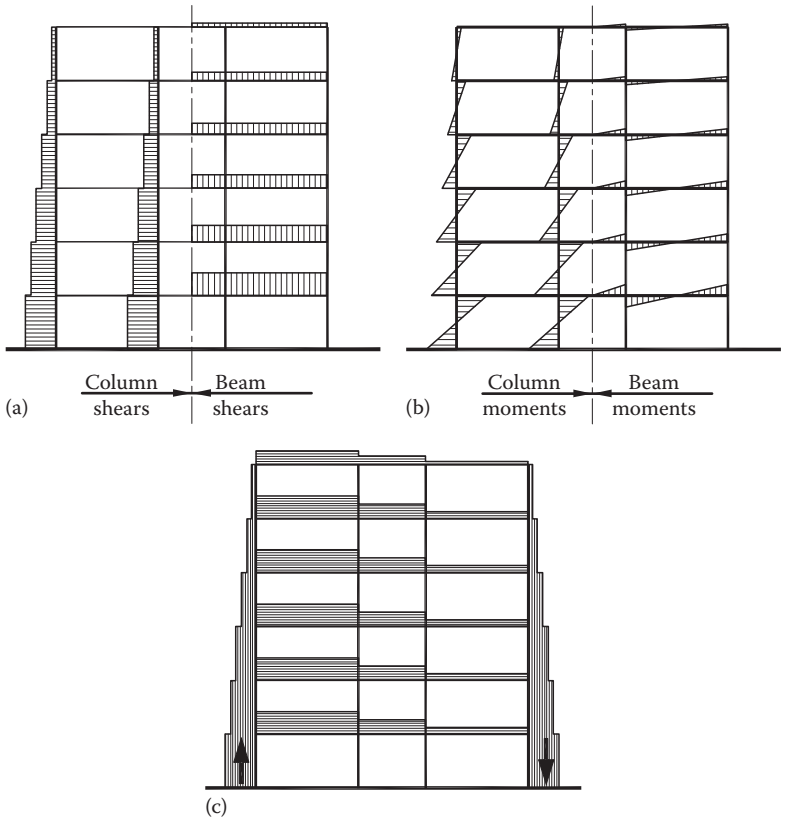


FIGURE 9.37 Schematic flow of forces, portal method: (a) shear forces; (b) bending moments; (c) axial forces.

The steps involved in the application of this method are as follows:

1. Locate the center of gravity of columns by taking moment of area of all the columns and dividing by sum of the areas of columns.
2. Determine the axial forces in columns. For this purpose, we start with the top story of the frame subject to a lateral force shown in Figure 9.38. The axial forces in the columns are represented by F_1, F_2, F_3, and F_4 with the columns placed at a distances of x_1, x_2, x_3, and x_4 from the centroidal axis respectively.

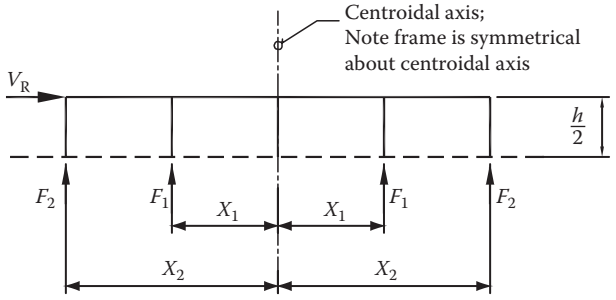


FIGURE 9.38 Cantilever method: axial force distribution in top story columns. $V_R \times h/2 = 2(F_2X_2 + F_1X_1)$.

The axial force in a given column is determined by using the moment equilibrium relation:

$$V_R \times h/2 = 2(F_2 x_2 + F_1 x_1)$$

3. The beam shears are determined, joint by joint, from the column axial forces.
4. The beam moments are determined by multiplying the shear in the beam by half the span of beam according to assumption (1).
5. The column moments are similarly found, joint by joint, from the beam moments. The column shears are obtained by dividing the column moments by the half-column heights using assumption (2).

9.7.3 Design Examples: Portal and Cantilever Methods

We will now consider the application of the portal and cantilever methods to a 30-story frame shown in Figure 9.39, consisting of two equal exterior bays and a smaller interior bay. Listed in the same figure are the lateral loads assumed in the analysis. The procedure in the portal method is as follows.

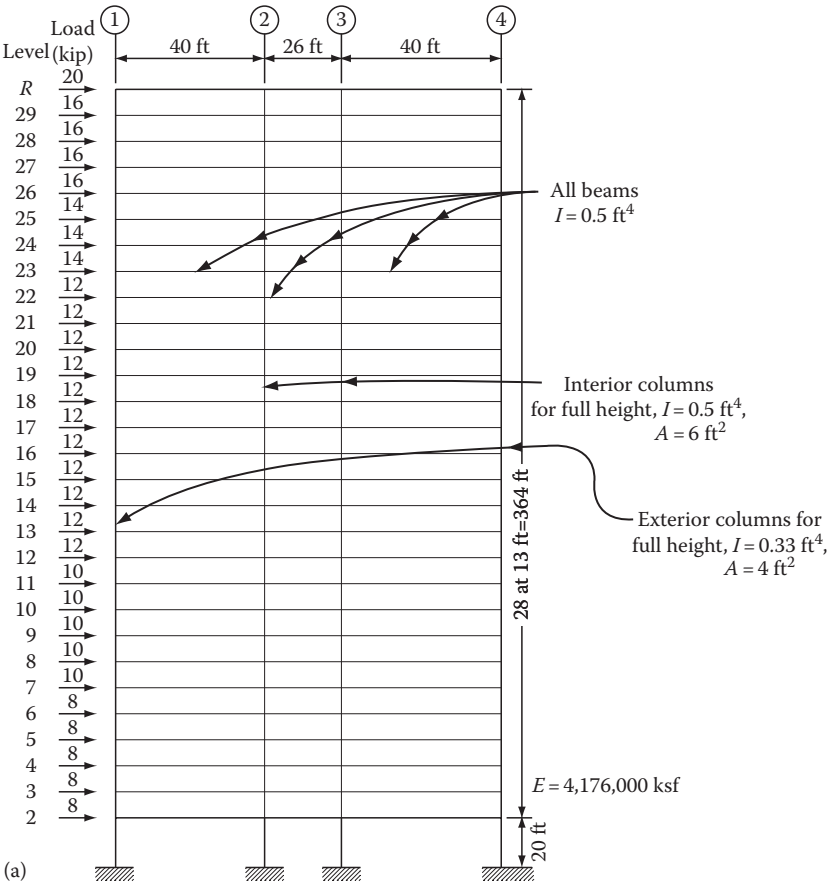
Distribute the accumulated story shears to each column in proportion to the aisle widths. Calculate the moments in the top and bottom of each column as a product of the known shear in the column and one-half of the story height. Next, starting at the upper left corner of the frame, determine the girder moments where the column and girder moments are the same. Since the points of contraflexure are assumed at the center of girder, the moments at each end are equal but opposite in sign. Determine the girder shears by the relation and shear multiplied by half of span lengths equal girder end moment. Next the axial forces in the exterior columns are determined directly from girder shears. The results for the example frame are shown in Figure 9.40.

For the analysis of the frame using the cantilever method, we refer to Figure 9.41a that shows a free-body diagram of the top story above the points of contraflexure in the columns. The frame axis of rotation is located at the center of gravity of the columns, which for the example problem coincides with the line of symmetry of the frame. The column axial forces for the top story are obtained by equating the moment of the column reactions about the frame centroidal axis to the external moment of the wind forces taken about the horizontal plane through the assumed hinges of the top floor. There are shown in Figure 9.41a.

In a similar manner, the axial forces in the columns of other stories are computed by passing a horizontal section through the points of contraflexure of columns of each story and considering the moment equilibrium of the frame above the section (Figure 9.41b–d).

After the column axial forces are found, the girder shears are determined at once. For example, in Figure 9.41b, the shear in the exterior girder at the 15th level is 210.72 – 187.08 = 23.64. Figure 9.41c shows the procedure.

With the girder shears known, the girder moments follow directly. These are equal the shear in the girder times one-half of the beam span. Using the relation that $\Sigma M = 0$ at a given joint, the moments in the columns at the roof are obtained from roof girder moments (Figure 9.41a). Since the points of contraflexure in the columns are at mid-height, the column moments above the 29th level have the same value as at the roof level (Figure 9.41b). Moments in the columns below the 29th level are obtained again, from the relation $\Sigma M = 0$, as are the moments in other floors. The column shears are obtained by diving column moments by half the column height. As a check, observe that the sum of shears in the columns of any level equals the sum of the horizontal external loads above that level. The moments and forces obtained by using the aforementioned procedure for the example problem are shown in Figure 9.41.



Level	Story shear (kip)	Accumulated Shear (kip)	Level	Story shear (kip)	Accumulated Shear (kip)
R	20	20	15	12	222
29	16	36	14	12	234
28	16	52	13	12	246
27	16	68	12	12	258
26	16	84	11	10	268
25	14	98	10	10	278
24	14	112	9	10	288
23	14	126	8	10	298
22	12	138	7	10	308
21	12	150	6	8	316
20	12	162	5	8	324
19	12	174	4	8	332
18	12	186	3	8	340
17	12	198	2	8	348
16	12	210			

(b)

FIGURE 9.39 Example frame: (a) dimensions and properties; (b) lateral loads.

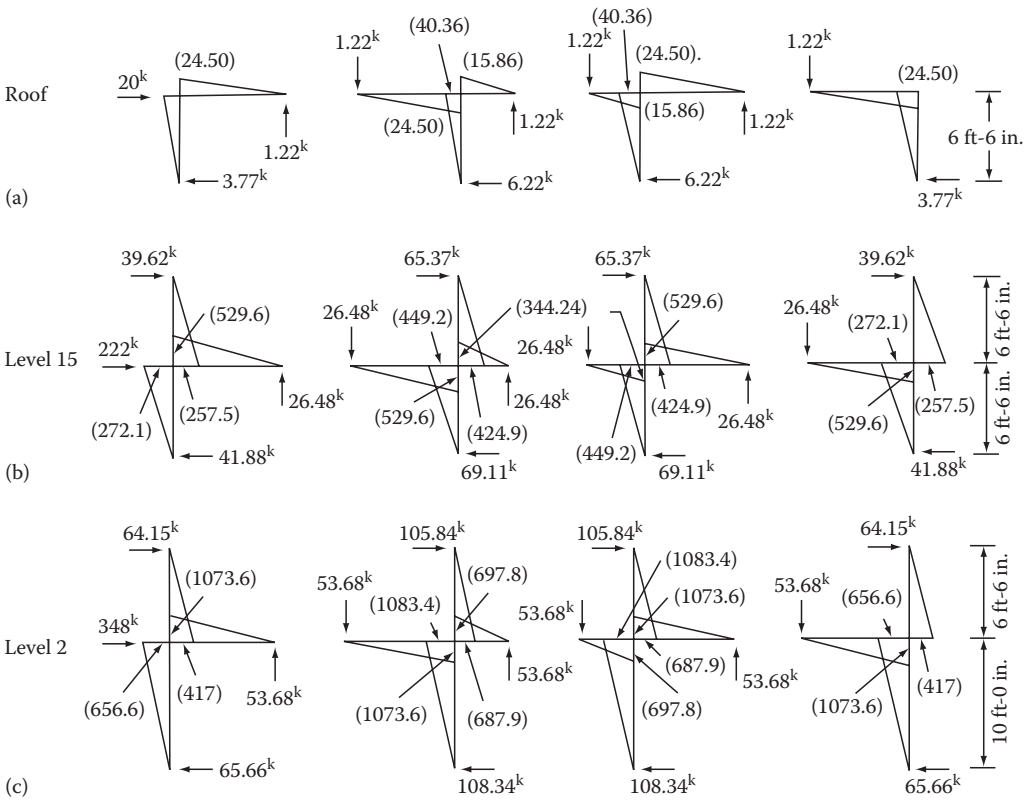


FIGURE 9.40 Portal method; moments and forces at (a) roof level; (b) level 15; (c) level 2. *Note*: Moments are in kip-ft.

Before the advent of computers, it was common practice to use the portal or cantilever method with some modifications for the final design of structures. The modifications not discussed in detail here consisted of a number of assumptions. Chief among them were the following:

1. Locate point of contraflexure in exterior girders at 0.55 of their length.
2. Locate the points of contraflexure in the bottom-story columns at 0.6 height from the base, and in the top-story columns at 0.65 height from the top.
3. Use of rather complicated rules to distribute the story shears among columns.

9.7.4 Framed Tubes

As mentioned earlier in Chapter 1, the framed tube system in its simplest form consists of closely spaced exterior columns tied at each floor by relatively deep spandrels. The behavior of the tube is in essence similar to that of a hollow perforated tube. The overturning moment due to lateral loads is resisted by compression and tension in the columns while the shear is resisted by the bending of columns and spandrels in the frames parallel to the direction of the lateral load. The forces and moments in these frames, also called web frames, can be evaluated using either the portal or the cantilever method of analysis. It is perhaps more accurate to use the cantilever method because tube systems are predominately used for tall buildings in the 40- to 80-story range in which the axial forces in the columns play a dominant role.

Because of the continuity of the structural system around corners of the building, the flange frames are also coaxed into resisting the overturning moment. Whether or not the entire flange

columns or only a portion thereof contribute to the tube action is a function of shear rigidity of the tube. For purposes of approximate analysis, the tube configuration may be considered as equivalent to two channels as shown in Figure 9.42(A)a and b. The assumed equivalent width of channel flange is subjected to engineering judgment and as a thumb rule is usually limited to 15%–20% of the width of the building. Although it is a function of the shear lag across the windward and leeward sides of the tube, the aforementioned thumb rule gives results sufficiently accurate for preliminary analysis.

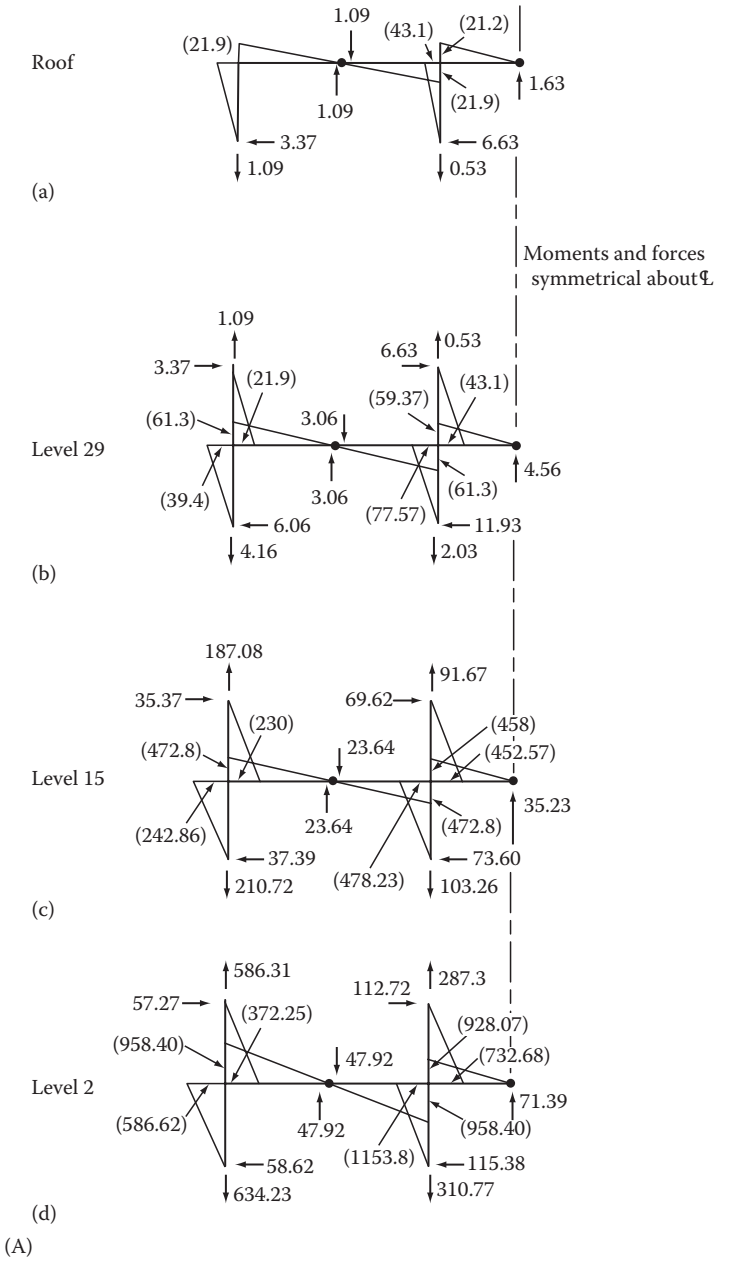


FIGURE 9.41 (A) Cantilever method; moments and forces at (a) roof level; (b) level 29; (c) level 15; (d) level 2. *Note*: Loads are in kips and moments in kip-ft.

(*continued*)

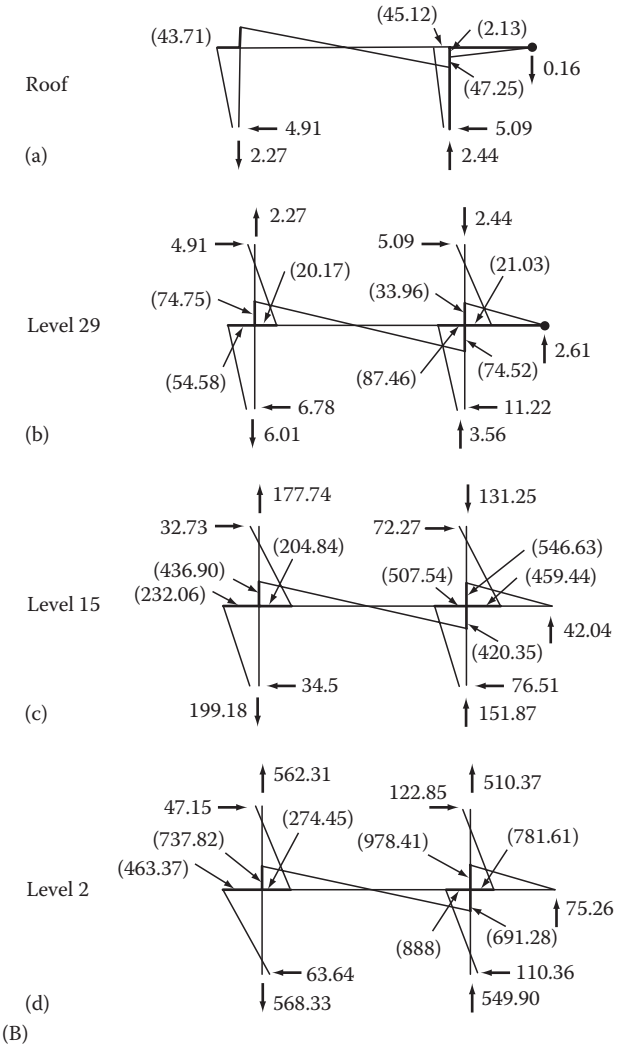


FIGURE 9.41 (continued) (B) 2D frame analysis, moments and forces at (a) roof level; (b) level 29; (c) level 15; and (d) level 2. *Note*: Loads are in kips and moments in kip-ft.

Shown in Figure 9.42(B) is the plan of a curvilinear framed tube delineating columns in the leeward and windward sides assumed as equivalent width of channels. The axial forces obtained on the basis of the equivalent structure are shown therein. For comparative purposes, the axial forces obtained from a 3D computer analysis are shown in Figure 9.42(C).

9.7.5 Vierendeel Truss

Consider a simply supported Vierendeel truss shown in Figure 9.43(A)a, carrying vertical loads at the top chord joints only. For purposes of preliminary analysis, let us assume that the loads are equal at the panel points. Determination of the preliminary shear, axial, and moment diagrams for the truss is the purpose of the section.

Because the loads are equal at panel points, the truss behaves much like a beam carrying a uniformly distributed vertical load (Figure 9.43(A)b). Further, by fixing the beam (Figure 9.43(A)c) at the point of zero slope (i.e., at mid-span of the beam), it can be shown the Vierendeel truss

symmetrically loaded by vertical loads behaves analogously to a cantilever rigid frame carrying lateral loads. Thus, the portal method can be used to determine the shear, axial, and moment diagrams by assuming hinges to form at mid-height of truss columns and mid-span of the top and bottom chords in each bay of the Vierendeel truss. It should be noted, however, that the equivalent cantilever is subjected simultaneously to vertical downward loads at the panel points, and a reaction at the support point acting vertically upward as shown in Figure 9.43c and d. (See deflection calculations, Figure 9.43e).

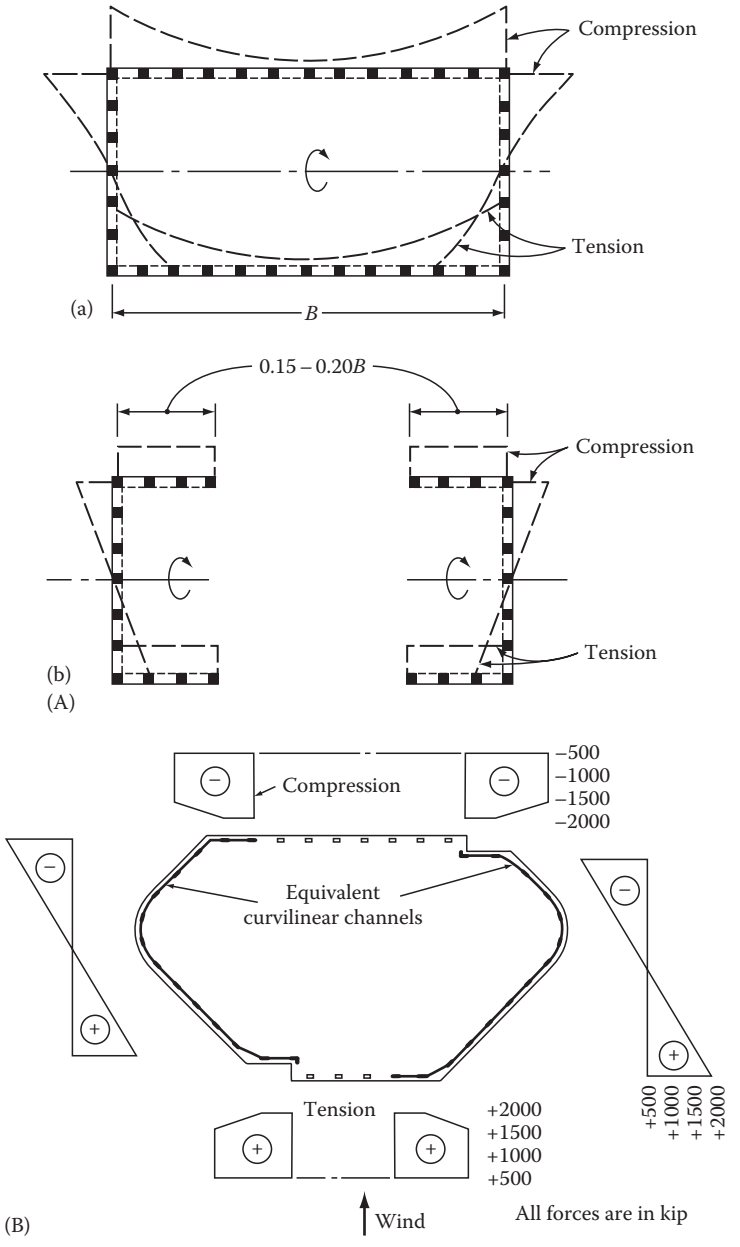


FIGURE 9.42 (A) Preliminary analysis of framed tube: (a) axial forces in columns using 3D analysis; (b) axial forces in columns using equivalent channels. (B) Curvilinear tube showing axial forces in columns assuming two equivalent channels.

(*continued*)

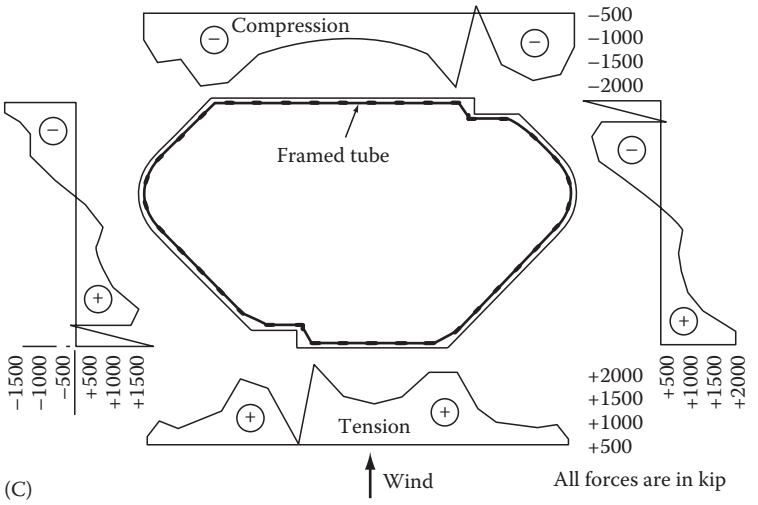


FIGURE 9.42 (continued) (C) Axial forces in columns from 3D analysis.

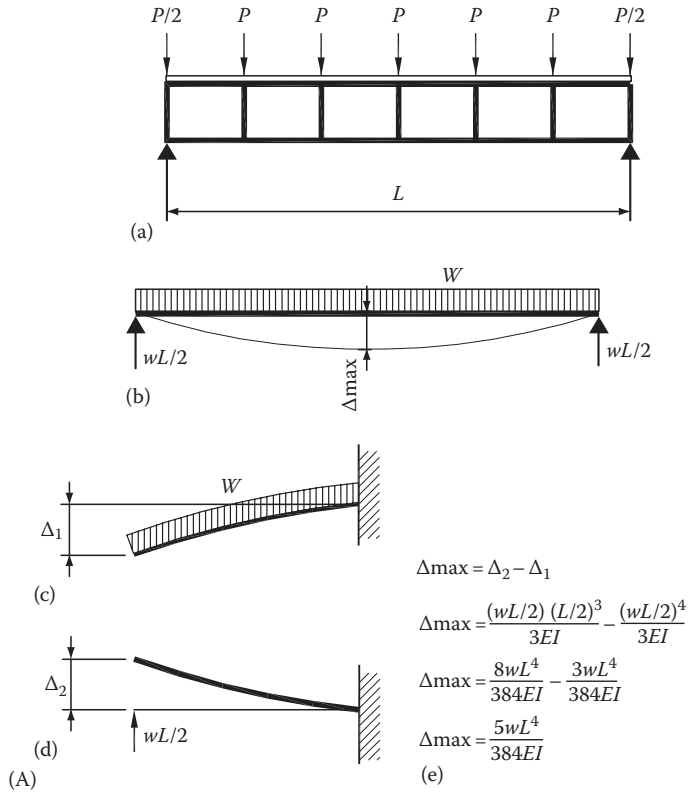


FIGURE 9.43 (A) Vierendeel truss preliminary analysis: (a) loads at panel points; (b) equivalent simple beam; (c) tip deflection due to uniformly distributed load; (d) tip deflection due to end reaction; (e) deflection calculations.

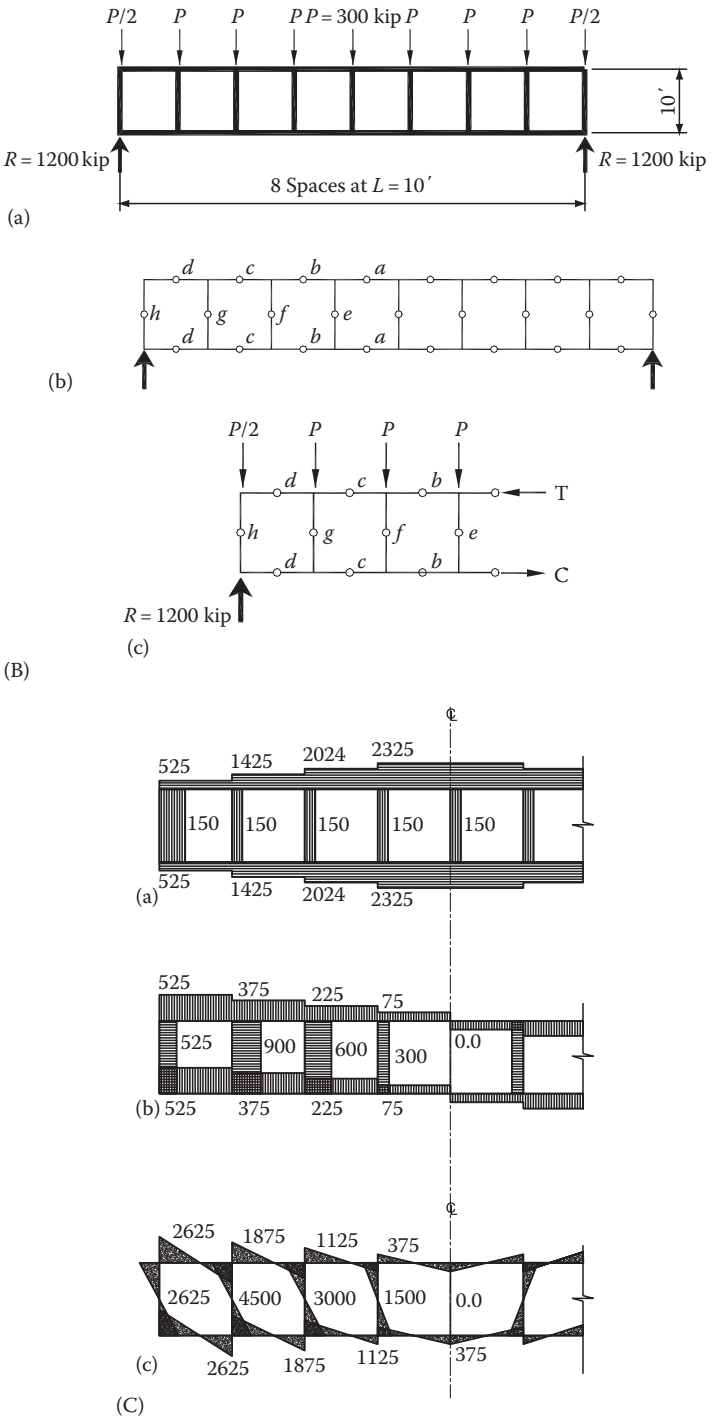


FIGURE 9.43 (continued) (B) (a) Truss geometry and loads; (b) points of contraflexure; (c) half-truss analytical model. (C) Schematics of flow of forces: (a) axial forces; (b) shear forces; (c) bending moments. *Note*: forces are in kips and bending moments in kip-ft.

To firm up our understanding of the behavior of Vierendeel truss, consider an 80 ft span, 15 ft deep truss shown in Figure 9.43(B)a, consisting of eight equal panels. Assume the load at the panel point, P, is equal to 300 kip.

Because of the symmetry in geometry and loading, only half the truss is considered. By cutting the free-body of the truss at mid-span of the chords, that is, at zero moment points, we ensure that only shear and axial forces will occur at that section.

$$\Sigma M_a = 1200 \times 35 - 150 \times 35 - 300 \times 25 - 300 \times 15 - 300 \times 5$$

$$= 23,250 \text{ kip-ft}$$

This moment is resisted by the equal and opposite axial tension and compression in the bottom and top chords, separated by the truss depth of 10 ft. Thus,

$$T = C = \Sigma M_a/10$$

$$= 23,250/10$$

$$= 2,325 \text{ kip (tension and compression)}$$

Vertical equilibrium yields the chord shear forces. At location "a"

$$\Sigma V = 0 = 1200 - 150 - 3(300) - 2V_4$$

$$V_4 = 75.0 \text{ kip}$$

A similar process is used to determine the shear and axial forces in other chord members. The shear and axial forces in the columns and bending moments in the top and bottom chords and columns are determined by equilibrium considerations. For this purpose, free-body diagrams cut at the assumed hinge locations are used. The results of the analysis along with schematic flow of forces are shown in Figure 9.43(c).

9.7.6 Preliminary Wind Loads

ASCE 7-05 Method 2, Reap

Velocity pressure q_z is given by

$$q_z = 0.0026 \times K_z \times K_{dz} K_{zt} \times V^2 \times I_w$$

Assuming $k_d = 0.85$, $K_{zt} = 1$ and $I_w = 1$

$$q_z = 0.0026 \times 0.85 \times K_z \times V^2$$

$$= 0.00221 \times K_z \times V^2$$

Shown in Figure 9.44 is a plot of the velocity pressure q_z up the building height versus the wind speed and Exposure Category. In this figure, the wind speed and Exposure Category are noted for each graph. For example, the notion 100c means that the graph refers to a 3 s gust of 100 mph in Exposure Category C.

The same information is given in Table 9.1 in a numerical format.

Using these figures, the engineer can quickly determine wind pressures for the design of main wind force resisting system (MWFRS) as illustrated in the following examples.

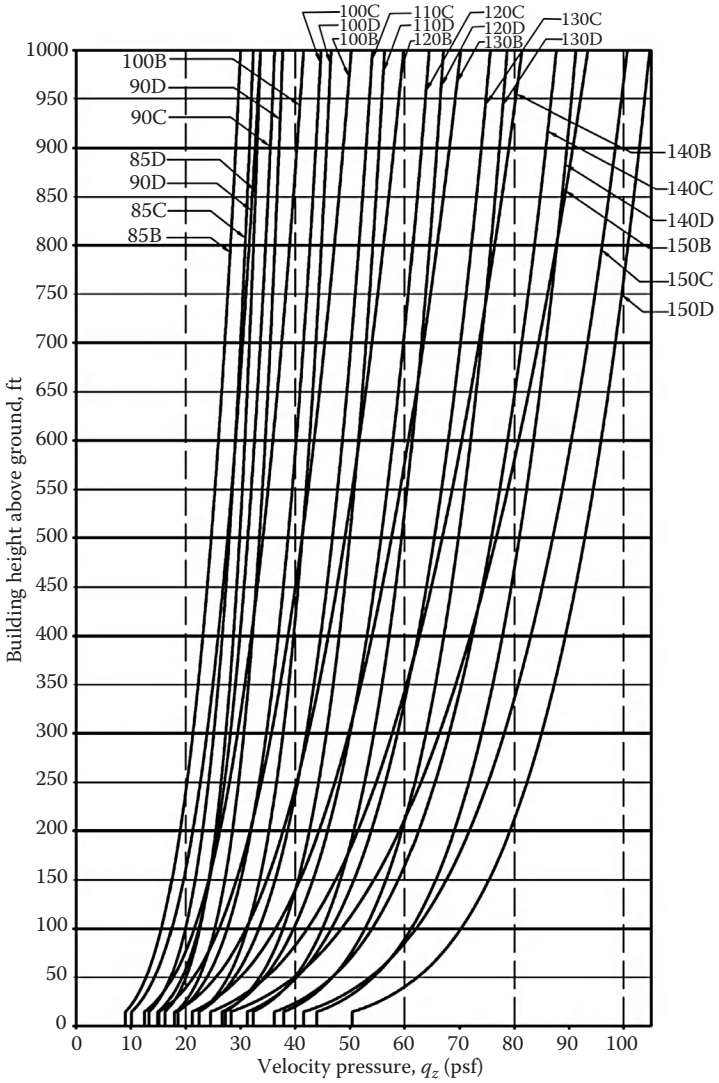


FIGURE 9.44 Variation of velocity pressure, q_z, versus wind speed and exposure categories. *Note*: $q_z = 0.00256K_zK_dK_{zt}V^2$, $K_d = 0.85$, $K_{zt} = 1.0$.

Shown in Tables 9.1 and 9.2 are values of K_z used in preparing the velocity pressures given in Figure 9.44.

Example 1

Given

A steel building located in a hurricane-prone region with the following characteristics:

- Building height = 450 ft (137.15 m)
- Building plan dimensions = 185 × 125 ft (56.396 × 38.10 m)
- Exposure Category = C
- Basic wind speed = 110 mph (49 m/s)
- Topographic factor $K_{zt} = 1.0$
- The building is for typical office occupancy. Therefore, $I_w = 1.0$
- Gust factor $G_f = 1.0$

TABLE 9.1
Variation of Velocity Pressure, q_z, versus Wind Speed and Exposure Categories (Numerical Values)

Building height above ground, ft	Velocity Pressure q_z (psf)											
	85B	85C	85D	90B	90C	90D	100B	100C	100D	110B	110C	110D
0 to 15	9.04	13.35	16.20	10.13	14.96	18.16	12.51	18.47	22.42	15.13	22.35	27.13
25.0	10.46	14.86	17.70	11.72	16.66	19.85	14.47	20.57	24.50	17.51	24.89	29.65
50.0	12.75	17.20	19.97	14.29	19.28	22.38	17.64	23.80	27.64	21.34	28.80	33.44
100.0	15.54	19.90	22.53	17.42	22.31	25.26	21.50	27.54	31.18	26.02	33.32	37.73
150.0	17.44	21.67	24.17	19.56	24.30	27.10	24.15	20.00	33.46	29.22	36.29	40.48
200.0	18.94	23.02	25.41	21.23	25.81	28.49	26.21	31.87	35.18	31.72	38.58	42.56
250.0	20.19	24.13	26.42	22.63	27.05	29.62	27.94	33.40	36.57	33.81	40.41	44.25
300.0	21.27	25.08	27.27	23.84	28.11	30.57	29.43	34.71	37.74	35.61	41.99	45.67
350.0	22.22	25.90	28.01	24.91	29.04	31.40	30.76	35.85	38.77	37.22	43.38	46.91
400.0	23.09	26.64	28.67	25.88	29.87	32.14	31.95	36.87	39.68	38.67	44.62	48.01
450.0	23.88	27.31	29.26	26.77	30.62	32.81	33.05	37.80	40.50	39.99	45.74	49.01
500.0	24.61	27.92	29.80	27.59	31.30	33.41	34.06	38.65	41.25	41.21	46.76	49.91
550.0	25.29	28.49	30.30	28.35	31.94	33.97	35.00	39.43	41.94	42.35	47.71	50.75
600.0	25.92	29.01	30.76	29.06	32.53	34.49	35.88	40.16	42.58	43.41	48.59	51.62
650.0	26.52	29.51	31.20	29.73	33.08	34.97	36.71	40.84	43.18	44.42	49.42	52.24
700.0	27.09	29.97	31.60	30.37	33.60	35.43	37.50	41.48	43.74	45.37	50.20	52.92
750.0	27.63	30.41	31.98	30.98	34.09	35.86	38.24	42.09	44.27	46.27	50.93	53.56
800.0	28.14	30.83	32.34	31.55	34.56	36.26	38.95	42.67	44.77	47.13	51.63	54.17
850.0	28.64	31.22	32.69	32.10	35.00	36.64	39.63	43.21	45.24	47.96	52.29	54.74
900.0	29.11	31.60	33.01	32.63	35.43	37.01	40.29	43.74	45.69	48.75	52.92	55.29
950.0	29.56	31.96	33.32	33.14	35.83	37.36	40.91	44.24	46.12	49.51	53.53	55.81
1000.0	30.00	32.31	33.62	33.63	36.22	37.69	41.52	44.72	46.54	50.24	54.11	56.31
	120B	120C	120D	130B	130C	130D	140B	140C	140D	150B	150C	150D
0 to 15	18.01	26.60	32.28	21.13	31.22	37.89	24.51	36.20	43.94	28.14	41.56	50.44
25.0	20.84	29.62	35.28	24.46	34.76	41.41	28.36	40.32	48.02	32.56	46.28	55.13
50.0	25.40	34.27	39.80	29.81	40.22	46.71	34.58	46.65	54.17	39.69	53.55	62.19
100.0	30.97	39.66	44.90	36.34	46.54	52.69	42.15	53.98	61.11	48.38	61.96	70.16
150.0	34.77	43.19	48.18	40.81	50.69	56.54	47.32	58.79	65.58	54.33	67.49	75.28
200.0	37.75	45.89	50.65	44.30	53.85	59.45	51.38	62.46	68.94	58.98	71.70	79.14
250.0	40.23	48.10	52.66	47.22	56.44	61.80	54.76	65.46	71.67	62.86	75.15	82.28
300.0	42.38	49.98	54.35	49.74	58.65	63.79	57.69	68.02	73.98	66.22	78.09	84.93
350.0	44.29	51.63	55.83	51.98	60.59	65.52	60.29	70.27	75.99	69.21	80.66	87.23
400.0	46.01	53.10	57.14	54.00	62.32	67.06	62.63	72.27	77.78	71.90	82.96	89.28
450.0	47.59	54.43	58.32	55.85	63.88	68.45	64.77	74.09	79.39	74.36	85.05	91.13
500.0	49.04	55.65	59.40	57.56	65.31	69.72	66.75	75.75	80.85	76.63	86.96	92.82
550.0	50.40	56.78	60.40	59.15	66.64	70.88	68.60	77.28	82.20	78.75	88.72	94.37
600.0	51.67	57.83	61.32	60.64	67.87	71.96	70.32	78.71	83.46	80.73	90.36	95.81
650.0	52.86	58.81	62.18	62.04	69.02	72.97	71.95	80.05	84.63	82.60	91.89	97.15
700.0	53.99	59.74	62.98	63.37	70.11	73.92	73.49	81.31	85.73	84.36	93.34	98.41
750.0	55.07	60.61	63.74	64.63	71.13	74.81	74.95	82.50	86.76	86.04	94.70	99.60
800.0	56.09	61.44	64.46	65.83	72.11	75.65	76.35	83.63	87.74	87.64	96.00	100.72
850.0	57.07	62.23	65.15	66.98	73.03	76.46	77.68	84.70	88.67	89.18	97.23	101.79
900.0	58.01	62.98	65.80	68.08	73.92	77.22	78.96	85.73	89.56	90.64	98.41	102.81
950.0	58.92	63.70	66.42	69.14	74.76	77.95	80.19	86.71	90.40	92.06	99.54	103.78
1000.0	59.79	64.39	67.01	70.16	75.57	78.65	81.37	87.65	91.21	93.41	100.62	104.71

Note: $q_z = 0.00256K_zK_dK_{zt}V^2$, $K_d = 0.85$, $K_{zt} = 1.0$.

TABLE 9.2
K_z Values for Buildings up to 1500 ft Tall

Velocity Pressure Exposure Coefficients, K_z[a,b]				
Height above Ground Level, *z*		**Exposure Category**		
ft	**(m)**	**B**	**C**	**D**
0–15	(0–4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89
550	(167.6)	1.61	1.81	1.93
600	(182.9)	1.65	1.85	1.96
650	(191.1)	1.69	1.88	1.98
700	(213.3)	1.72	1.91	2.01
750	(228.6)	1.76	1.93	2.03
800	(243.8)	1.79	1.96	2.06
850	(259.1)	1.82	1.99	2.08
900	(274.3)	1.85	2.01	2.10
950	(289.5)	1.88	2.03	2.12
1000	(304.8)	1.91	2.06	2.14
1050	(320)	1.93	2.08	2.16
1100	(335.3)	1.96	2.10	2.17
1150	(350.5)	1.99	2.12	2.19
1200	(365.7)	2.01	2.14	2.21
1250	(381)	2.03	2.15	2.22
1300	(396.2)	2.06	2.17	2.24
1350	(411.5)	2.08	2.19	2.26
1400	(426.7)	2.10	2.21	2.27

(*continued*)

TABLE 9.2 (continued)
K_z Values for Buildings up to 1500 ft Tall

Velocity Pressure Exposure Coefficients, K_z[a,b]				
Height above Ground Level, *z*		**Exposure Category**		
ft	**(m)**	**B**	**C**	**D**
1450	(441.9)	2.12	2.22	2.28
1500	(457.2)	2.14	2.24	2.29

Source: Adapted from Table 6.3 of ASCE 7-05.

[a] The velocity pressure exposure coefficient K_z may be determined from the following formula:
For 15 ft $\le z \le z_g$, $K_z = 2.01\ (z/z_g)^{2/\alpha}$.
For $z < 15$ ft, $K_z = 2.01\ (15/z_g)^{2/\alpha}$.

[b] All main wind-force-resisting systems in buildings and in other structures except those in low-rise buildings.

Required
Using the graph given in Figure 9.44 (or numerical values given in Table 9.1), determine design wind pressures for the MWFRS of the building for wind parallel to the short side.

Solution
From the building's plan dimensions, $L/B = 125/180 = 0.694 < 1.0$. Therefore, C_p for the windward face = 0.8, and C_p for the leeward face = −0.5. From Figure 9.44, select the curve identified as 110C. C stands for exposure C, and 110 stands for $V = 110$ mph. Use the graph (Figure 9.44) or the numerical table (Table 9.1) to determine the values of q_z at various heights. For example, at $h = 150$ ft, $q_z = 36.3$ psf.

Observe that in Figure 9.44 and Table 9.1, the q_z and q_h values are normalized for $K_{zt} = 1.0$, $K_d = 0.85$ and $I_w = 1.0$. Since $K_{zt} = 1.0$ and $I_w = 1.0$ as given in the statement of the problem, there is no need to modify the values given in Figure 9.44 or in Table 9.1.

Observe that the suction q_h referenced at roof height remains constant for the entire height of leeward wall.

Column 6, in Table 9.3 gives the total design wind pressure. It is the summation of $0.8q_z$, the positive pressure on the windward surface plus $0.5q_h$, the suction on the leeward surface.

Example 2

Given

- Building height 1000 ft
- Building plan dimensions = 270 × 270 ft
- Exposure Category = C
- Basic 3 s gust speed = 90 mph
- Topographic factor $K_{zt} = 1.0$
- The building is for typical occupancy; *t*herefore, $I_w = 1.0$
- Gust effect factor, G_f, based on exposure category, wind speed, building period, and damping = 1.15 (the reader may want to look up the publication "Steel, Concrete, & Composite Design of Tall Buildings" by the author for parametric studies of G_f)

Required
Using Figure 9.44 or Table 9.1 determine wind pressures for the design of the MWFRS of the building.

TABLE 9.3
Design Wind Pressures, Example 1

1	2	3	4	5	6
Height (ft)	K_z	K_h	q_z	q_h	Design Pressure $P = 0.8q_z + 0.5q_h$
450	1.74	1.74	45.74	45.74	59.46
400	1.69	1.74	44.62	45.74	58.57
350	1.65	1.74	43.38	45.74	57.57
300	1.59	1.74	42.00	45.74	56.47
250	1.53	1.74	40.41	45.74	55.20
200	1.46	1.74	38.58	45.74	53.73
150	1.38	1.74	36.29	45.74	51.90
100	1.26	1.74	33.32	45.74	49.53
50	1.09	1.74	28.80	45.74	45.91
30	0.98	1.74	24.90	45.74	42.80
15	0.85	1.74	22.35	45.74	40.75

Solution
From the building plan dimensions, L/B = 270/270 = I. Therefore, C_p for windward surface is 0.8, and 0.5 for the leeward surface. From Figure 9.44, we select the column with the heading C90. C is the Exposure Category and 90 mph is the 3 s gust speed as specified in the problem. Observe that 90 mph is the 3 s gust speed that applies to most of United States and C is the default exposure category that may be used in most designs.

The design pressures are obtained from Table 9.1 in exactly the same manner as for the previous problem. The only difference is we multiply the design pressures by the gust factor equal to 1.15 as stated at the beginning of the problem. The results are shown in Table 9.4.

TABLE 9.4
Design Wind Pressures, Example 2

1	2	3	4	5	Design Pressure
Height (ft)	K_z	K_h	q_z	q_h	$P = G_f(0.8q_h + 0.5q_z)$
1000	2.06	2.06	36.22	36.22	54.15
900	2.01	2.06	35.43	36.22	53.42
800	1.96	2.06	34.56	36.22	52.62
700	1.91	2.06	33.60	36.22	51.74
600	1.85	2.06	32.53	36.22	50.75
500	1.77	2.06	31.30	36.22	49.62
400	1.69	2.06	29.87	36.22	48.31
300	1.59	2.06	28.11	36.22	46.69
200	1.46	2.06	25.81	36.22	44.57
100	1.26	2.06	22.31	36.22	41.35
50	1.09	2.06	19.28	36.22	38.56
25	0.94	2.06	16.66	36.22	36.15
15	0.85	2.06	14.96	36.22	34.59

Note: G_f = 1.15 as given in the statement of the problem.

TABLE 9.5
Design Wind Pressures, Example 3

1	2	3	4	5	Design Pressure
Height (ft)	K_z	K_h	q_z	q_h	$P = G_f(0.8q_h + 0.5q_z)$
1000	2.06	2.06	100.62	100.62	156.97
900	2.01	2.06	98.41	100.62	154.85
800	1.96	2.06	96	100.62	152.53
700	1.91	2.06	93.34	100.62	150.0
600	1.85	2.06	90.36	100.62	147.12
500	1.77	2.06	86.96	100.62	143.85
400	1.69	2.06	82.96	100.62	140.0
300	1.59	2.06	78.09	100.62	135.34
200	1.46	2.06	71.70	100.62	129.20
100	1.26	2.06	61.96	100.62	148.65
50	1.09	2.06	53.35	100.62	111.59
25	0.94	2.06	46.28	100.62	104.80
15	0.85	2.06	41.56	100.62	100.27

Note: $G_f = 1.20$ as given in the statement of the problem.

Example 3

Given

- A building of height 1000 ft located in coastal region subject to hurricane winds. The building has the same dynamic characteristics as the one in Example 2.
- Building plan dimensions = 270 × 270 ft.
- Exposure Category C.
- Basic 3 s gust speed of 150 mph.
- Topographic factor $K_{zt} = 1.0$.
- The building is for typical office occupancy. Therefore, $I_w = 1.0$.
- Gust effect factor = 1.2. Observe that although the building has the same dynamic characteristics as that in Example 2, it has a higher gust factor because the design wind speed is a lot higher. (Determination of gust factors is not discussed here. The reader is referred to the reference cited earlier.)

Required
Wind pressures for the design of the MWFRS of the building.

Solution
As for Example 2, $C_p = 0.8$ for the windward face and is equal to 0.5 for the leeward surface. We select the column with the heading 150D to calculate the wind loads. The results are shown in Table 9.5.

9.7.7 Preliminary Seismic Loads

We start this section by asking ourselves a question "what would be the approximate base shear for a Seismic Design Category (SDC) D, steel building with a fundamental period of say 1.25 s, located in downtown San Diego, in terms of its seismic weight *W*? And what would be the base shear if the building was located in any of the other 48 cities in USA, shown in Figure 9.45".

Answers to a general question like this would be of great assistance in preliminary designs and in ensuring that we are in the right ball park with our computer results. Additionally, they provide

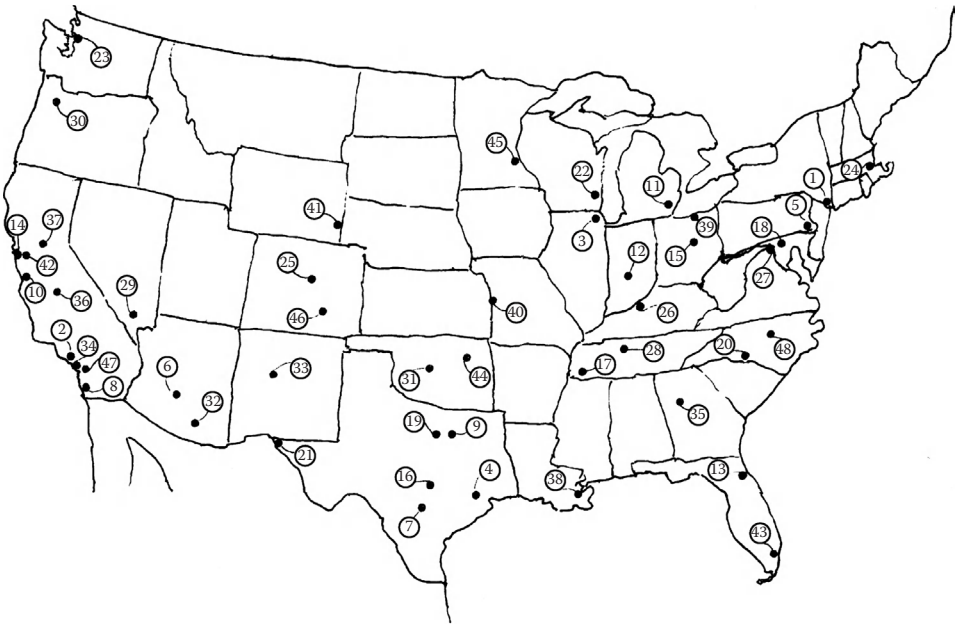


FIGURE 9.45 Locations of selected 48 U.S. cities cited in Tables 9.6, 9.7 and Figure 9.47.

an effective way of catching flawed analysis that may have resulted from computer modeling procedures. These answers would be particularly beneficial to graduates entering the structural engineering profession, and to peer reviewers eyeballing designs using the proverbial back-of-the-envelope procedures.

The question we posed for ourselves, however, is deceptively simple because meaningful answers would require consideration of additional factors such as:

1. What is the seismic-force-resisting system of the building?
2. Is the building height within the limits given in ASCE Table 12.2-1 for the assigned SDC of the building?
3. What is the site class of the soil on which the building rests upon?
4. What is the value for the seismic response modification factor, R?

Because the building's dynamic characteristics has an impact on the base shear, it is unwieldy at best, if not impossible to respond to the question in general terms without addressing the dynamic properties of the building. The only recourse, it seems, would be to particularize the base shear using certain assumptions for the height and other dynamic characteristics of the building, and then to adjust it as required for the given building.

The base shear values calculated for selected 48 cities in the US are given in Table 9.6 for site class D, and Table 9.7 for site class C.

The assumptions used in preparing the tables are as follows:

1. Building height is 160 ft corresponding to a 12-story office building at approximately 13 ft floor-to-floor height, or a 16-story residential building with a 10 ft floor-to-floor height. We have chosen 160 ft as the height because, with the exception of only a few structural systems (see ASCE 7-05, Table 12.2-1 for exceptions), all other systems are permitted up to this height for buildings assigned to SDC D or lower.

TABLE 9.6
Approximate Base Shear as a Percent of Building Weight: Site Class D

#	City	Map Location (Latitude, Longitude)	Site Class	S_S	S_1	F_a	F_v	S_{MS} (g)	S_{MT} (g)	S_{DS} (g)	S_{D1} (g)	SDC	T_L, Long-Period Transition (Informational Purposes)	C_uT_A	C_s, Base Shear as a Percentage of Building Weight	#
1	New York, New York	40° 47′ N 73° 58′ W	D	0.364	0.070	1.509	2.400	0.549	0.169	0.366	0.113	C	6	1.51	3.06%	1
2	Los Angeles, California	34° 3′ N 118° 14′ W	D	2.176	0.729	1.000	1.500	2.176	1.093	1.451	0.729	D	12	1.26	19.92%	2
3	Chicago, Illinois	41° 53′ N 87° 38′ W	D	0.162	0.059	1.600	2.400	0.260	0.142	0.173	0.095	B	12	1.53	2.57%	2
4	Houston, Texas	29° 59′ N 95° 22′ W	D	0.090	0.038	1.600	2.400	0.144	0.091	0.096	0.060	A	12	1.53	1.33%	4
5	Philadelphia, Pennsylvania	39° 53′ N 75° 15′ W	D	0.266	0.059	1.587	2.400	0.422	0.142	0.282	0.094	B	6	1.53	2.57%	5
6	Phoenix, Arizona	33° 26′ N 112° 1′ W	D	0.180	0.061	1.600	2.400	0.288	0.147	0.192	0.098	B	6	1.53	2.66%	6
7	San Antonio, Texas	29° 32′ N 98° 28′ W	D	0.103	0.030	1.600	2.400	0.165	0.073	0.110	0.049	A	6	1.53	1.33%	7
8	San Diego, California	32° 44′ N 117° 10′ W	D	1.573	0.613	1.000	1.500	1.573	0.919	1.049	0.613	D	8	1.26	16.75%	8
9	Dallas, Texas	32° 51′ N 96° 51′ W	D	0.118	0.050	1.600	2.400	0.188	0.121	0.125	0.081	B	12	1.53	2.19%	9
10	San Jose, California	37° 22′ N 121° 56′ W	D	1.500	0.600	1.000	1.500	1.500	0.900	1.000	0.600	D	8	1.26	16.4%	10
11	Detroit, Michigan	42° 25′ N 83° 1′ W	D	0.118	0.044	1.600	2.400	0.190	0.106	0.126	0.070	B	12	1.53	1.92%	11
12	Indianapolis, Indiana	39° 44′ N 86° 17′ W	D	0.203	0.086	1.600	2.400	0.325	0.207	0.217	0.138	C	12	1.46	3.76%	12
13	Jacksonville, Florida	30° 30′ N 81° 42′ W	D	0.151	0.064	1.600	2.400	0.242	0.152	0.161	0.102	B	8	1.53	2.80%s	13
14	San Francisco, California	37° 46′ N 122° 26′ W	D	1.502	0.731	1.000	1.500	1.502	1.097	1.001	0.731	D	12	1.26	19.8%	14
15	Columbus, Ohio	40° 0′ N 82° 53′ W	D	0.145	0.058	1.600	2.400	0.232	0.140	0.155	0.093	B	12	1.53	2.53%	15
16	Austin, Texas	30° 18′ N 97° 42′ W	D	0.082	0.033	1.600	2.400	0.131	0.079	0.087	0.053	B	12	1.53	1.33%	16
17	Memphis, Tennessee	35° 3′ N 90° 0′ W	D	1.130	0.316	1.048	1.768	1.184	0.559	0.790	0.372	D	12	1.26	10.17%	17
18	Baltimore, Maryland	39° 11′ N 76° 40′ W	D	0.163	0.050	1.600	2.400	0.261	0.121	0.174	0.081	B	6	1.53	2.19%	18
19	Fort Worth, Texas	32° 50′ N 97° 3′ W	D	0.14	0.049	1.600	2.400	0.183	0.118	0.122	0.079	B	12	1.53	2.19%	19
20	Charlotte, North Carolina	35° 13′ N 80° 56′ W	D	0.318	0.106	1.546	2.377	0.491	0.251	0.328	0.168	C	8	1.41	4.59%	20
21	El Paso, Texas	31° 48′ N 106° 24′ W	D	0.336	0.108	1.531	2.367	0.514	0.256	0.343	0.171	C	6	1.40	4.65%	21
22	Milwaukee, Wisconsin	42° 57′ N 87° 54′ W	D	0.110	0.045	1.600	2.400	0.176	0.109	0.117	0.073	B	12	1.53	1.97%	22
23	Seattle, Washington	47° 39′ N 122° 18′ W	D	1.306	0.444	1.000	1.556	1.306	0.691	0.870	0.460	D	6	1.26	12.59%	23
24	Boston, Massachusetts	42° 22′ N 71° 2′ W	D	0.282	0.068	1.574	2.400	0.445	0.163	0.296	0.109	B	6	1.51	3.11%	24

25	Denver, Colorado	39° 45′ N 104° 52′ W	D	0.210	0.055	1.600	2.400	0.337	0.133	0.225	0.089	B	4	1.53	2.4%	25
26	Louisville-Jefferson County, Kentucky	38° 11′ N 85° 44′ W	D	0.247	0.103	1.600	2.389	0.395	0.245	0.263	0.164	C	12	1.41	4.48%	26
27	Washington, District of Columbia	38° 51′ N 77° 2′ W	D	0.153	0.050	1.600	2.400	0.244	0.121	0.163	0.081	B	8	1.53	2.19%	27
28	Nashville-Davidson, Tennessee	36° 7′ N 86° 41′ W	D	0.331	0.129	1.535	2.283	0.508	0.295	0.339	0.197	C	12	1.36	5.37%	28
29	Las Vegas, Nevada	36° 5′ N 115° 10′ W	D	0.554	0.171	1.357	2.115	0.752	0.362	0.501	0.241	D	6	1.31	6.59%	29
30	Portland, Oregon	45° 36′ N 122° 36′ W	D	0.907	0.317	1.137	1.767	1.031	0.559	0.687	0.373	D	16	1.26	10.2%	30
31	Oklahoma City, Oklahoma	35° 24′ N 97° 36′ W	D	0.354	0.076	1.516	2.400	0.537	0.182	0.358	0.121	C	12	1.49	3.32%	31
32	Tucson, Arizona	32° 7′ N 110° 56′ W	D	0.282	0.080	1.575	2.400	0.444	0.192	0.296	0.128	B	6	1.48	3.49%	32
33	Albuquerque, New Mexico	35° 3′ N 106° 37′ W	D	0.571	0.172	1.343	2.112	0.767	0.363	0.511	0.242	D	6	1.31	6.61%	33
34	Long Beach, California	33° 49′ N 118° 9′ W	D	1.693	0.644	1.000	1.500	1.693	0.966	1.129	0.644	D	8	1.26	17.60%	34
35	Atlanta, Georgia	33° 39′ N 84° 26′ W	D	0.218	0.084	1.600	2.400	0.348	0.202	0.232	0.134	C	12	1.47	3.67%	35
36	Fresno, California	36° 46′ N 119° 43′ W	D	0.488	0.219	1.410	1.962	0.688	0.430	0.459	0.287	D	12	1.27	7.83%	36
37	Sacramento, California	38° 31′ N 121° 30′ W	D	0.629	0.251	1.297	1.898	0.816	0.477	0.544	0.318	D	12	1.26	8.68%	37
38	New Orleans, Louisiana	29° 59′ N 90° 15′ W	D	0.111	0.048	1.600	2.400	0.177	0.115	0.118	0.077	B	12	1.53	2.09%	38
39	Cleveland, Ohio	41° 24′ N 81° 51′ W	D	0.179	0.051	1.600	2.400	0.289	0.123	0.191	0.082	B	12	1.53	2.23%	39
40	Kansas City, Missouri	39° 7′ N 94° 35′ W	D	0.127	0.058	1.600	2.400	0.204	0.140	0.136	0.094	B	12	1.53	2.53%	40
41	Omaha, Nebraska	41° 18′ N 95° 54′ W	D	0.118	0.042	1.600	2.400	0.188	0.100	0.125	0.066	B	12	1.53	1.84%	41
42	Oakland, California	37° 49′ N 122° 19′ W	D	1.500	0.600	1.000	1.500	1.500	0.900	1.000	0.600	D	8	1.26	16.4%	42
43	Miami, Florida	25° 48′ N 80° 16′ W	D	0.051	0.020	1.600	2.400	0.082	0.047	0.055	0.031	A	8	1.53	1.33%	43
44	Tulsa, Oklahoma	36° 12′ N 95° 54′ W	D	0.159	0.067	1.600	2.400	0.255	0.160	0.170	0.107	B	12	1.52	2.93%	44
45	Minneapolis, Minnesota	44° 53′ N 93° 13′ W	D	0.060	0.027	1.600	2.400	0.096	0.065	0.064	0.043	A	12	1.53	1.33%	45
46	Colorado Springs, Colorado	38° 49′ N 104° 43′ W	D	0.203	0.058	1.600	2.400	0.324	0.139	0.216	0.093	B	4	1.53	2.53%	46
47	Irvine, California	33° 40′ N 117° 47′ W	D	1.476	0.522	1.000	1.500	1.476	0.783	0.984	0.522	D	8	1.26	14.39%	47
48	Raleigh, North Carolina	35° 52′ N 78° 47′ W	D	0.202	0.079	1.600	2.400	0.323	0.189	0.215	0.126	B	8	1.48	3.45%	48

Note: h_o = 160ft. I_E = 1.0. T_a = 1.22s (see text). Site Class = D (Stiff soil, see ASCE 7-05, Chapter 20), R = 3.0. For systems with differing R values, multiply base shear by the ratio $\left(\frac{3.0}{R}\right)$.

TABLE 9.7
Approximate Base Shear as a Percent of Building Weight: Site Class C

#	City	Map Location (Latitude, Longitude)	Site Class	S_S	S_1	F_a	F_v	S_{MS} (*g*)	S_{M1} (*g*)	S_{DS} (*g*)	S_{D1} (*g*)	SDC	T_L, Long-Period Transition (Informational Purposes)	C_uT_A	C_s, Base Shear as a Percentage of Building Weight	#
1	New York, New York	40° 47′N 73° 58′W	C	0.364	0.070	1.200	1.700	0.437	0.120	0.291	0.080	B	6	1.53	2.17%	1
2	Los Angeles, California	34° 3′N 118° 14′W	C	2.176	0.729	1.000	1.300	2.176	0.948	1.451	0.632	D	12	1.26	17.27%	2
3	Chicago, Illinois	41° 53′N 87° 38′W	C	0.162	0.059	1.200	1.700	0.195	0.101	0.130	0.067	A	12	1.53	1.33%	3
4	Houston, Texas	29° 59′N 95° 22′W	C	0.090	0.038	1.200	1.700	0.108	0.064	0.072	0.043	A	12	1.53	1.33%	4
5	Philadelphia, Pennsylvania	39° 53′N 75° 15′W	C	0.266	0.059	1.200	1.700	0.319	0.100	0.213	0.067	B	6	1.53	1.82%	5
6	Phoenix, Arizona	33° 26′N 112° 1′W	C	0.180	0.061	1.200	1.700	0.216	0.104	0.144	0.069	B	6	1.53	1.88%	6
7	San Antonio, Texas	29° 32′N 98° 28′W	C	0.103	0.030	1.200	1.700	0.124	0.052	0.083	0.034	A	6	1.53	1.33%	7
8	San Diego, California	32° 44′N 117° 10′W	C	1.573	0.613	1.000	1.300	1.573	0.796	1.049	0.531	D	8	1.26	14.52%	8
9	Dallas, Texas	32° 51′N 96° 51′W	C	0.118	0.050	1.200	1.700	0.141	0.086	0.094	0.057	A	12	1.53	1.33%	9
10	San Jose, California	37° 22′N 121° 56′W	C	1.500	0.600	1.000	1.300	1.500	0.780	1.000	0.520	D	8	1.26	14.21%	10
11	Detroit, Michigan	42° 25′N 83° 1′W	C	0.118	0.044	1.200	1.700	0.142	0.075	0.095	0.050	A	12	1.53	1.33%	11
12	Indianapolis, Indiana	39° 44′N 86° 17′W	C	0.203	0.086	1.200	1.700	0.244	0.146	0.163	0.098	B	12	1.53	2.66%	12

13	Jacksonville, Florida	30° 30′N 81° 42′W	C	0.151	0.064	1.200	1.700	0.181	0.108	0.121	0.072	B	8	1.53	1.98%	13
14	San Francisco, California	37° 46′N 122° 26′W	C	1.502	0.731	1.000	1.300	1.502	0.951	1.001	0.634	D	12	1.26	17.26%	14
15	Columbus, Ohio	40° 0′N 82° 53′W	C	0.145	0.058	1.200	1.700	0.174	0.099	0.116	0.066	A	12	1.53	1.33%	15
16	Austin, Texas	30° 18′N 97° 42′W	C	0.082	0.033	1.200	1.700	0.098	0.056	0.065	0.037	A	12	1.53	1.33%	16
17	Memphis, Tennessee	35° 3′N 90° 0′W	C	1.130	0.316	1.000	1.484	1.130	0.469	0.753	0.313	D	12	1.26	8.55%	17
18	Baltimore, Maryland	39° 11′N 76° 40′W	C	0.163	0.050	1.200	1.700	0.196	0.086	0.130	0.057	A	6	1.53	8.27%	18
19	Fort Worth, Texas	32° 50′N 97° 3′W	C	0.114	0.049	1.200	1.700	0.137	0.083	0.091	0.056	A	12	1.53	3.33%	19
20	Charlotte, North Carolina	35° 13′N 80° 56′W	C	0.318	0.106	1.200	1.694	0.381	0.179	0.254	0.119	B	8	1.49	1.33%	20
21	El Paso, Texas	31° 48′N 106° 24′W	C	0.336	0.108	1.200	1.692	0.403	0.183	0.269	0.122	B	6	1.49	3.3%	21
22	Milwaukee, Wisconsin	42° 57′N 87° 54′W	C	0.110	0.045	1.200	1.700	0.132	0.077	0.088	0.051	A	12	1.53	1.33%	22
23	Seattle, Washington	47° 39′N 122° 18′W	C	1.306	0.444	1.000	1.356	1.306	0.602	0.870	0.401	D	6	1.26	10.96%	23
24	Boston, Massachusetts	42° 22′N 71° 2′W	C	0.282	0.068	1.200	1.700	0.339	0.116	0.226	0.077	B	6	1.53	2.11%	24
25	Denver, Colorado	39° 45′N 104° 52′W	C	0.210	0.055	1.200	1.700	0.253	0.094	0.168	0.063	B	4	1.53	1.71%	25
26	Louisville-Jefferson County, Kentucky	38° 11′N 85° 44′W	C	0.247	0.103	1.200	1.697	0.296	0.174	0.197	0.116	B	12	1.50	3.19%	26
27	Washington, District of Columbia	38° 51′N 77° 2′W	C	0.153	0.050	1.200	1.700	0.183	0.086	0.122	0.057	A	8	1.53	1.33%	27
	Nashville Davidson, Tennessee	36° 7′N 86° 41′W		0.331	0.129	1.200	1.671	0.397	0.216	0.265	0.144	C	12	1.45	3.92%	28

(*continued*)

TABLE 9.7 (continued)
Approximate Base Shear as a Percent of Building Weight: Site Class C

#	City	Map Location (Latitude, Longitude)	Site Class	S_S	S_1	F_a	F_v	S_{MS} (*g*)	S_{M1} (*g*)	S_{DS} (*g*)	S_{D1} (*g*)	SDC	T_L, Long-Period Transition (Informational Purposes)	C_uT_A	C_s, Base Shear as a Percentage of Building Weight	#
29	Las Vegas, Nevada	36° 5′N 115° 10′W	C	0.554	0.171	1.178	1.629	0.653	0.279	0.435	0.186	C	6	1.38	5.08%	29
30	Portland, Oregon	45° 36′N 122° 36′W	C	0.907	0.317	1.037	1.483	0.940	0.469	0.627	0.313	D	16	1.26	8.56%	30
31	Oklahoma City, Oklahoma	35° 24′N 97° 36′W	C	0.354	0.076	1.200	1.700	0.425	0.129	0.284	0.086	B	12	1.53	2.36%	31
32	Tucson, Arizona	32° 7′N 110° 56′W	C	0.282	0.080	1.200	1.700	0.338	0.136	0.225	0.091	B	6	1.53	2.48%	32
33	Albuquerque, New Mexico	35° 3′N 106° 37′W	C	0.571	0.172	1.172	1.628	0.669	0.280	0.446	0.187	C	6	1.37	5.11%	33
34	Long Beach, California	33° 49′N 118° 9′W	C	1.693	0.644	1.000	1.300	1.693	0.837	1.129	0.558	D	8	1.26	15.25%	34
35	Atlanta, Georgia	33° 39′N 84° 26′W	C	0.218	0.084	1.200	1.700	0.261	0.143	0.174	0.095	B	12	1.53	2.6%	35
36	Fresno, California	36° 46′N 119° 43′W	C	0.488	0.219	1.200	1.581	0.585	0.346	0.390	0.231	D	12	1.32	6.31%	36
37	Sacramento, California	38° 31′N 121° 30′W	C	0.629	0.251	1.148	1.549	0.722	0.389	0.481	0.259	D	12	1.30	7.08%	37

38	New Orleans, Louisiana	29° 59′N 90° 15′W	C	0.111	0.048	1.200	1.700	0.133	0.081	0.089	0.054	A	12	1.53	1.33%	38
39	Cleveland, Ohio	41° 24′N 81° 51′W	C	0.179	0.051	1.200	1.700	0.214	0.087	0.143	0.058	A	12	1.53	1.33%	39
40	Kansas City, Missouri	39° 7′N 94°35′W	C	0.127	0.058	1.200	1.700	0.153	0.099	0.102	0.066	A	12	1.53	1.33%	40
41	Omaha, Nebraska	41° 18′N 95° 54′W	C	0.118	0.042	1.200	1.700	0.141	0.071	0.094	0.047	A	12	1.53	1.33%	41
42	Oakland, California	37° 49′N 122° 19′W	C	1.500	0.600	1.000	1.300	1.500	0.780	1.000	0.520	D	8	1.26	14.21%	42
43	Miami, Florida	25° 48′N 80° 16′W	C	0.051	0.020	1.200	1.700	0.062	0.033	0.041	0.022	A	8	1.53	1.33%	43
44	Tulsa, Oklahoma	36° 12′N 95° 54′W	C	0.159	0.067	1.200	1.700	0.191	0.113	0.127	0.076	B	12	1.53	2.08%	44
45	Minneapolis, Minnesota	44° 53′N 93° 13′W	C	0.060	0.027	1.200	1.700	0.072	0.046	0.048	0.031	A	12	1.53	1.33%	45
46	Colorado Springs, Colorado	38° 49′N 104° 43′W	C	0.203	0.058	1.200	1.700	0.243	0.099	0.162	0.056	A	4	1.53	1.33%	46
47	Irvine, California	33° 40′N 117° 47′W	C	1.476	0.522	1.000	1.300	1.476	0.679	0.984	0.452	D	8	1.26	12.36%	47
48	Raleigh, North Carolina	35° 52′N 78° 47′W	C	0.202	0.079	1.200	1.700	0.242	0.134	0.161	0.089	B	8	1.53	2.44%	48

Note: h_n = 160ft. l_E = 1.0. T_a = 1.22s (see text). Site Class = C (Very dense soil and soft rock, see ASCE 7-05, Chapter 20) R = 3.0. For Systems with differing R values, multiply base shear by the ratio $\left(\frac{3.0}{R}\right)$.

2. The period T_a also denoted as T_A in Tables 9.6 and 9.7 is calculated using ASCE 7-05, Section 12.8.2.1, Table 12.8-2 as follows:
   a. For steel moment frame

$$T_a = C_t(h_n)^x$$

$$= 0.28 \times (160)^{0.8} = 1.62 \text{ s}$$

   b. For eccentrically braced steel frame

$$T_a = 0.03 \times (160)^{0.75} = 1.34 \text{ s}$$

   c. For all other structural systems

$$T_a = 0.02 \times (160)^{0.75} = 0.90 \text{ s}$$

   Since the lateral-load-resisting system for the building is presumably not known at this preliminary stage, we use the average $T_a = (1.62 + 1.34 + 0.9)/3 = 1.286$ s
   However, we use a slightly conservative value of 1.22 s.
3. Site Class C or D as noted in Tables 9.6 and 9.7.
4. Response modification coefficient, $R = 3$. Observe that AISC 341-05 provisions are not intended to apply to buildings with R equal to 3 or less. These not-so-ductile systems are permitted in SDC A, B or C but not in SDC D, E or F.
5. Importance factor $I_E = 1.0$.

9.7.7.1 Building Height, H_n = 160 ft

Example 1

Given
A 160 ft tall building in Boston, sited at the map location given in item # 24, in Table 9.7. The building has an R equal to 3 and is located on Site Class C.

Required
Base shear as a percent of building weight *W*.

Solution
Because the site class is C as stated for the example building, we select Table 9.7. In item #24 of the table, the base shear is given as a percent of the building weight *W* and is equal to 2.11%. Observe the building period used in the calculation of base shear is equal to $C_u T_a$. See ASCE 7-05 Table 12.8-1 for values of coefficient C_u.

Example 2

Given
A 160 ft tall steel building with special steel concentrically brace frames. (See ASCE 7-05, Table 12.2-1, and item B3). The building is located on Site Class D, in San Diego, California.

Required
The base shear as a percent of building weight, *W*.

Solution
Immediately, we notice our building is in seismic country, California, with an assigned SDC D. A special steel concentrically braced frame is permitted up to a height of 160 ft. And because

AISC Seismic Provisions mandate stringent detailing requirements for this system, the seismic response modification coefficient, R, is equal to 6 instead of 3 that was used in developing Tables 9.6 and 9.7.

From Table 9.6, item #8, the required base shear $V = 16.75\%$ W. However, as stated previously, the base shears in the tables are normalized using $R = 3$. Therefore, base shear for our building is equal to $V = (16.75 \times 3/6)\%$ W $= 8.38\%$ W.

9.7.7.2 Buildings Taller than 160 ft

It is clear from previous discussion that the base shear tables (Tables 9.6 and 9.7) are good for buildings of height $h_n = 160$ ft (Figure 9.40). If h_n is considerably different from 160 ft, we are out of luck, we cannot use the tables. The only recourse we have, it seems, is to revert back to using generalized response spectrum, discussed at length in Chapter 5. For this purpose, shown in Figure 9.47 are the response spectra for the 48 selected cities in the USA (see Figure 9.45).

Example 3

Given

A steel building of height h_n equal to 300 ft with a dual system as seismic-force-resisting system. It consists of special moment frames capable of resisting at least 25% of seismic forces, and buckling-restrained braced frame. The corresponding $R = 8$. The building is situated in Los Angeles, California, on a site classified as D, located at the intersection of latitude 34° 3′ N and 110° 14′ W.

Required

Base shear as a percent of building's seismic weight, W, using the response spectra given in Figure 9.46.

Solution

$$h_n = 300 \text{ ft (given in the statement of the example)}$$

Calculate T_a as follows:

$$T_a = 0.03(h_n)^{0.75}$$

$$= 0.03\,(300)^{0.75}$$

$$= 2.16 \text{ s}$$

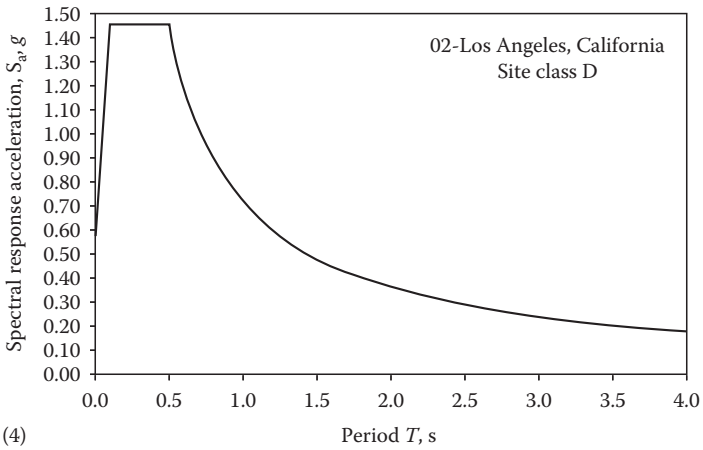


FIGURE 9.46 Design response spectrum for a selected site in downtown Los Angeles.

From AISC 7-05 Table 12.8-1

$$\text{For } S_{D1} = 0.729 > 0.4$$

$C_u = 1.4$, and the adjusted period for determination of base shear = $C_u T_a = 1.4 \times 2.16 = 3$ s.

Using the response spectrum for the given site in Los Angeles, site class D, shown in Figure 9.46.

For $C_u T_a = 3$ s, $S_a = 0.25$ g
Using $R = 8$, $V = 0.25/8 = 0.0313$ g
In other words, $V = 3.13\%$ W

Granted, the procedure is somewhat trivial once the response spectrum is known. However, the response spectrum curves shown for site class D and C, in Figure 9.47 are expected to assit engineers in their preliminary designs.

9.7.8 Differential Shortening of Columns

Columns in tall buildings experience large axial displacements because they are relatively long and accumulate gravity loads from a large number of floors. A 60-story column of a steel building, for example, may shorten as much as 2 to 3 in. (50 to 76 mm) at the top. If such a shortening is not given due consideration, problems may develop in providing level floors and assuring trouble-free performance of building cladding systems. Proper awareness of this problem is necessary on the part of structural engineer, architect, and curtain wall supplier to avoid unwelcome arguments, lost time, and money.

The maximum shortening of a column occurs at the roof level, reducing to zero at the base. Very little can be done to minimize the physical phenomenon of shortening, but the design team should be aware of the problem particularly at the building exterior, so that soft joints are provided if need be, between the building frame and cladding to prevent axial load being unwittingly transferred in to the building's façade. Before fabrication of cladding, the in-place elevations of structural frame should be verified, and if required, the cladding should be manufactured to fit the existing field condition of the steel frame. The design should provide for sufficient space between the cladding panels to allow for the movement of the structure. Insufficient space may result in bowed cladding components or, in extreme cases, the cladding panels may even pop put of the building.

A similar problem occurs when mechanical and plumbing lines are attached rigidly to the structure. Frame shortening may force the pipes to act as structural columns resulting in their distress. A general remedy is to make sure that nonstructural elements are not brought in to bear vertical loads by separating them from the structural elements.

The axial loads in all columns of a building are seldom the same, giving rise to the problem of so-called differential shortening. The problem is more acute in a composite structure because steel erection columns that are later encased in concrete are typically slender and are therefore subject to large axial loads during construction. Determining the magnitude of axial shortening in a composite system is complicated because many of the variables that contribute to the shortening cannot be predicted with sufficient accuracy. Consider, for example, the lower part of the composite column that is continually undergoing creep. The steel erection column during construction is partly enclosed in concrete at the lower floors, with the bar steel column projecting several floors above the concreted level. Another factor difficult to predict is the gravity load redistribution due to frame action of columns and, if the building is founded on compressible material, the settlement of the foundation is another factor that influences the relative changes in the elevation of the columns. The magnitude of load imbalance between any two columns is continually changing, making an accurate assessment of column shortening rather a challenging task.

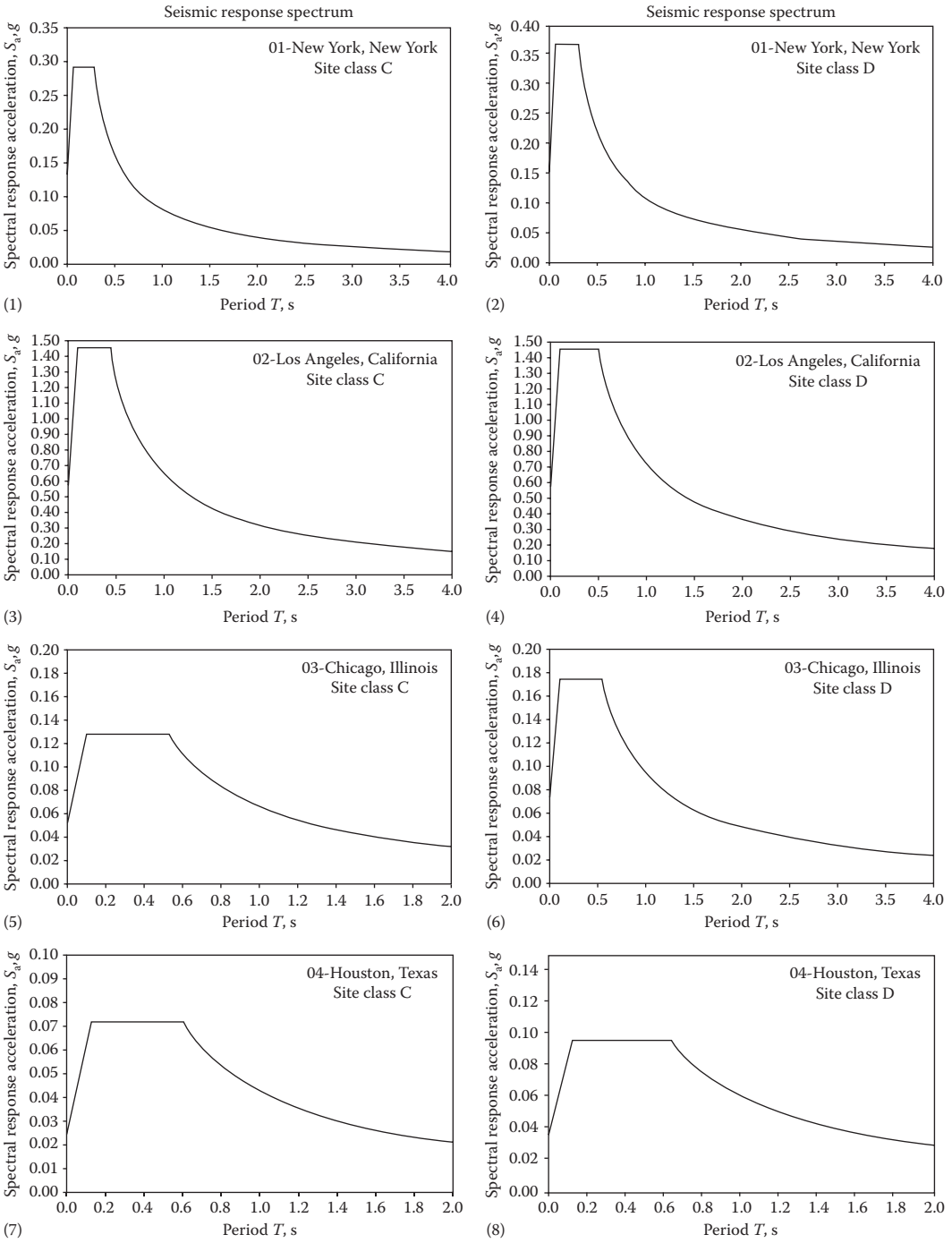


FIGURE 9.47 Design response spectrum for selected 48 U.S. cities.

(*continued*)

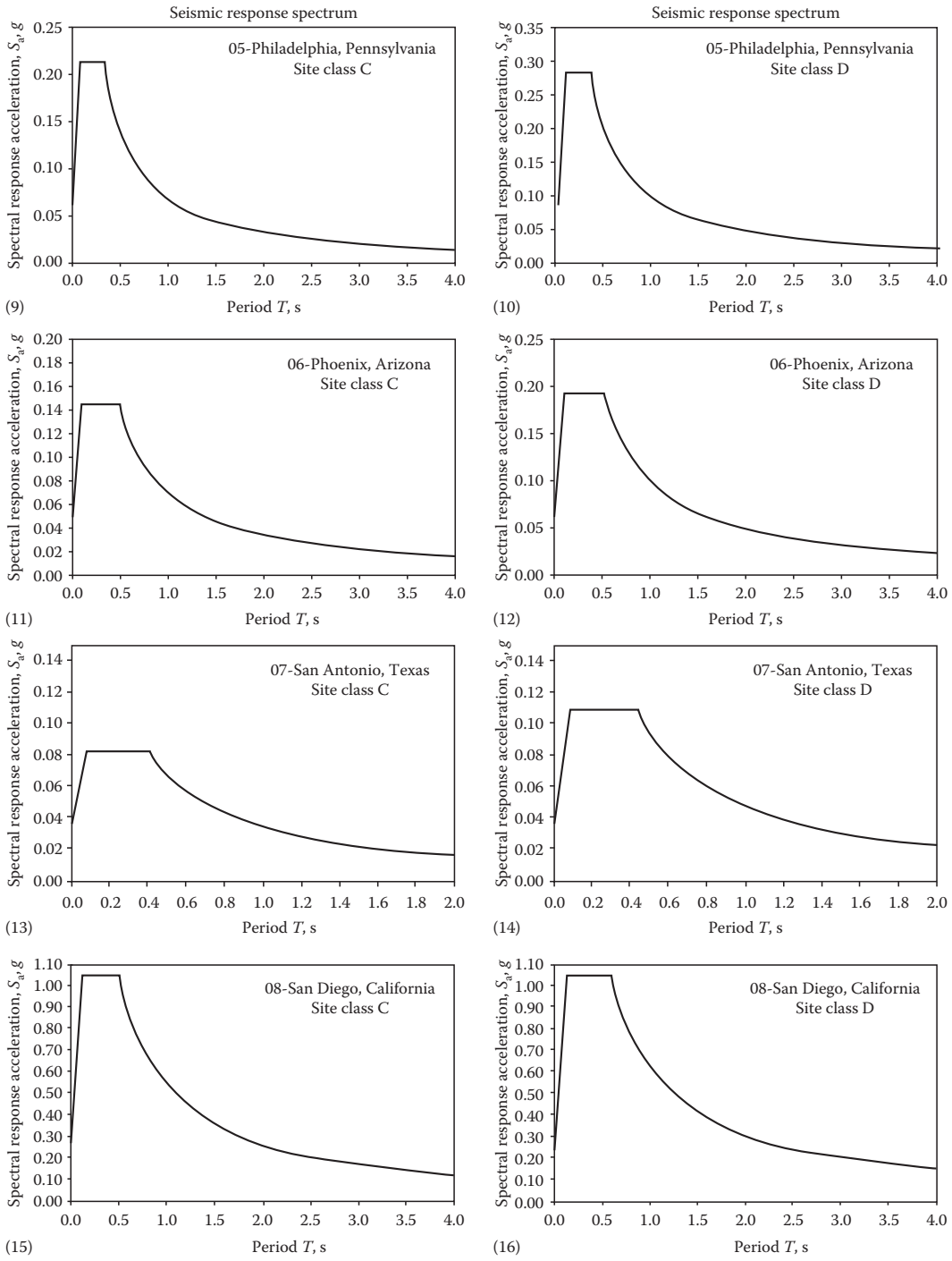


FIGURE 9.47 (continued)

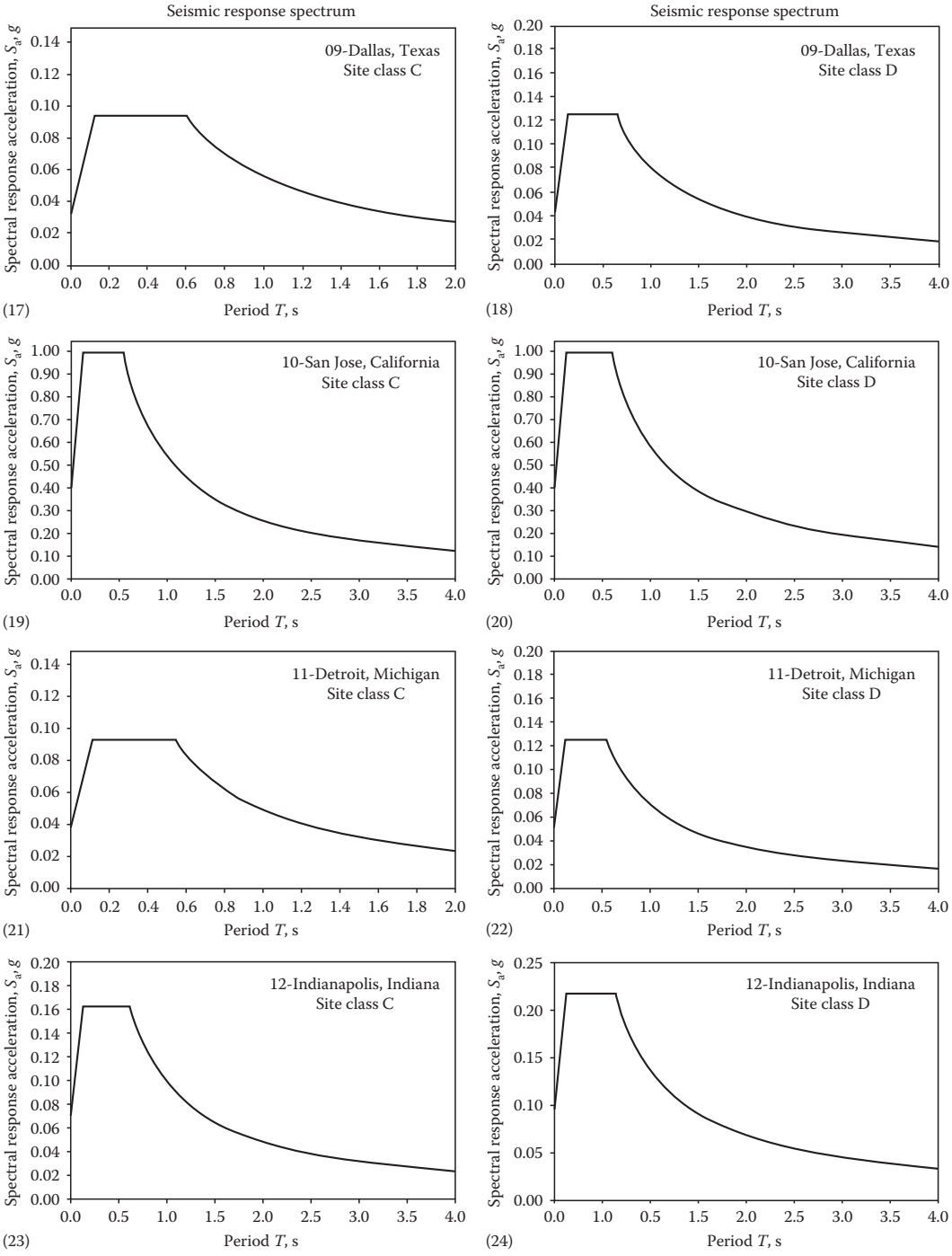


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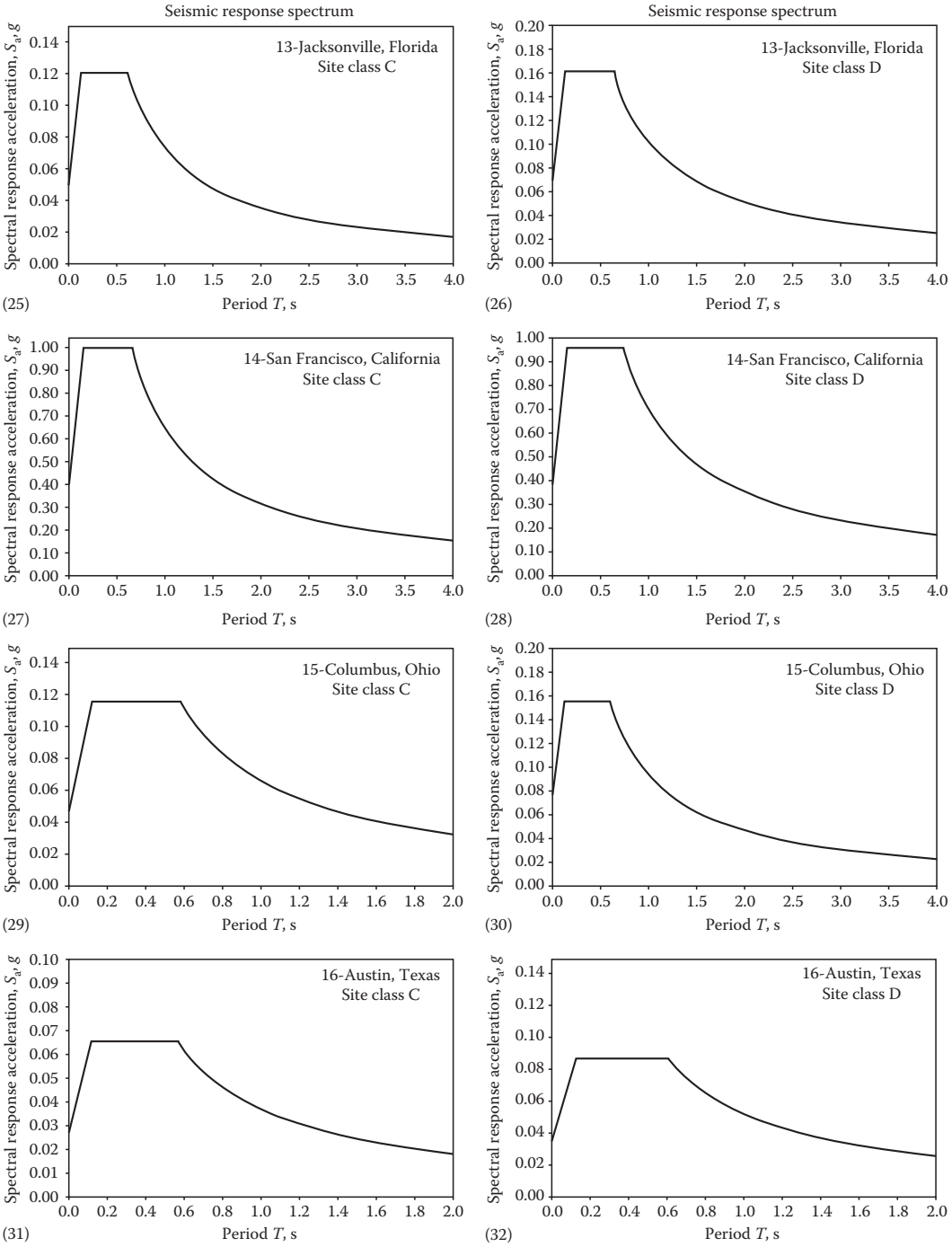


FIGURE 9.47 (continued)

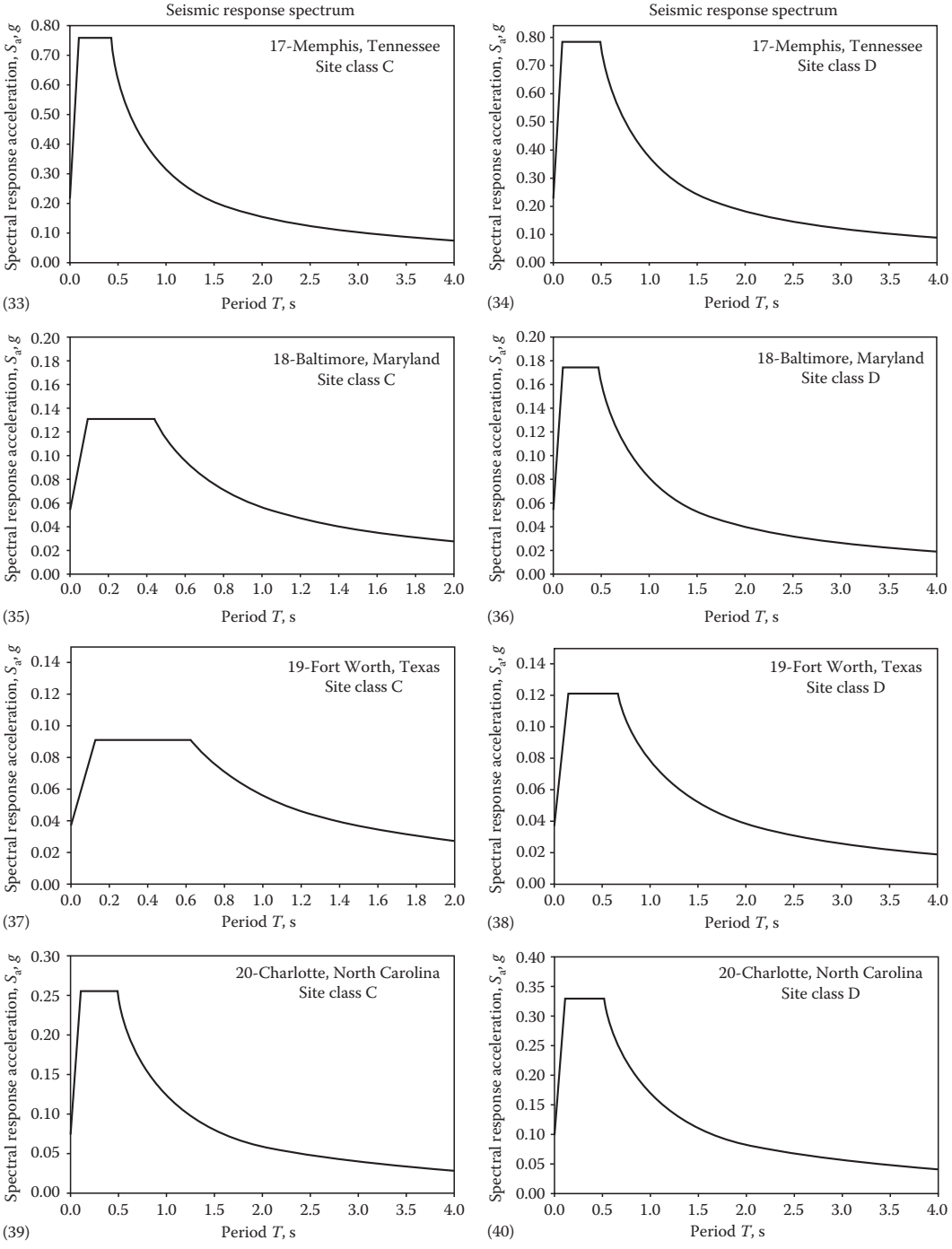


FIGURE 9.47 (continued)

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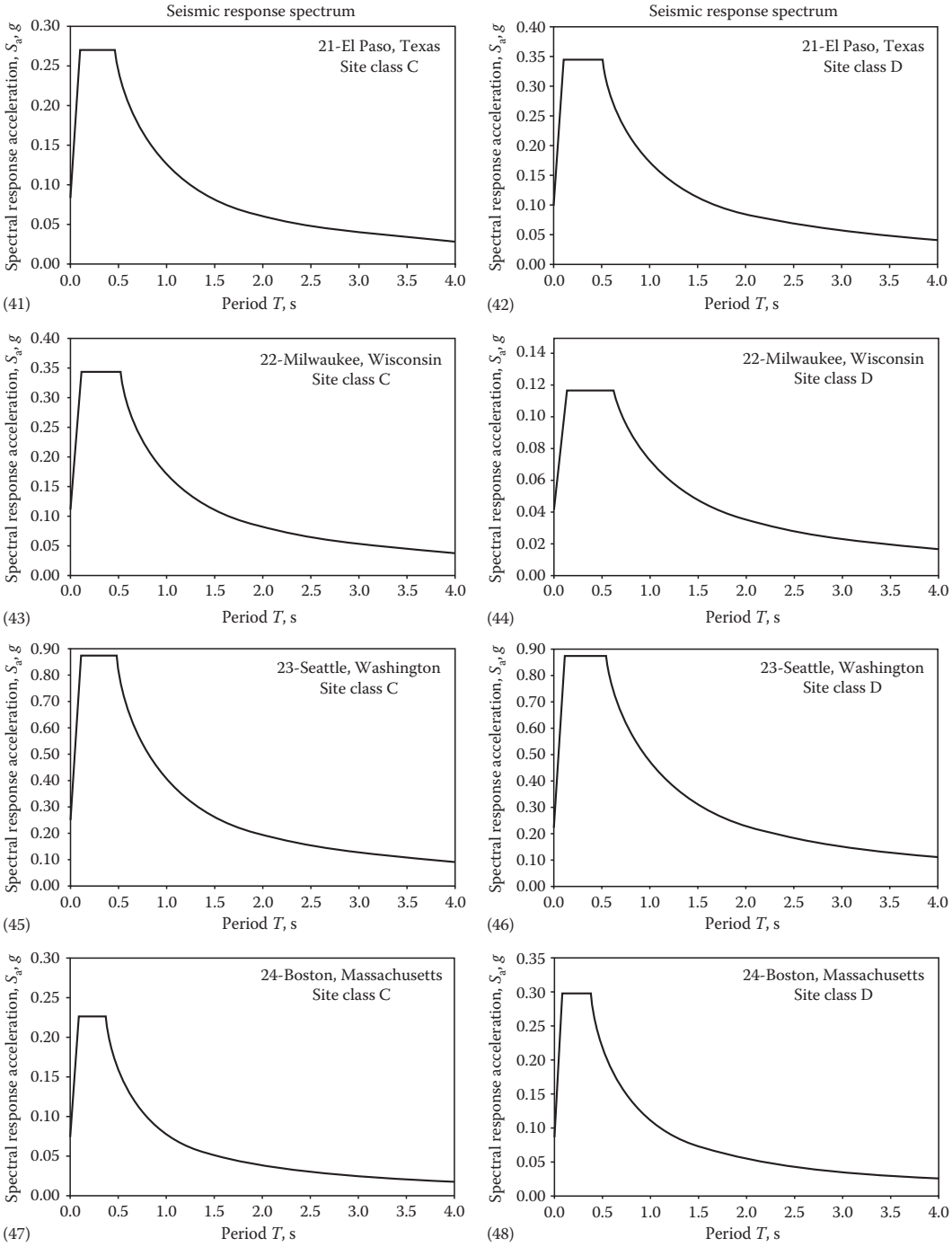


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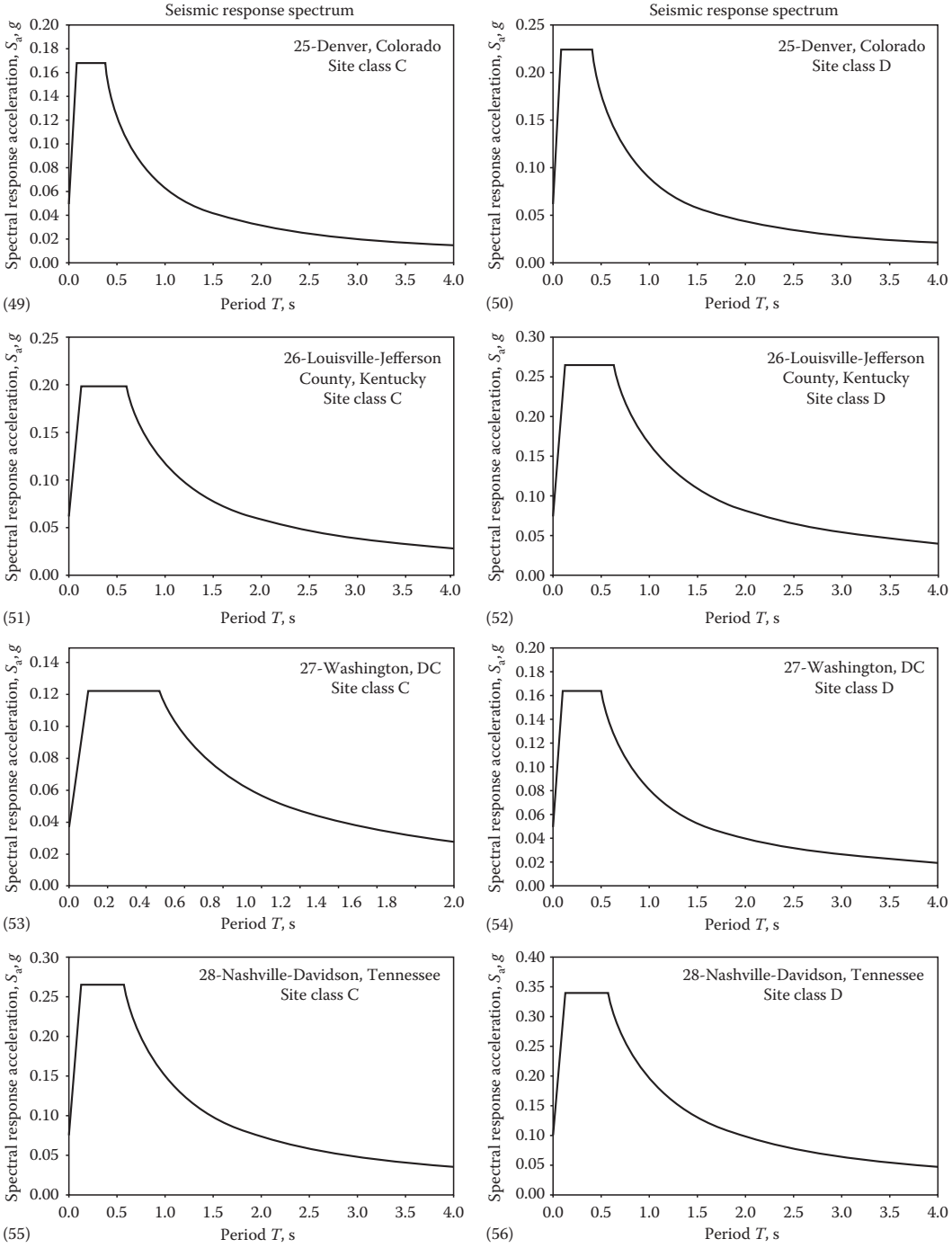


FIGURE 9.47 (continued)

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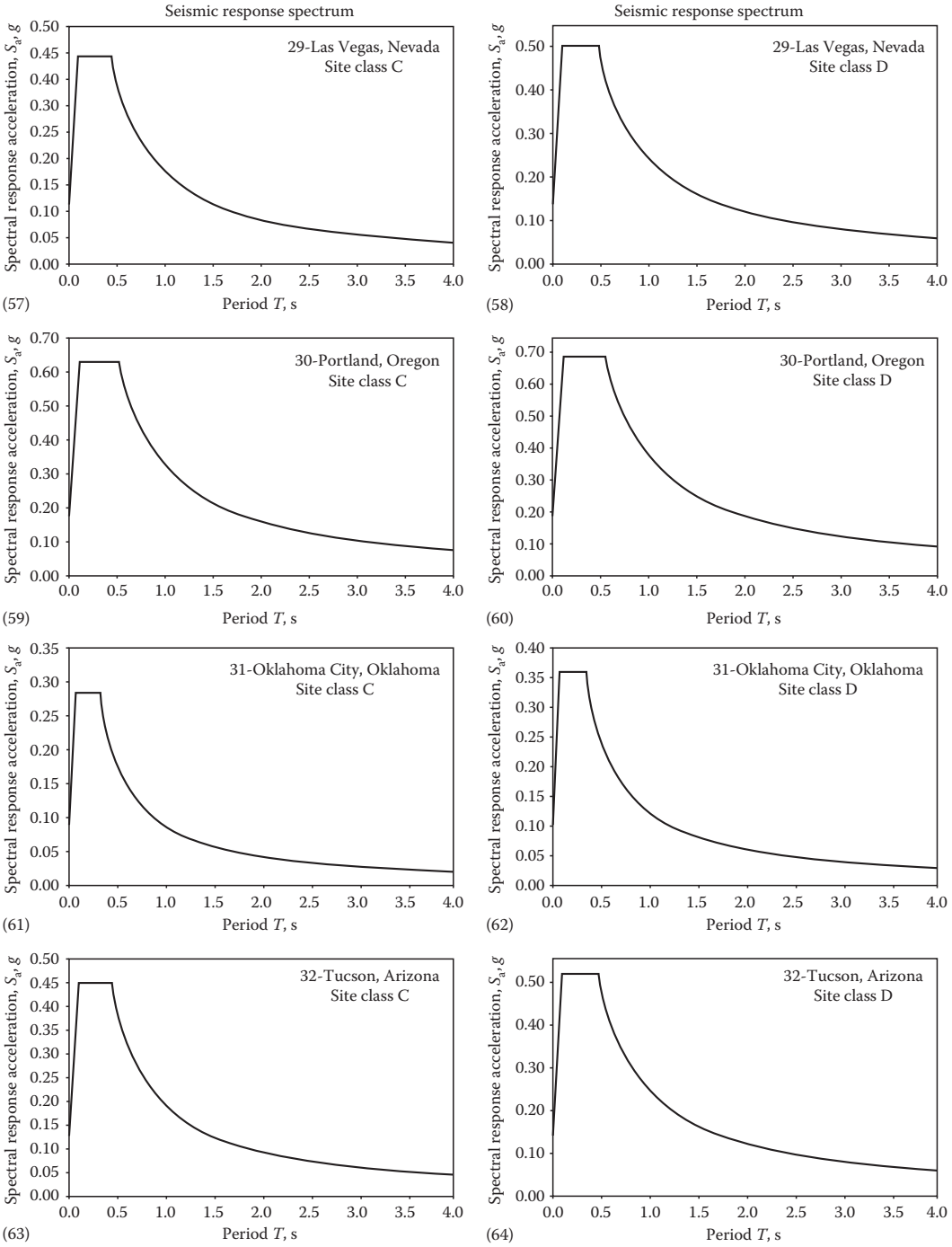


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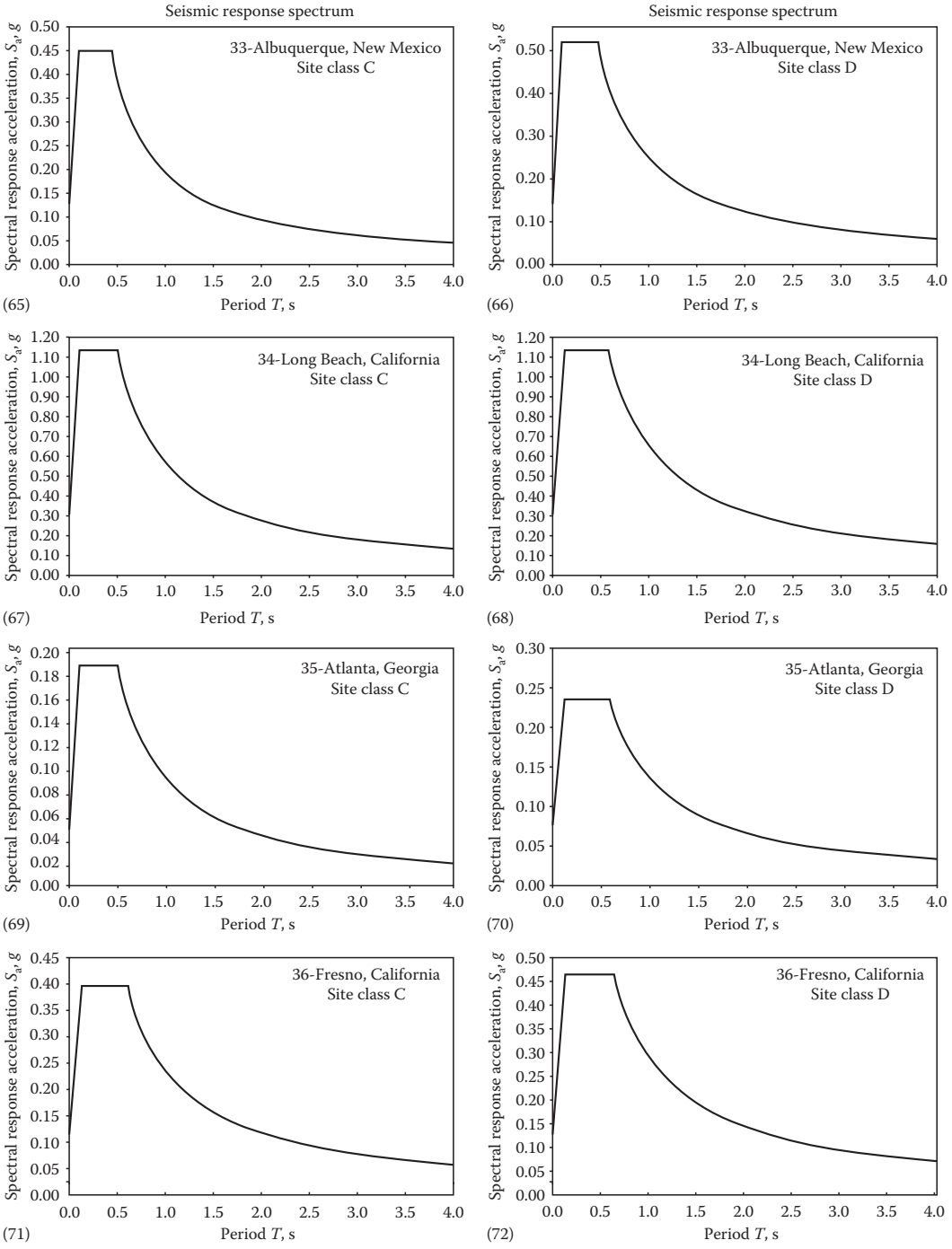


FIGURE 9.47 (continued)

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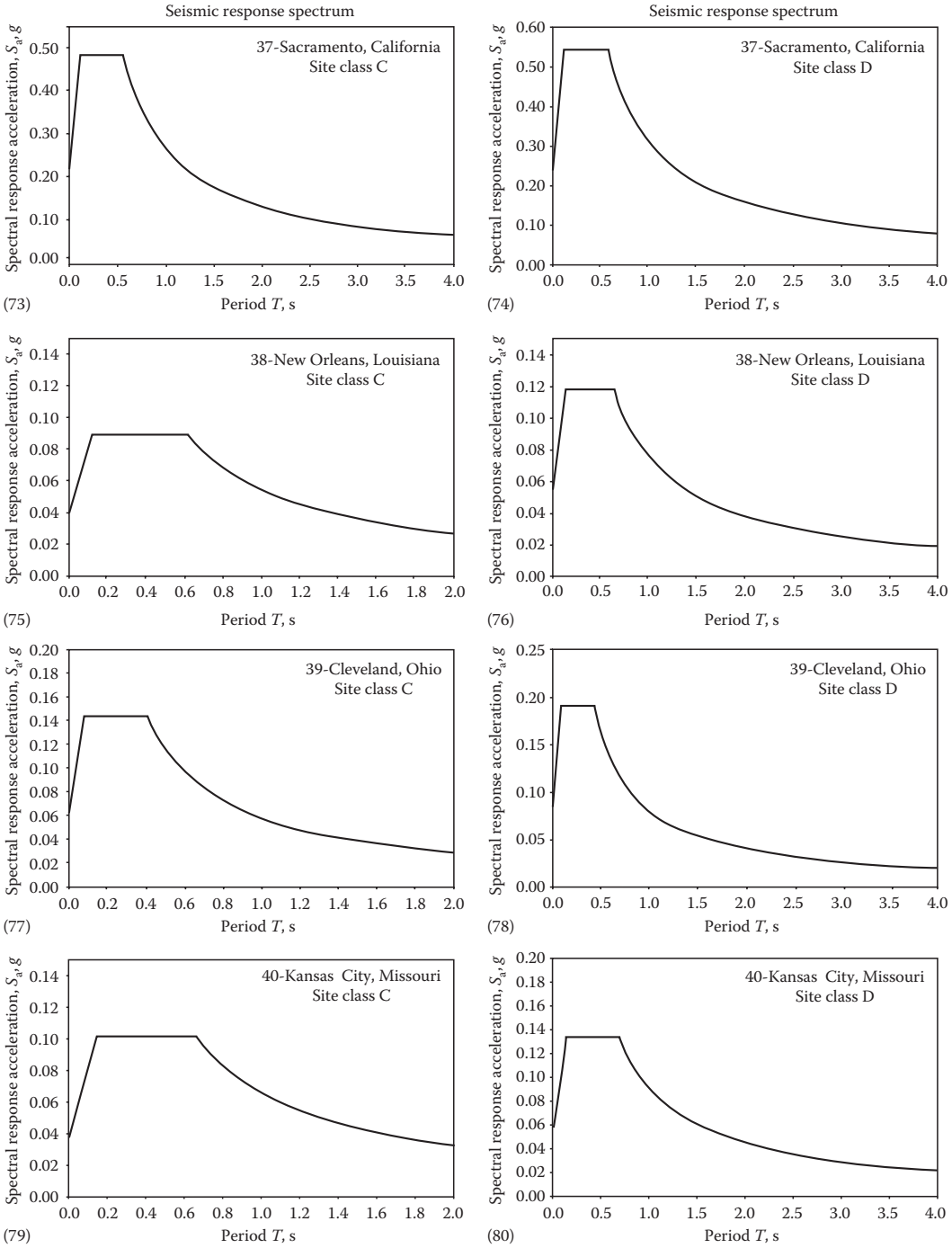


FIGURE 9.47 (continued)

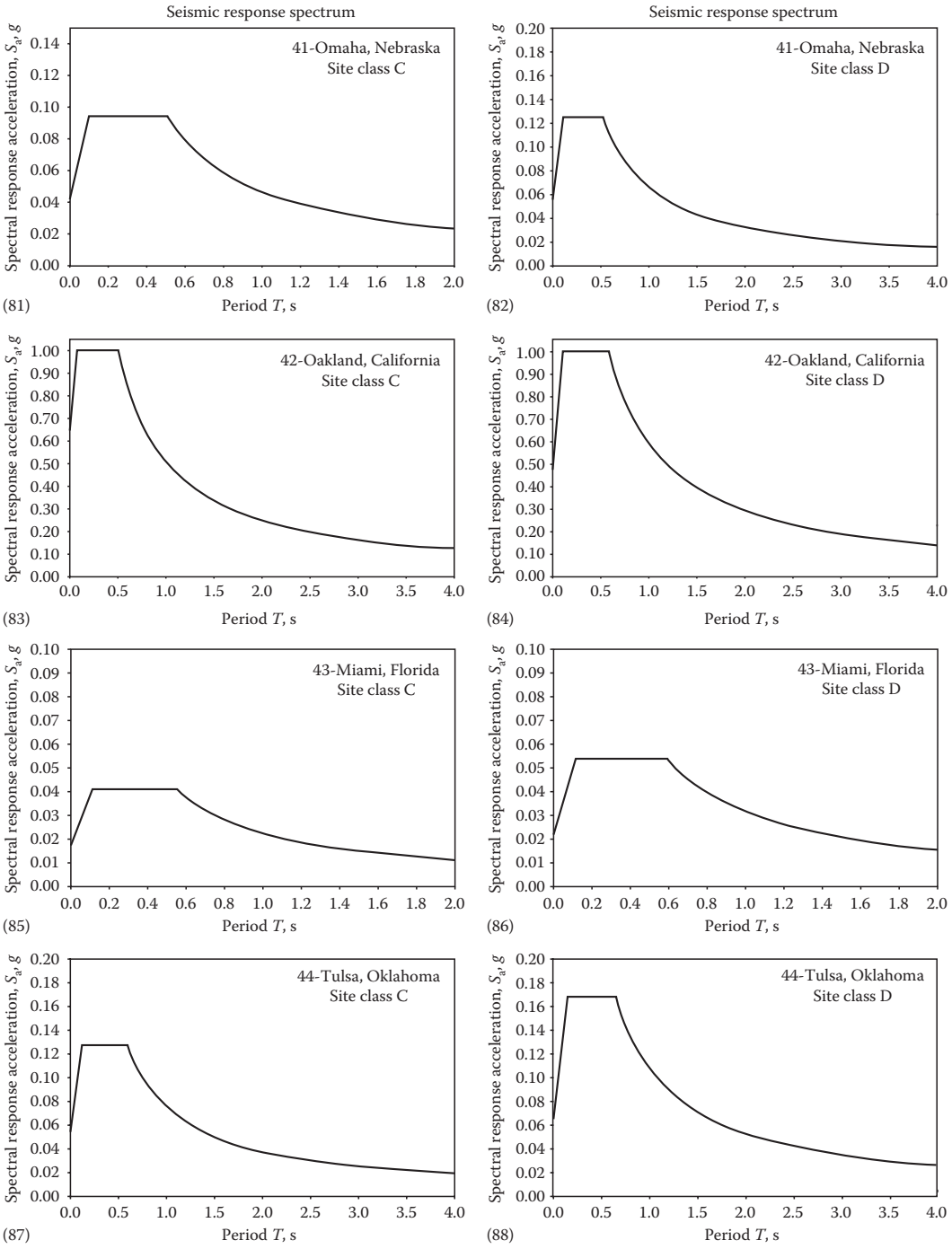


FIGURE 9.47 (continued)

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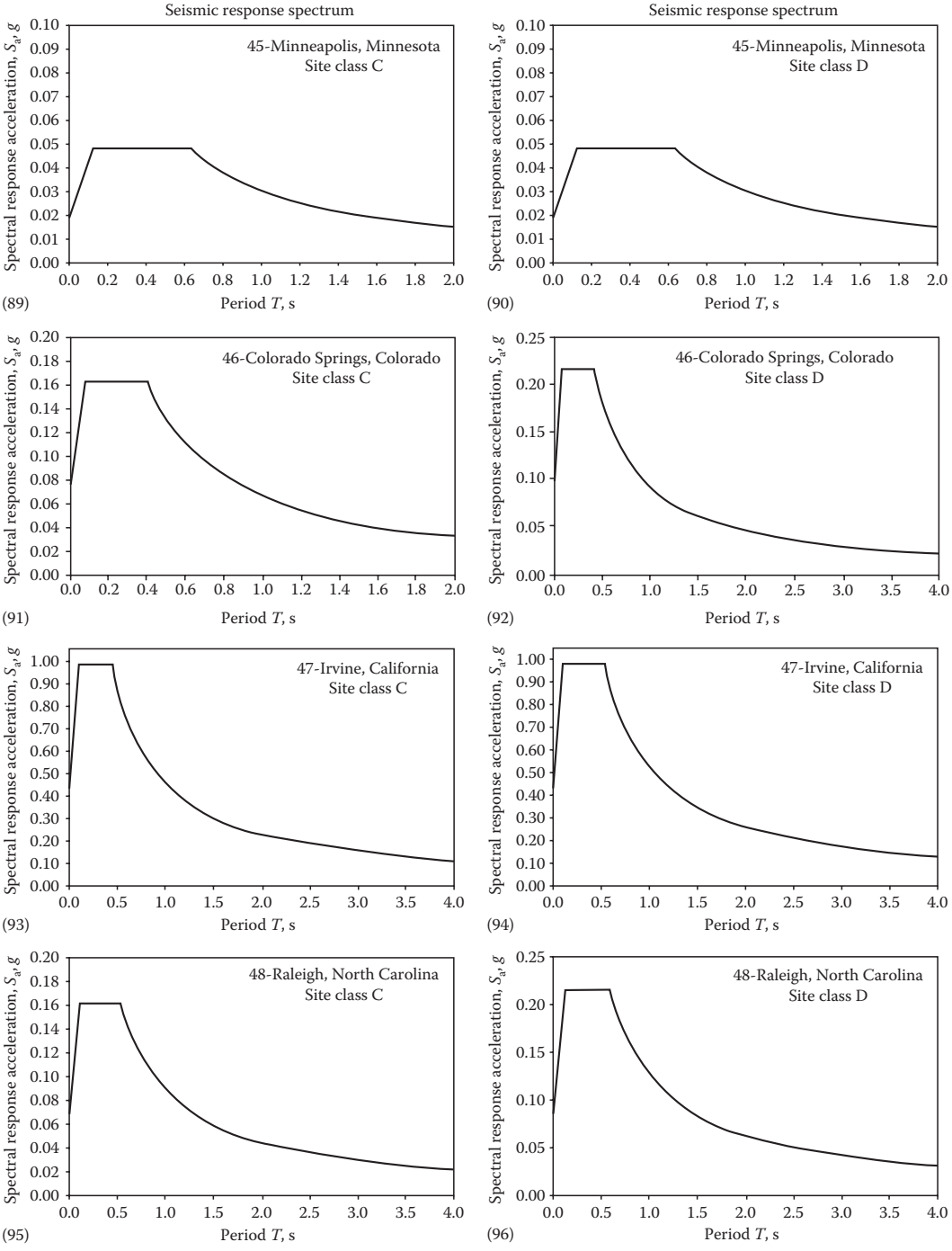


FIGURE 9.47 (continued)

Differential rather than the absolute shortening of column is more significant. Relative displacement between columns occurs because of the difference between the axial stresses, *P*/*A* ratios of columns. *P* is the axial load and *A* is the area of the column under consideration. If all columns in a building were to have the same *P*/*A* ratio under gravity loads, there would be no relative vertical movement between the columns. In typical buildings, however, this condition is seldom present. This is because, in buildings, not all columns are designed for the same axial loads. For example, the design of frame column is governed by the combined gravity and lateral loads while nonframe columns are designed essentially for gravity loads only. This results in a rather large difference in the *P*/*A* ratios between the exterior and interior columns. The differential column shortening between perimeter and interior columns may result in sloping floors leading to unwelcome problems in setting partitions, doors, and ceilings, in floors that slope excessively.

Consider, for example, a tubular system with closely spaced exterior columns and widely spaced interior columns. Because of their large tributary areas, the interior columns are more than likely to have large *P*/*A* ratios. The exterior columns, on the other hand, usually have a small *P*/*A* ratio for two reasons. First, their tributary areas are small because of their close spacing. Second, they are sized for stiffness to limit lateral displacements resulting in areas much in excess of these required from strength consideration alone. Because of this imbalance in the gravity stress level, these two groups of columns undergo different axial shortenings; the interior columns shorten much more than the exterior columns.

A reversed condition occurs in buildings with interior-braced core columns and widely spaced exterior columns; the exterior columns experience more axial shortening than the interior columns. The behavior of columns in buildings with other types of structural systems, such as interacting core and exterior frames, tends to be somewhere in between these two limiting cases.

9.7.8.1 Simplified Method of Calculating Δ_z, Axial Shortening of Columns

In a steel building, typically the cross-sectional area of a column increases in two-story increments from a minimum at top to a maximum at the base as shown in Figure 9.43. The incremental steps in column areas are due to the finite choice of available column shapes. Similarly, the axial load on a column increases at each floor in a stepwise manner up the building height. In tall buildings, the significance of these incremental steps diminishes rapidly, allowing us to make the following assumptions that can be used to derive a simplified formulation for axial shortening of columns. The first assumption relates to the variation of gravity loads, which may be assumed to increase linearly from top to the bottom. The second is similar to the first but applies to the variation of column areas. However, a linear variation using the actual column area at the bottom appears to underestimate the actual shortening of columns. A slight modification in which the column area at the bottom is taken equal to 0.9 times the actual area ($0.9 \times A_b$) appears to work well in predicting axial shortening.

9.7.8.2 Derivation of Simplified Expression for Δ_z

Integral calculus in conjunction with the simplified assumptions stated earlier is used to derive the following equation for Δ_z.

$$\Delta_z = P_b/E\ [-1/\alpha \ln\ (1 = \alpha z/A_b)] - \beta/E\ [-1/\alpha^2\{a\, z + A_b \ln(1 - \alpha z/A_b)\}]$$

where

L is the height of the building

Δ_z is the axial shortening at a height x (also denoted as z), above the foundation level

A_t is the column area at top

A_b is the modified column area at bottom equal to 0.9 × actual area of column at bottom = $0.9 \times A_b$

A_x is the area of column at height x (also denoted as z) above foundation level
α is the rate of change of area of column
P_t is the axial load at top
P_b is the axial load at bottom
P_x is the axial load at height x above foundation
β is the rate of change of axial load
E is the modulus of elasticity of steel

The reader is refered to the publication "Wind and Earthquake Resistant Buildings, Structural Analysis and Design" for the derivation of Δ_z.

Definitions of the variables used in the equation are shown schematically in Figure 9.48(A).

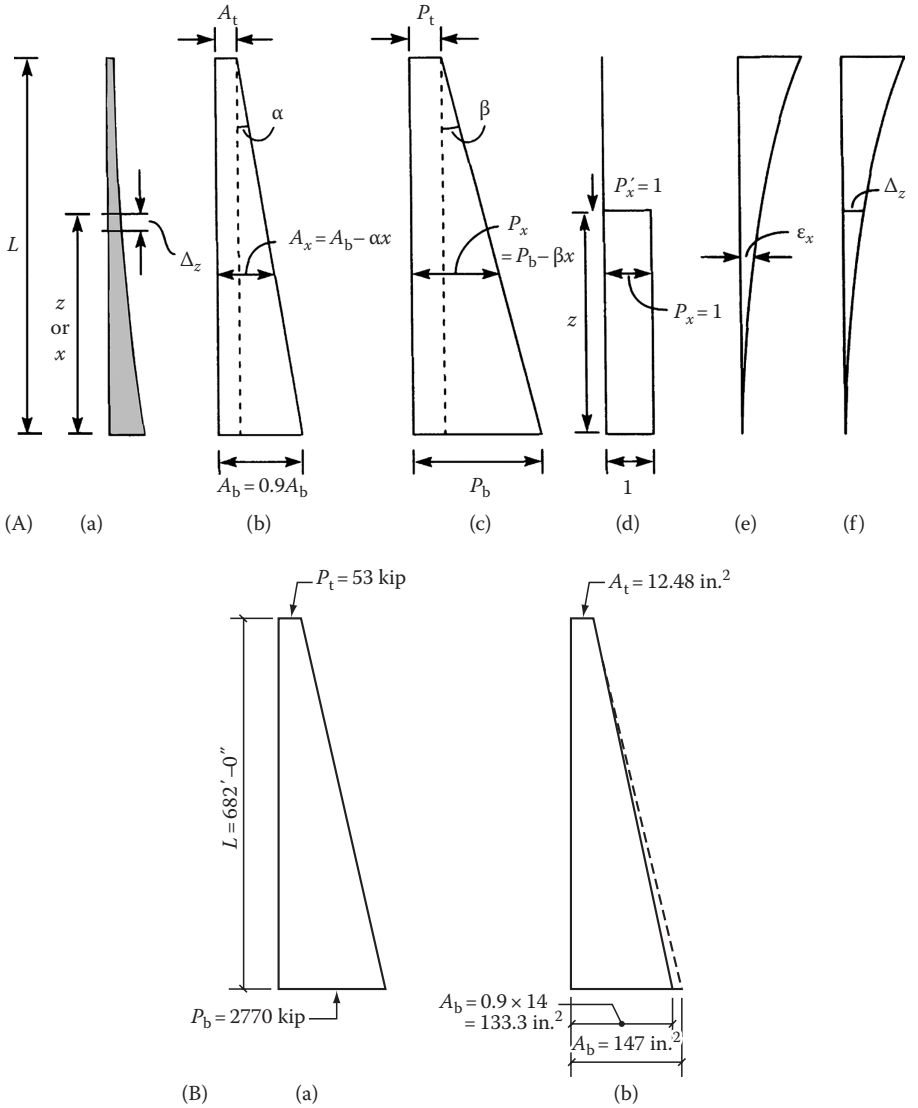


FIGURE 9.48 (A) Axial shortening of columns, closed-form solution: (a) axial shortening Δ_z; (b) variation of column areas; (c) axial load variation; (d) unit load at height z; (e) axial strain variation; (f) axial shortening. (B) Example 1: (a) axial load variation; (b) actual and assumed columns cross-sectional areas.

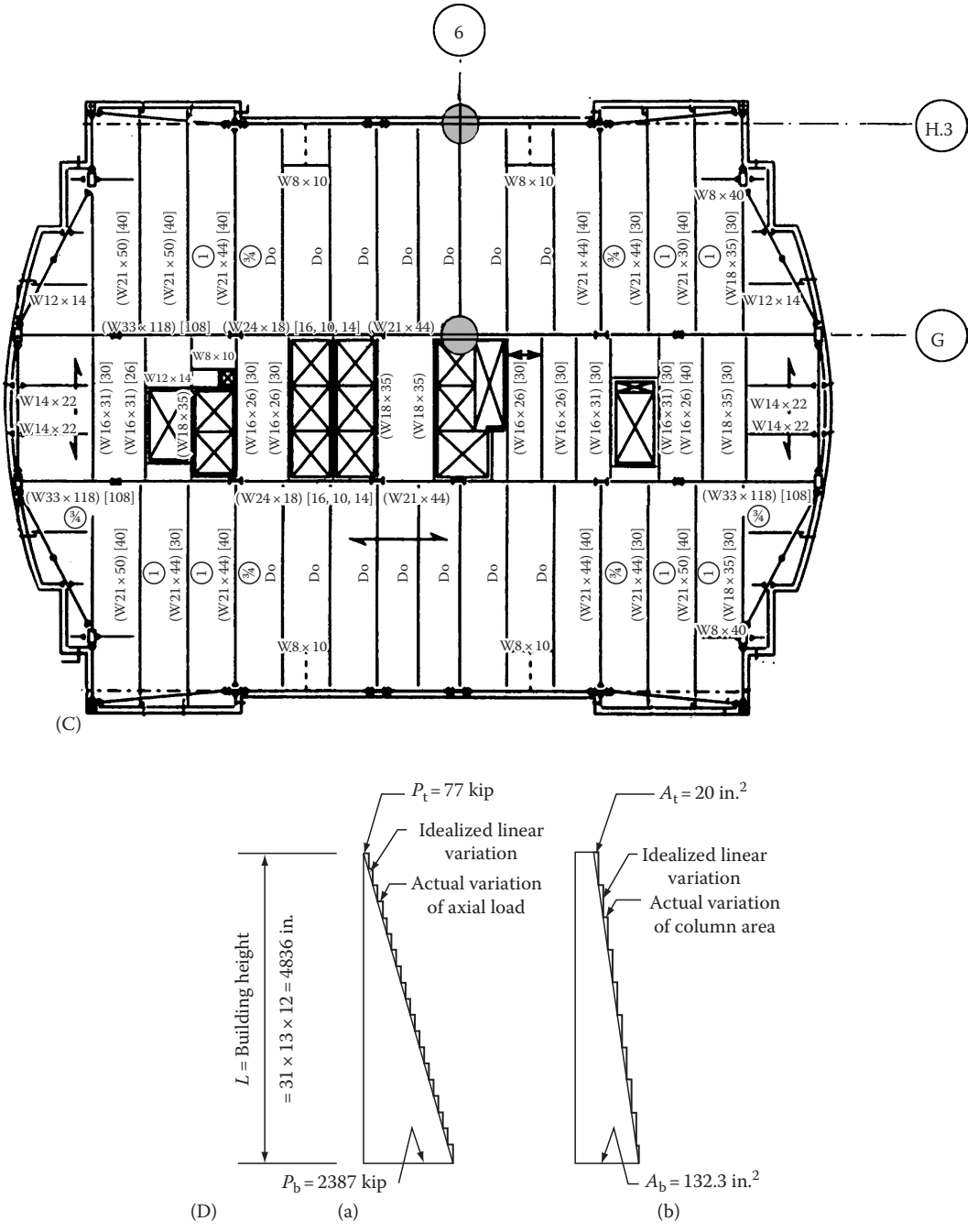


FIGURE 9.48 (continued) (C) Schematic framing plan. (D) Example 2, interior column G-6: (a) axial load variation; (b) variation of column areas.

(*continued*)

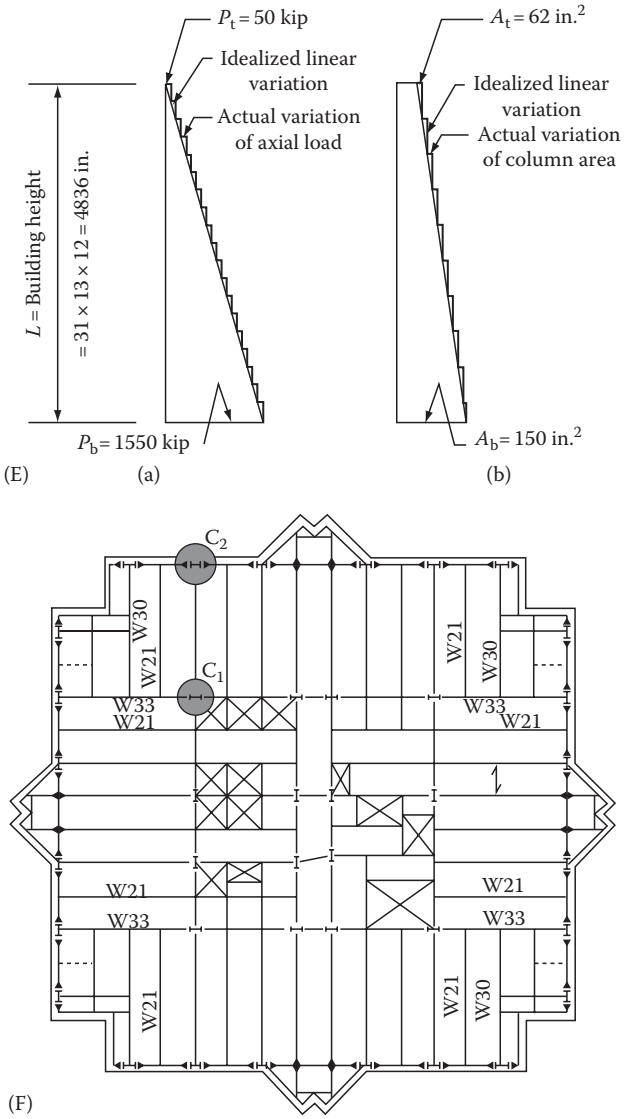


FIGURE 9.48 (continued) (E) Example 2, exterior column H.3-6: (a) axial load variation; (b) variation of column areas. (F) Framing plan. Column C, designed for gravity loads only, shortens more than C2, designed for both gravity and lateral loads. Compensating for relative elevation difference between columns is of importance in tall buildings.

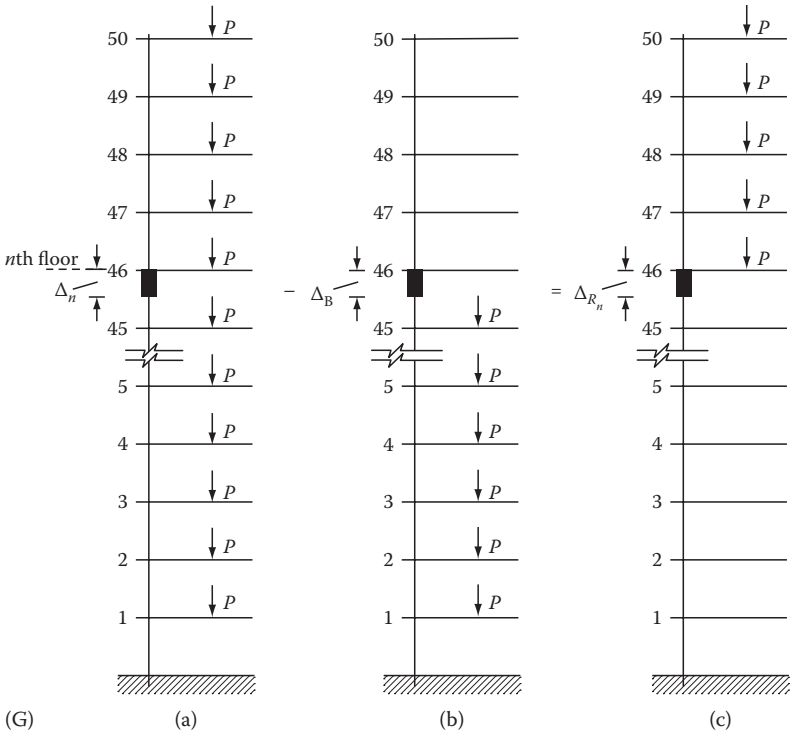


FIGURE 9.48 (continued) (G) Interpretation of column overlength: (a) Δ_n = column shortening at nth level due to loads on the entire height of column; (b) Δ_B = column shortening at nth level due to loads imposed at and below nth level; (c) column shortening yet to occur due to loads above nth level.

Example 1

Given (See Figure 9.48(B))

Height of building: $L = 682$ ft = 8184 in. (207.8 m)

Modulus of elasticity: $E = 29{,}000$ ksi (200×103 MPa)

Axial of load at top: $P_t = 53$ kip (237.5 kN)

Area of column at top: $A_t = 12.48$ in.^{2} (8052 mm^2)

Axial load at base: $P_b = 2770$ kip (12.32×103 kN)

Actual column area at base: $A_b = 147$ in.^{2} (94.84×10^3 mm^2)

Reduced column area at base: $A_b = 0.9 \times 147 = 133.3$ in.^{2} (86.0×10^3 mm^2)

Required

Axial shortening of column at top.

Solution

Since column shortening is calculated at top, $z = L$,

$$\alpha = (A_b - A_t)/L = (133 - 12.48)/8184 = 0.01476 \text{ in.}^2/\text{in.}$$

$$\beta = (P_b - P_t)/L = (2770 - 53)/8184 = 0.332 \text{ kip/in.}$$

$$\ln(1 - \alpha L/A_b) = \ln(1 - (0.01476 \times 8184)/133.3)$$

$$= \ln(0.09362)$$

$$= -2.36847$$

$$\Delta_L \text{ at top} = 2{,}770/29{,}000\ \{-1/0.01476 \times (-2.36847)\} - 0.332/29{,}000$$
$$\times \{-4590.15\ (0.01476 \times 8184\ + 133.3 \times -2.36847)\}$$
$$= 15.327 - 10.2$$
$$= 5.127 \text{ in.}$$

Similarly, the axial shortening is calculated at various heights by substituting appropriate values for z.

Example 2

Given

A steel building 403 ft tall with 31 framed levels including the roof. Tributary areas for gravity load calculations for the exterior column H.3 = 472 ft^2 per floor, and for the interior column G.6 is 810 ft^2. See Figure 9.48(C) for a schematic framing plan, and Table 9.9 for an abbreviated column schedule.

Typical loads for estimating axial shortening columns are as follows:

Interior Col G.6 3¼ lt. wt. on 3 in. deck = 50 psf

Partitions = 10 psf

Allowance for floor finishes,

ceiling, mech., etc. = 10 psf

Structural frame = 10 psf

Live load (reduced) = 15 psf

95 psf × 810/1000 = 76.9 kip/floor

Use 77 kip/floor

See Figure 9.48(D) for load distribution up the building height.

Exterior Col. H.3-6 3¼ lt. wt. on 3 in. deck = 50 psf

Partitions = 10 psf

Allowance for floor finishes,

ceiling, mech., etc. = 10 psf

Structural frame = 15 psf

Live load (reduced) = 15 psf

Exterior cladding = 5 psf

105 psf × 472/1000 = 49.56 kip/floor

Use 50 kip/floor

See Figure 9.48(E) for load distribution.

Required

Compute axial shortening of columns H.3-6 and G-6 at the roof due to gravity loads using the closed form equation given earlier. Provide column length corrections at levels 8, 16, 24, and the roof.

Solution

The variation of axial loads and cross-sectional areas for the two columns are shown in Figure 9.48(D) and (E). Observe that A_b is the actual area of the column at the foundation level multiplied by a factor of 0.9.

Thus, A_b for the interior column = 0.9 × 147 = 132.3 in.^{2}

A_b for the exterior column = 0.9 × 170 = 153 in.^{2}

The loads P_b for the exterior and interior columns at foundation level are as follows:

$P_b = 31 \times 50 = 1550$ kip (exterior column)

$P_b = 31 \times 77 = 2387$ kip (interior column)

Load $P_t = 50$ kip for the exterior, and 77 kip for the interior column.

Column length-shortening computations for column G.6 (interior column)

$L = 4836$ in.

$E = 29{,}000$ ksi

$P_t = 77$ kip

$P_b = 2387$ kip

$A_t = 20$ in.^{2}

$A_b = 0.9 \times 147 = 132.3$ in.^{2}

$\alpha = A_b - A_t/L = 132.3 - 20/4836 = 0.02322$ in.^{2}/in.

$\beta = P_b - P_t/L = 2387 - 77/4836 = 0.4777$ kip/in.

$I_n(1 - \alpha L/A_b) = I_n\,(1 - 0.02322/132.2 \times 4836)$

$= I_n\,(0.151233)$

$= -1.8889$

$\Delta_L = 2387/29000[-1/0.2333 \times -1.8889] - 0.4777/29000[-1854.7(0.02322 \times 4836 + 132.2 (-1.8889))]$

$= 6.6958 - 4.1869$

$= 2.50$ in.

Column length–shortening calculations for column H.3-6 (exterior column)

$L = 31 \times 13 \times 12 = 4836$ in.

$E = 29{,}000$ ksi

$P_t = 50$ kip

$P_b = 1550$ kip

$A_t = 62$ in.^{2}

$A_b = 0.9 \times 170 = 153$ in.^{2}

$\alpha = A_b - A_t/L = 153 - 62/4836 = 0.018817$ in.^{2}/in.

$\beta = P_b - P_b/L = 1550{-}50/4836 = 0.3102$ kip/in.

$I_n = (1 - \alpha L/A_b) = \ln(1 - 0.018817 \times 4836/153)$

$= I_n(0.4052)$

$= -0.90328$

$\Delta_L = 1550/29000[-1/0.018817 \times -0.90328]$

$= 0.3102/29000[-2824.2\{0.018817 \times 4836 + 153(-0.90328)\}]$

$= 2.56 - 1.42 = 1.134$ in.

9.7.8.3 Column Length Corrections, Δ_c

After determining axial shortening of building columns, the next step is to assign a column length correction Δ_c for each column. The objective is to attain as level a floor as practical. Δ_c is thus the difference between the theoretical height of a given column and its actual height after it has shortened. The magnitude of correction Δ_c in a tall building of say, 60 stories is rather small, perhaps 1/8 in. (3.17 mm) per floor, at the most. Therefore, instead of specifying this small correction at each level, in practice it is usual to specify lumped corrections at a few selected floors. For example, in lieu of 1/8 in. correction at each level, one would lump the correction, say, at every eighth floor. Thus, Δ_c would be equal to $1/8 \times 8 = 1$ in. (25.4 mm); to emphasize this concept, let us consider Table 9.8 that shows axial shortening computations for an assumed column of a 60-story building.

TABLE 9.8
Axial Shortening Computations

Level	$\bullet\ P_i$ Accumulated Load, kip	Column Section	L_k Story Height, in.	Δ_n Column Shortening, in.	Column Length Correction Each Level, in.	Lumped Column Length Correction, in.	Column Shortening, in.
50	53	W14 × 43	156	5.14	0.023	0.73	5.11
49	106	43	210	5.12	0.061		5.08
48	159	53	168	5.05	0.051		5.02
47	212	53	156	5.00	0.073		4.95
46	265	68	156	4.93	0.071		4.89
45	318	68	156	4.86	0.086		4.82
44	371	84	156	4.77	0.081		4.75
43	424	84	156	4.69	0.092		4.67
42	477	95	156	4.60	0.092		4.59
41	530	95	156	4.51	0.102		4.50
40	583	111	156	4.41	0.09	1.02	4.42
39	636	111	156	4.32	0.105		4.33
38	689	127	156	4.21	0.09		4.24
37	742	127	156	4.12	0.107		4.15
36	795	142	156	4.01	0.103		4.04
35	848	142	156	3.91	0.109		3.96
34	901	167	156	3.80	0.09		3.86
33	954	167	156	3.71	0.105		3.76
32	1007	176	156	3.62	0.105		3.66
31	1060	176	156	3.50	0.110		3.56
30	1113	202	156	3.39	0.101	1.07	3.46
29	1166	202	156	3.29	0.106		3.36
28	1219	211	156	3.19	0.106		3.26
27	1272	211	156	3.08	0.110		3.16
26	1325	228	156	2.97	0.106		3.12
25	1378	228	156	2.86	0.111		2.95
24	1431	246	156	2.75	0.107		2.85
23	1484	246	156	2.65	0.111		2.74
22	1537	264	156	2.53	0.107		2.64
21	1590	264	156	2.43	0.110		2.53
20	1643	287	156	2.32	0.104	1.06	2.42
19	1696	287	156	2.20	0.108		2.32
18	1749	314	156	2.10	0.101		2.21
17	1802	314	156	2.00	0.105		2.10
16	1855	314	156	1.90	0.108		1.99
15	1908	314	156	1.79	0.111		1.88
14	1961	342	156	1.68	0.104		1.78
13	2014	342	156	1.58	0.107		1.67
12	2067	370	156	1.47	0.101		1.56
11	2120	370	156	1.37	0.104		1.45
10	2173	370	156	1.26	0.107	0.53	1.34
9	2226	370	156	1.16	0.109		1.23

TABLE 9.8 (continued)
Axial Shortening Computations

Level	$\bullet\ P_i$ Accumulated Load, kip	Column Section	L_k Story Height, in.	Δ_n Column Shortening, in.	Column Length Correction Each Level, in.	Lumped Column Length Correction, in.	Column Shortening, in.
8	2279	398	156	1.05	0.104		1.12
7	2332	398	156	0.94	0.107		1.01
6	2385	398	156	0.84	0.11		0.89
5	2488	398	210	0.73	0.11		0.78
4	2491	426	168	0.62	0.11		0.67
3	2544	426	156	0.51	0.11		0.56
2	2597	500	156	0.40	0.09		0.45
Mezzanine	2650	500	240	0.31	0.15		0.17
1	2770	W14 × 500	240	0.16	0.16		0.17

NS = Number of stories = 50.

TABLE 9.9
Example 2: Column Schedule

Level	Interior Column	Area (in.²)	Exterior Column	Area (in.²)
R	W14 × 68	20	W30 × 211	62
28	W14 × 90	26.5	W30 × 235	69
26	W14 × 120	35.3	W30 × 261	76.7
24	W14 × 132	35.8	W30 × 292	85.7
22	W14 × 145	42.7	W30 × 326	95.7
20	W14 × 193	56.7	W30 × 326	95.7
18	W14 × 233	68.5	W30 × 357	104
16	W14 × 257	75.6	W30 × 357	104
14	W14 × 283	83.3	W30 × 351	114
12	W14 × 311	91.4	W30 × 391	114
10	W14 × 342	101	W30 × 433	127
8	W14 × 370	109	W30 × 433	127
6	W14 × 398	117	W30 × 477	140
4	W14 × 426	125	W30 × 477	140
2	W14 × 455	134	W30 × 526	154
G	W14 × 500	147	W30 × 581	170

The assumed column sections, lengths, and variation of axial loads are given in the table. The last column shows the lumped corrections at levels 2, 10, 20, 30, 40, and the roof. Basically, these corrections represent the lengths to be added to the theoretical lengths of the column to achieve the specified height after the column has shortened due to gravity loads. For example, Δ_c = 1¼ in. (31.75 mm) at the tenth level means that the actual fabricated length of column from its base to the tenth level should be made 1¼ in. longer than the theoretical length. This overlength could be achieved by increasing the length of column in each tier by ¼ in. (6.35 mm) (ten stories equal five tiers; therefore, ¼ in. times 5 gives 1¼ in.). However, the fabricator may elect to increase the column length in each story by 1/8 in. (3.2 mm) instead of ¼ in. per tier. This, and other similar options are, of course, permissible because the end result of achieving a desired Δ_c will be the same.

TABLE 9.10
Column Length Correction

	Interior Column		Exterior Column	
Level	Column Shortening	Correction to Scheduled Length	Column Shortening	Correction to Scheduled Length
Roof	2.50 in.	4@ 3/16 in. = 0.75 in.	1.13 in.	2@ 1/8 in. = 0.25 in.
24	1.80 in.	4@ 1/8 in. = 0.50 in.	0.75 in.	2@ 1/8 in. = 0.25 in.
16	1.25 in.	4@ 3/16 in. = 0.75 in.	0.50 in.	2@ 3/16 in. = 0.375 in.
8	0.625 in.	4@1/8 in. = 0.50 in.	0.25 in.	2@ 1/8 in. = 0.25 in.

The value of Δ_c= 2 in. (50.8 mm) at the 20th floor means the overlength of columns between levels 1 and 20 should be 2 in. However, an overlength of 1¼ in. (31.75 mm) up to the 10th level has already been achieved by specifying Δ_c = 1¼ in. at the 10th level. Therefore, the increment between the 10th and 20th levels should be 2 in. less 1¼ in. = ¾ in. (19.0 mm).

Let us apply this procedure to the two columns G.6 and H.3-6 of Example 2. The calculated axial shortening for these two columns at top are 2.50 and 1.134 in. A practical method of specifying corrections Δ_c for these columns is shown in Table 9.10.

9.7.8.4 Column Shortening Verification during Construction

This concept is best explained with reference to Figure 9.48(F) that shows a framing plan for a hypothetical building, say, some 48 stories tall. Identified therein are two columns: C_1, an interior column with a large tributary area; and C_2, an exterior column of framed tube with a relatively small tributary area. Under gravity loads, C_1 would shorten more than C_2 because (1) C_1, designed only for gravity loads, has a *P*/*A* ratio that is relatively high; and (2) C_2, designed as a frame column, has its *P*/*A* ratio significantly less than that for C_1, because it is lightly loaded under gravity loads.

Assume that you as the engineer for the project have specified column length corrections to C_1 at levels 8, 16, 24, 32, 40, and 48 with correction of 2 in. specified at level 24. Let us say that when steel erection is at the level, 24, the steel erector surveys the top elevations of columns, reports the top of column C_1 is 1 in. higher than the top of C_2, and requests you, the engineer to confirm if this is acceptable in view of the fact that additional shortening of the column is yet to occur.

Further calculations are needed to verify that this 1 in. overlength of C_1 will indeed come down after the application deadloads at levels 24 through roof. This concept of verifying the overlength of columns during construction is shown in Figure 9.48(G). Note that Δ_{Rn} shown therein corresponds to the 1 in. discussed here for the hypothetical building.

9.7.9 Unit Weight of Structural Steel for Preliminary Estimate

The total quantity of structural steel required for a building divided by its gross area has always been, and will always be, an item of great interest to building developers and designers alike. In U.S. practice, the unit quantity of steel is expressed in terms of pounds per square foot as shown in Table 9.11 for selected buildings built between 1930 and 1971.

In selecting a structural system for a given building, as stated previously, it is common practice to evaluate several possible structural schemes that meet the owner's requirements along with architectural aspirations. The deciding factor most often, then, is the unit quantity of material for the systems. Although the unit weight of steel required for a given building may not address all factors relevant to construction cost, it is a component that most often pushes a particular scheme to the forefront.

TABLE 9.11
Unit Quantity of Steel for Selected High- and Medium-Rise Buildings

Year	Stories	Height/ Width	psf	Building
1930	102	9.3	42.2	Empire State Building, New York
1968	100	7.9	29.7	John Hancock Center, Chicago
1972	110	6.9	37.0	World Trade Center, New York
1974	109	6.4	33.0	Sears & Roebuck, Chicago
1963	60	7.3	55.2	Chase Manhattan, New York
1969	60	5.7	38.0	First National, Bank, Chicago
1971	64	6.3	30.0	U.S. Steel Building, Pittsburgh
1971	57	6.1	17.9	I.D.S. Center, Minneapolis
1957	42	5.1	28.0	Seagram Building, New York
1970	41	4.1	21.0	Boston Co. Building, Boston
1965	30	5.7	38.0	Civic Center, Chicago
1969	26	4.0	26.0	Alcoa Building, San Francisco
1971	10	5.1	6.3	Low Income Housing, Brockton, Massachusetts

Unit quantities are relatively easy to evaluate once complete working drawings and specifications have been prepared. Prior to this point, however, one must use "conceptual estimating" to determine approximate costs. Conceptual estimate for a given building demands considerable engineering judgment in addition to determining unit quantities. This is because we need to adjust the so-called average unit costs to reflect complexity of construction operations, expected time required for construction, cost of borrowing money for the length of construction, etc.

Prior to the advent of high efficiency systems such tubular and megaframe frames, most high rises were designed using braced and/or moment frames as lateralload-resisting systems. Consequently, their poundage is relatively high compared to present-day buildings.

It is of interest for conceptual estimating purposes to assemble unit structural steel quantities for buildings that have been built in the past. Figure 9.49a through c, and Table 9.11 show the general trend in the increase of unit weight of steel as the building height is increased.

Four distinct regions A, B, C, and D are shown in Figure 9.49c. Region A is for buildings up to 30 stories, B for buildings between 30 and 50 stories, C for buildings 50 to 70 stories, and D for high rises in excess of 70 stories. Use of the figure is best explained using the following examples.

Example 1

A proposed 10-story building in a wind-controlled low-seismic-risk area. The engineer is asked to come up with a unit quantity of structural steel for purposes of a conceptual cost estimate.

For a 10-story building, it is seen from Figure 9.49c. that the lower and upper bounds for the unit weight are 10.5 and 15 psf, respectively, which is a rather wide range. The engineer now has to make some judgment calls, depending on what is known about the building at this stage of the game. Some relevant questions are as follows: (1) What are the typical spans for the floor framing? (2) Are there any undue restrictions for the beam and girder depths? (3) What is the likely lateral framing system? If none of these questions can be answered with any great certainty, it is perhaps prudent to use an average of two unit weights. Thus, the unit weight for our building would be equal to (10.5 + 15)/2 = 12.75, rounded to 13 psf.

Example 2

Fifty-story building. Using a similar procedure, the engineer determines that the lower- and upper-bound values are 21 and 32 psf, giving an average unit quantity of 26.5 psf.

Example 3

Twenty-two-story building, assigned to SDC D. Design controlled by seismic loads. The straight line designated as S in Figure 9.49c represents an approximate unit weight of steel for buildings located in high-risk seismic zones. Observe that this line stops at 25 stories, implying that design of tall buildings, in general, is controlled by wind loads. For the example building, the unit weight of steel as given by the straight line S is 22 psf.

Example 4

Now consider the same building in a seismically low-risk area such as Houston, Texas. We go to the region designated as an average zone on the graph. From the graph, the unit weight is seen to vary from a low of 13.5 psf to a maximum of 19.5 psf. As in previous cases, we use an average unit quantity of 16.5 psf for the preliminary conceptual estimate.

Example 5

Seventy-story composite building. The line designated as CB2 shows unit weight for steel buildings with composite frame columns. As a preliminary estimate, a unit quantity of 23 psf is appropriate for the building,

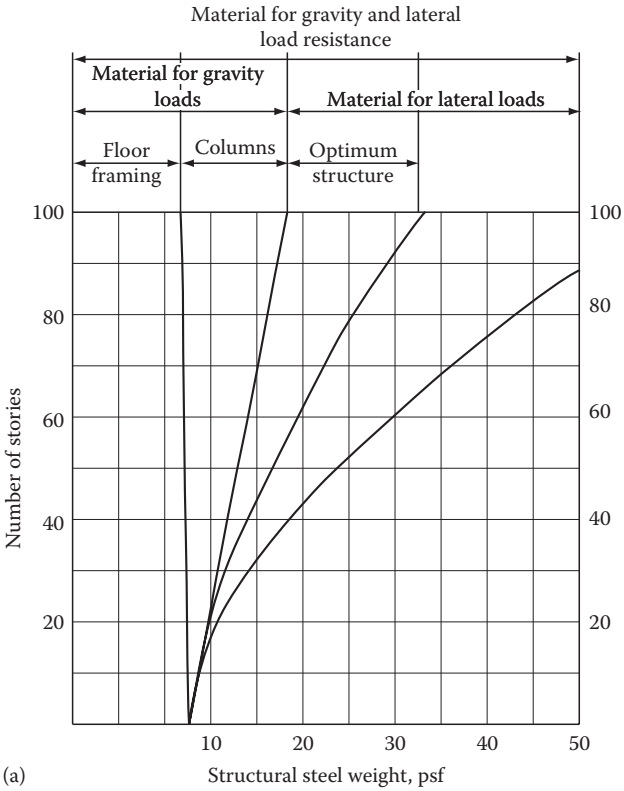


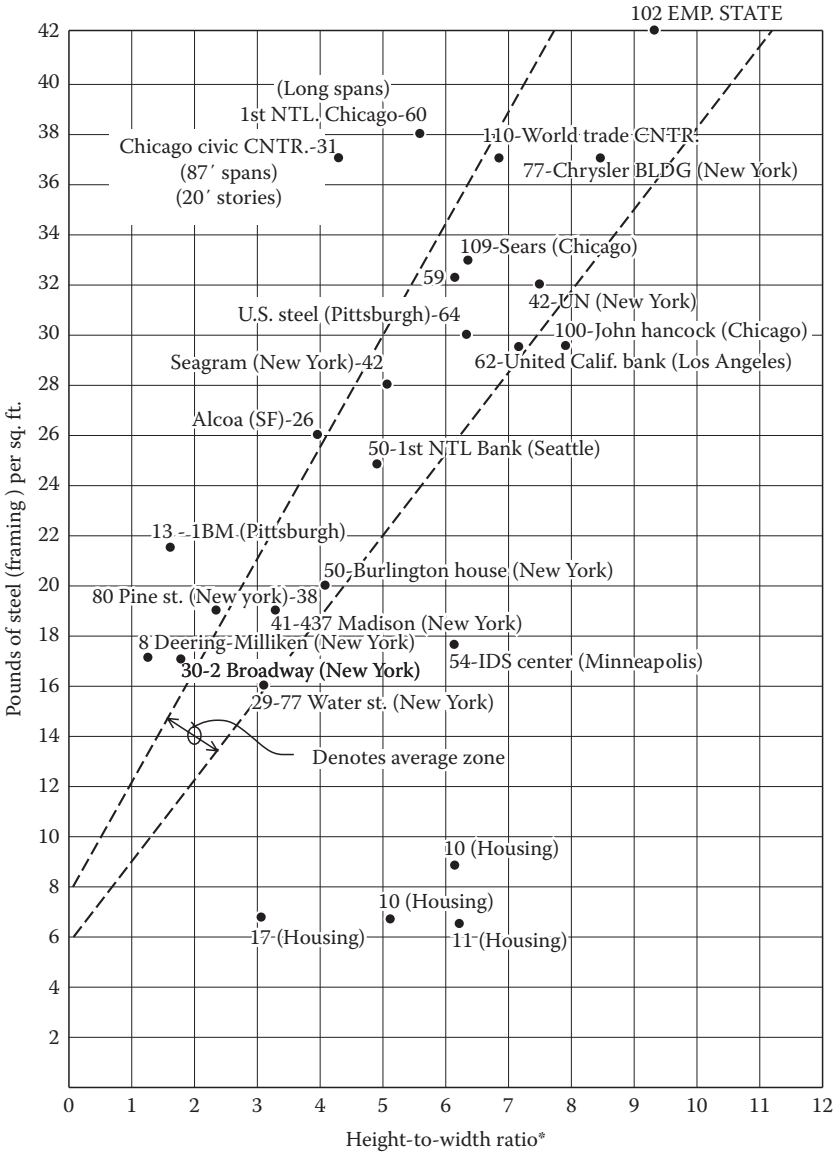
FIGURE 9.49 (a) Steel quantities for gravity and lateral load resistance.

Example 6

Fifty-story composite building with composite columns and composite girders (buildings with composite columns and composite shear walls may be considered similar). From the graph noted as CB2, we get a unit quantity of 13.5 psf for this building.

9.7.9.1 Concept of Premium for Height

If there were no lateral loads such as wind or earthquake, a given high-rise building could be designed primarily for gravity loads. Such a design would not impose a premium for height. Since there is no way to circumvent gravity loads the absolute minimum quantity of material required for



(b) * The number beside each entry indicates building height in stories.

FIGURE 9.49 (continued) (b) Unit weight of steel as it relates to building's height-to-width ratio.

(*continued*)

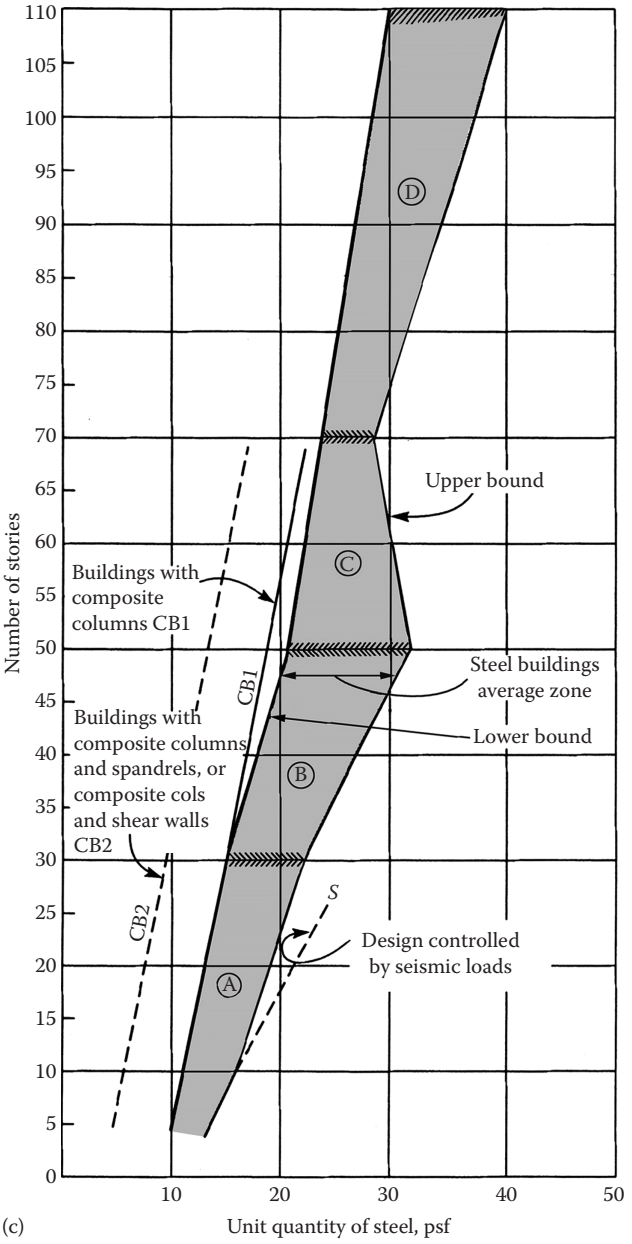


FIGURE 9.49 (continued) (c) Structural steel unit quantities. A design aid for conceptual estimate.

a building, irrespective of its height, would not be less than that required for gravity loads alone. From a structural point of view, this would correspond to the most efficient or optimum system.

Assuming equal bays, the material quantities required for gravity floor framing in low- and high-rise structures are essentially identical; it makes no difference in steel quantities whether the floor being framed is at the second level of a low rise or at the 70th level of a high-rise building. The steel required for floor framing is a function of the column-to-column span and not the building height. However, the material required for columns in a high rise is substantially more than that for a low rise. In fact, the steel increases in the ratio $(n + 1)/2$, where n is the number of floors. This is because columns in taller are designed to carry cumulative gravity loads, thus requiring more steel than a one-story structure of the same floor area.

The quantity of materials required for resisting lateral loads is even more pronounced and would soon far outstrip all other structural costs if rigid frame action were employed in very tall buildings. The graph shown in Figure 9.49a illustrates how the unit weight of structural steel increases as the number of floors increases. Wind begins to show its dominance at about 50 stories and becomes increasingly important with greater height. For example, in a steel building using rigid frame action, the total weight of, say, 24 psf (1149 Pa) of structural steel is split evenly at about 8 psf (393PA) for each of the three subsystems, namely, (1) floor framing (2) gravity columns, and (3) wind bracing system. Above 50 stories, lateral bracing ingenuity often makes the difference between an economical solution and an expensive one. The objective is to arrive at a lateral bracing system that keeps the additional material required for lateral loads to a reasonable amount.

Low- to medium-rise buildings, particularly those controlled by wind are normally designed for gravity loads, then checked for their ability to resist lateral loads. For high-rise buildings, however, there is a drastic increase in the amount of material needed for resistance of lateral forces. With respect to gravity loads, the weight of the structures increases almost linearly with the number of stories. However, the amount of material needed for resistance of lateral forces increases at an accelerating rate. For example, using the rigid frame principle, one would estimate about 55 psf of steel for a 90-story building instead of the tubular system with only 34 psf (e.g., Standard Oil Building, Chicago).

Weight-to-area ratios, (which we referred to as unit weight, in the previous sections) for some typical high-rise buildings are given in Figure 9.49b. The Empire State Building has a ratio of 42.2 psf in contrast to the 29.7 of the diagonally braced tubular John Hancock Center. An even larger contrast is the 60-story Chase Manhattan Building with 55 psf and the slightly lower 54-story IDS Building with only 17.9 psf. The Chase Manhattan Building is a long-span rigid frame that needs huge girders to resist wind forces. The IDS Building owes its efficiency to the belt truss system. Disproportionate amounts of steel do not necessarily indicate that the structural design of the building is poor. For instance, the Civic Center Building in Chicago uses about twice as much steel as other buildings in its height range. However, it has to satisfy the functional requirements of size and location of courtrooms. Thus, the girder spans are 87 ft and the floor heights are much larger than usual (i.e., 30 stories for 640 ft).

10 Connection Details

PREVIEW

Connections are assemblages of parts that transmit from one member to others, the forces and moments that the member must resist at the point of connection. Their design is the nonquantifiable, nonautomatic phase of design, what engineers call their art, the art of linking together members that results in higher quality structures with the least amount of material.

The connection itself, isolated as a free-body, must fulfill the same requirements of equilibrium and compatibility as the total structure or any of its members. The actual stress conditions, however, are often too complex for complete analysis by simple elastic methods, forcing engineers to rely much more on test results than they do in member selection.

Connections of steel members are usually made in one of the four different ways:

1. With multiple bolts
2. By direct bearing (limited to compression transfer)
3. By welds
4. By pins

This chapter will focus primarily on bolted and welded connections. A number of connection details representative of the engineering practice in the west coast of United States are given in Figures 10.1 through 10.66.

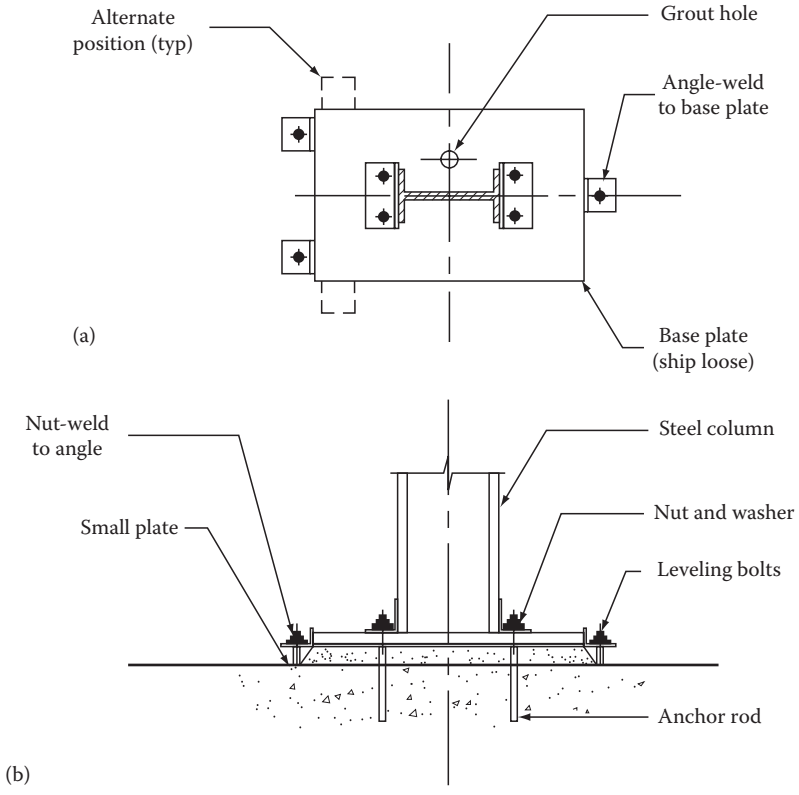


FIGURE 10.1 (BP-1) Base plate with leveling bolts: (a) plan; (b) section.

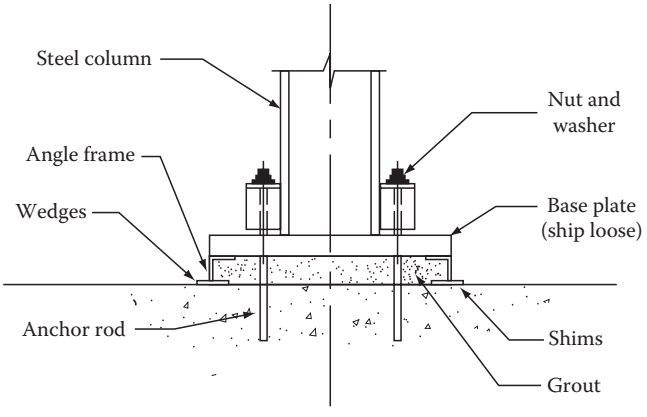


FIGURE 10.2 (BP-2) Base plate with leveling angles, plan.

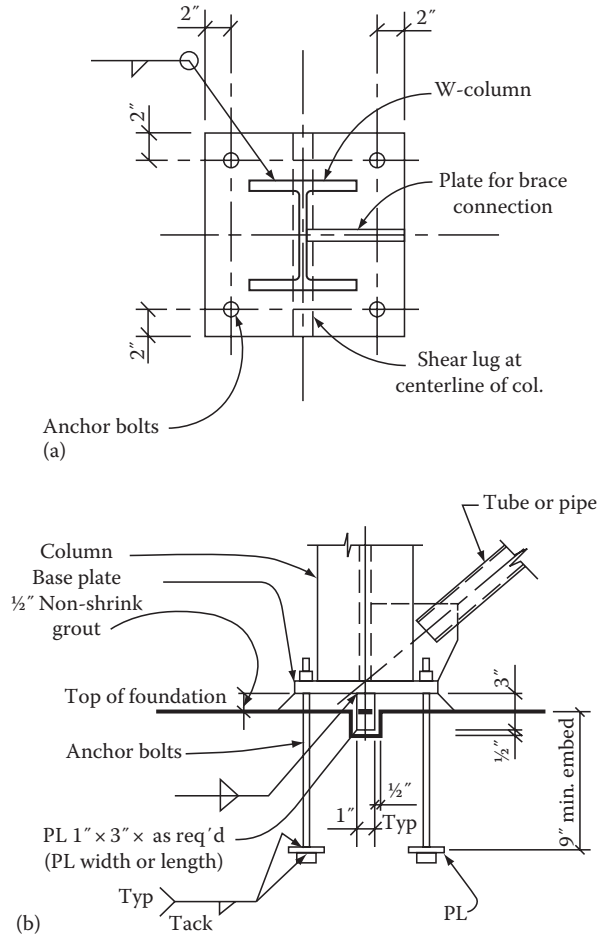


FIGURE 10.3 (BP-3) Base plate with gusset plate at column web: (a) plan; (b) section.

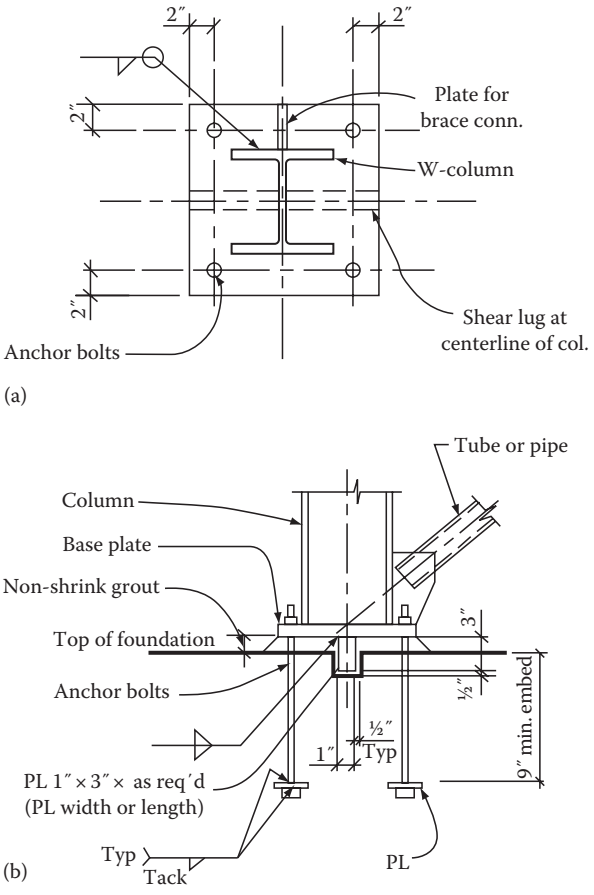


FIGURE 10.4 (BP-4) Base plate with gusset plate at column flange: (a) plan; (b) section.

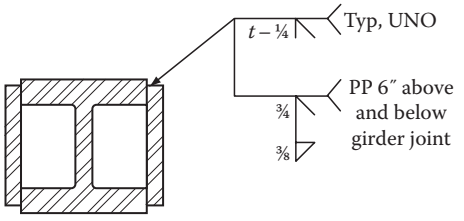


FIGURE 10.5 (C-1) Added plates to WF column.

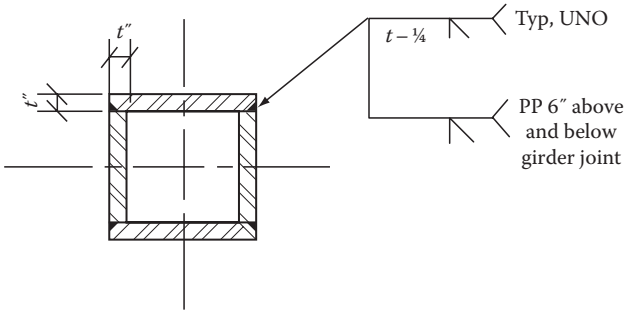


FIGURE 10.6 (C-2) Welding of box column.

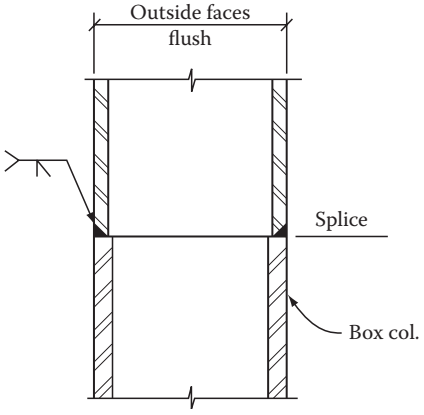


FIGURE 10.7 (C-3) Box column splice.

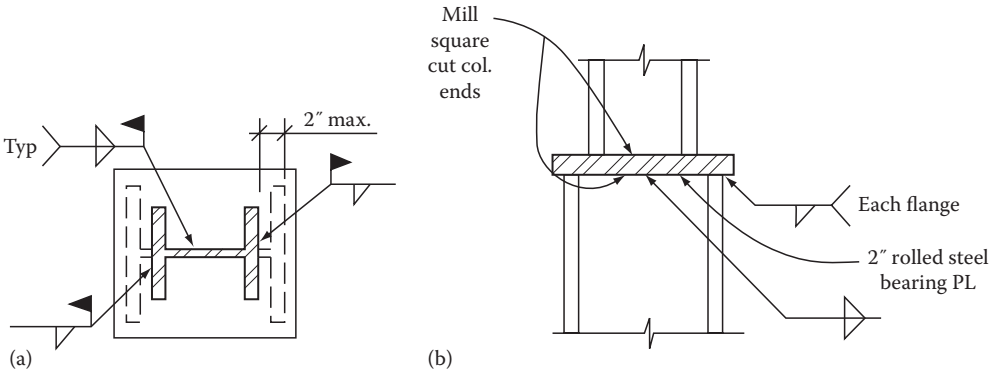


FIGURE 10.8 (C-4) WF column splice, columns vary in size.

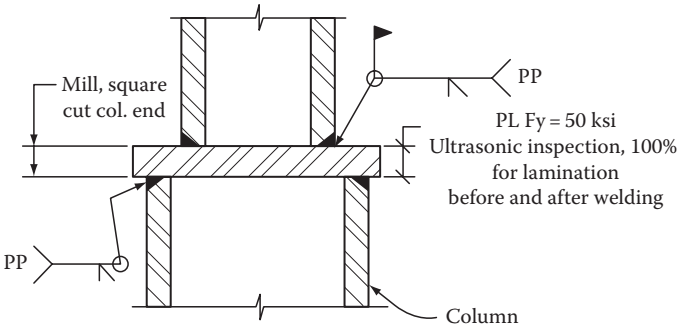


FIGURE 10.9 (C-5) Box column splice, columns vary in size.

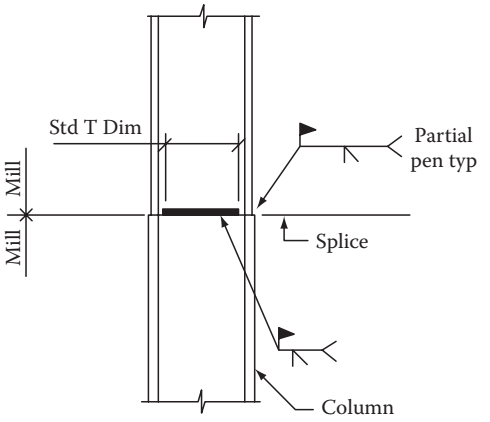


FIGURE 10.10 (C-6) WF column splice, same size wide flanges.

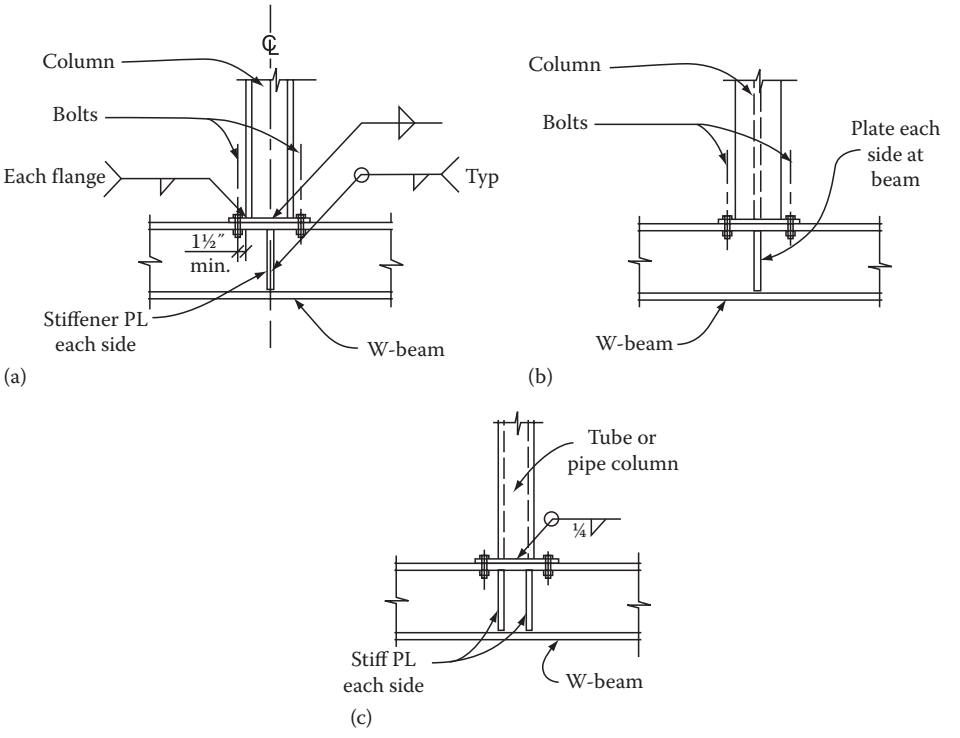


FIGURE 10.11 (C-7) Carried columns, bolted connection: (a) webs parallel; (b) webs perpendicular; (c) pipe column.

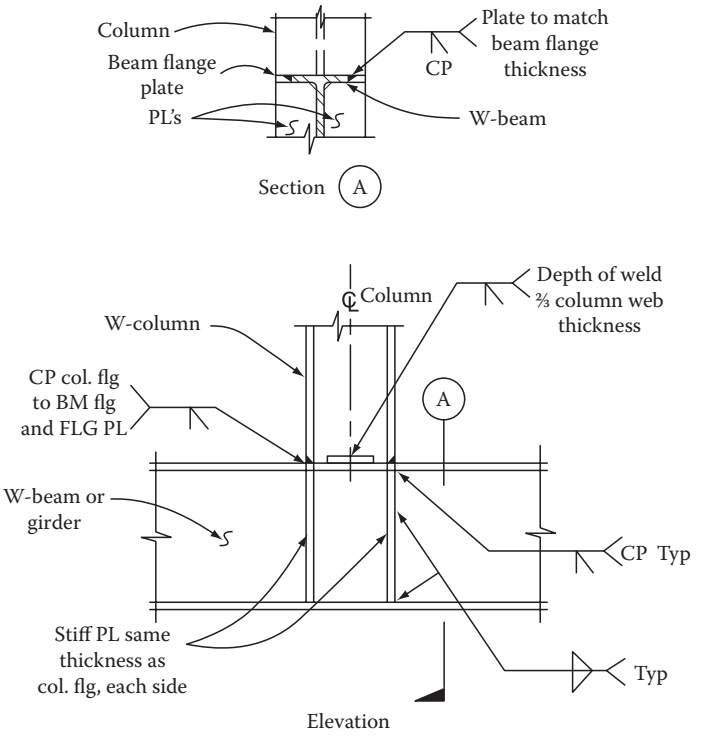


FIGURE 10.12 (C-8) Carried column, welded connection.

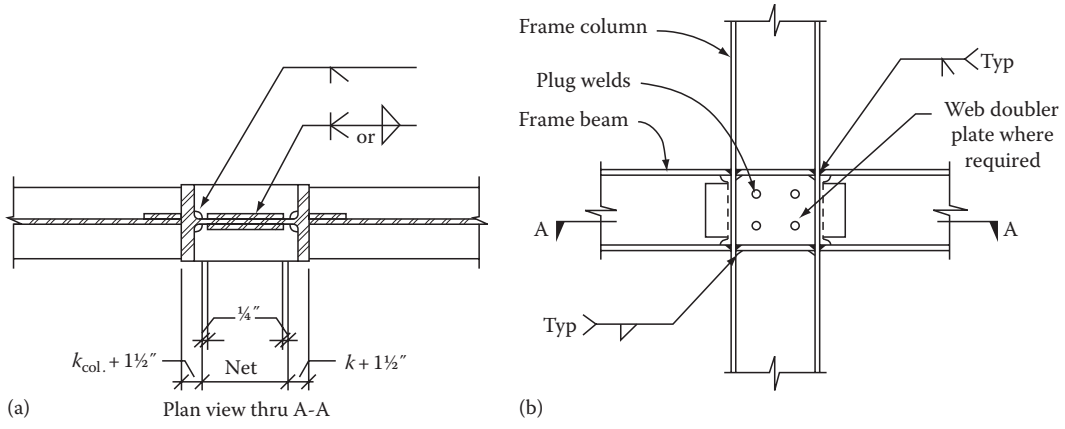


FIGURE 10.13 (C-9) Column continuity and doubler plates: (a) plan; (b) elevation.

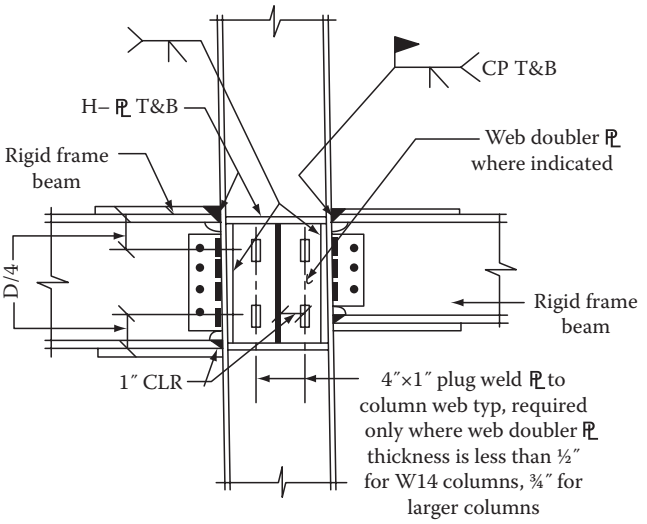


FIGURE 10.14 (C-10) Column doubler plates at moment connections.

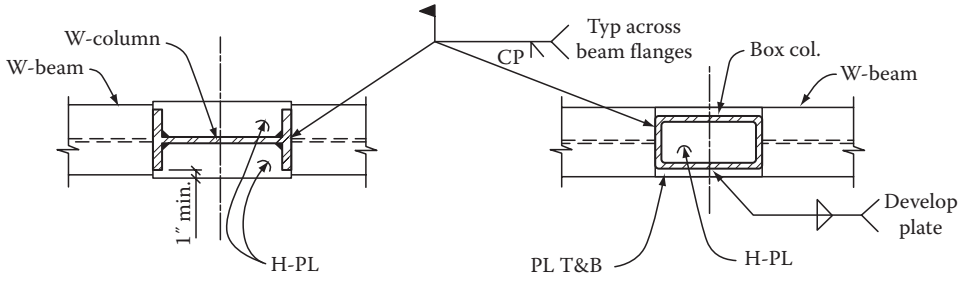


FIGURE 10.15 (C-11) Column H plate, beam flange wider than column flange.

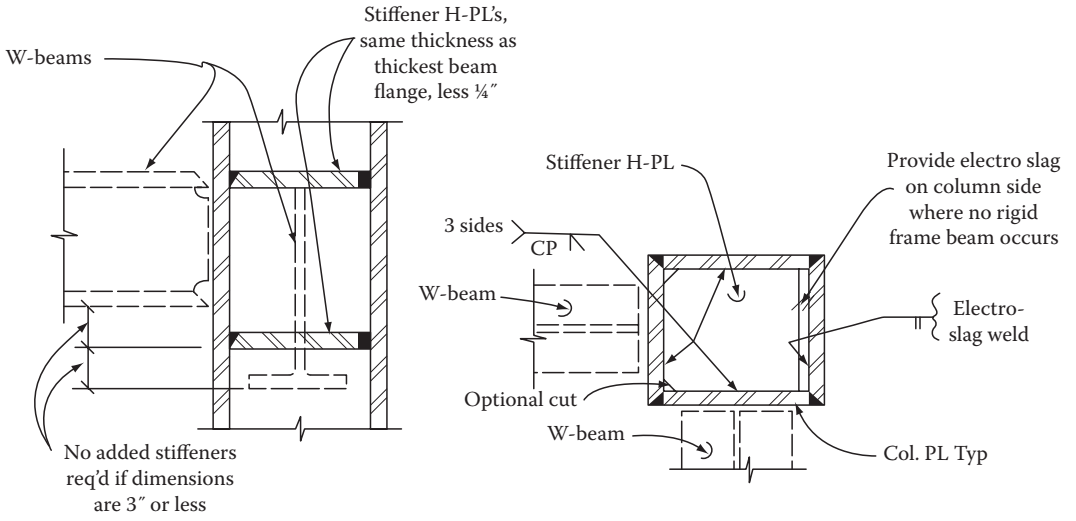


FIGURE 10.16 (C-12) Box column to beam moment connection.

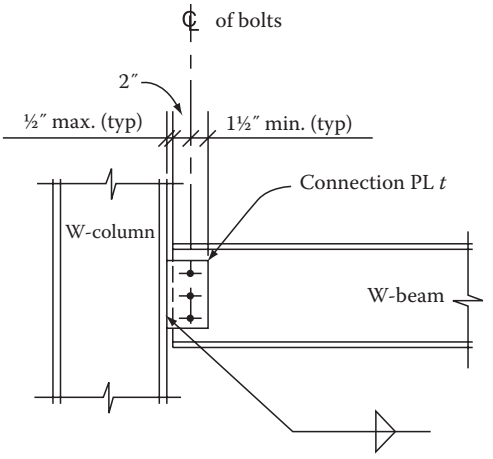


FIGURE 10.17 (BC-1) Beam to column flange simple connection.

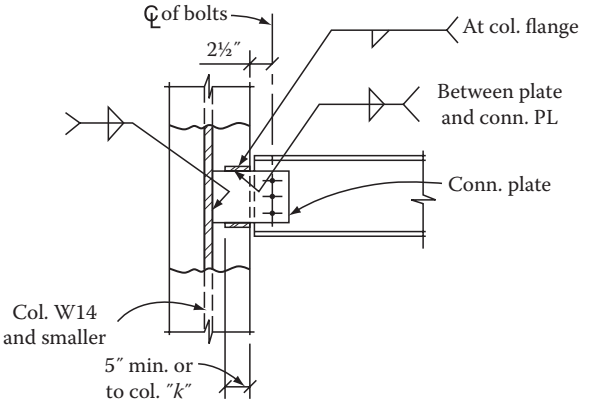


FIGURE 10.18 (BC-2) Beam to column web simple connection.

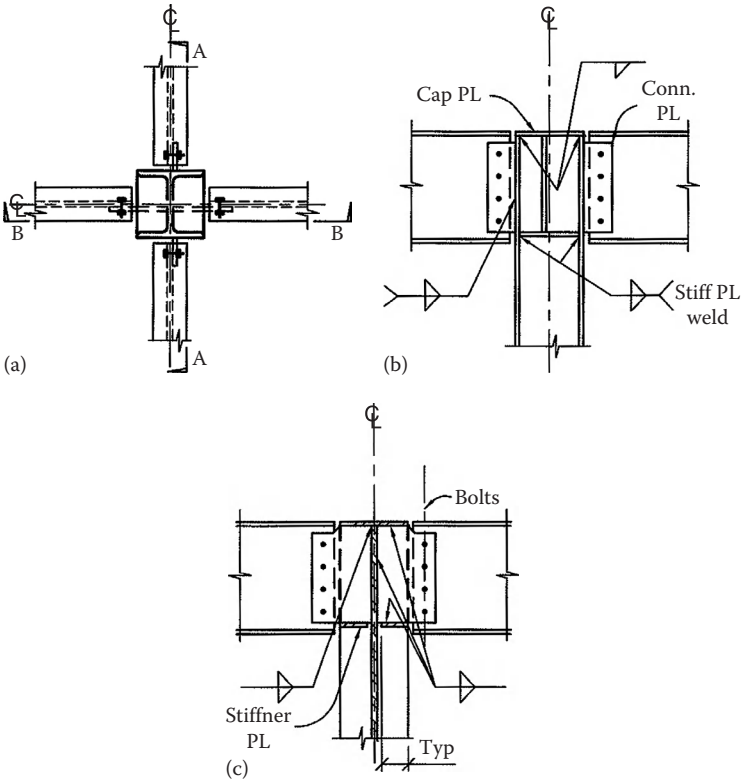


FIGURE 10.19 (BC-3) Simple connections at roof: (a) plan; (b) section A; (c) section B.

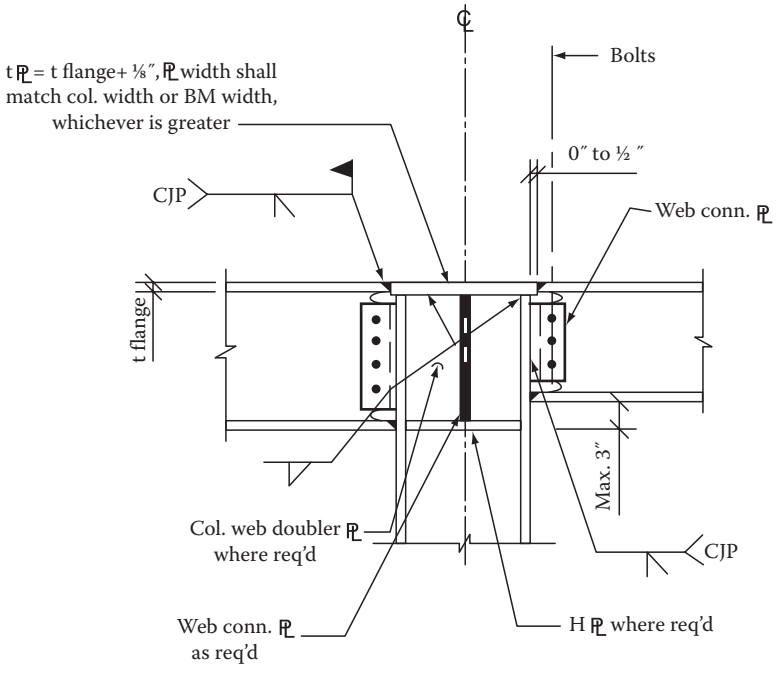


FIGURE 10.20 (BC-4) Rigid connection at top of column.

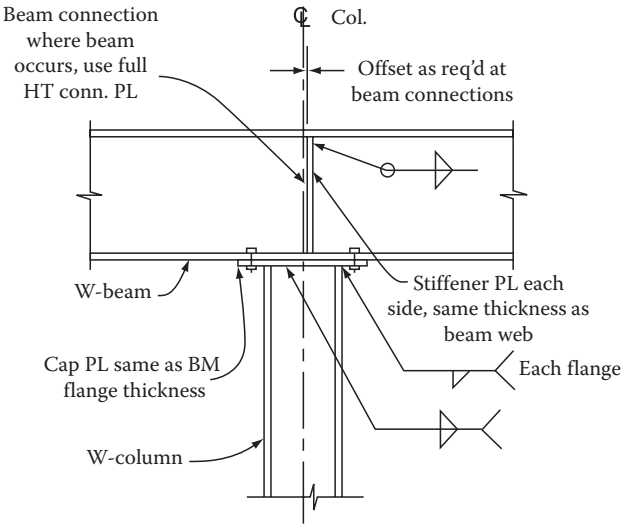


FIGURE 10.21 (BC-5) Beam continuous over WF column.

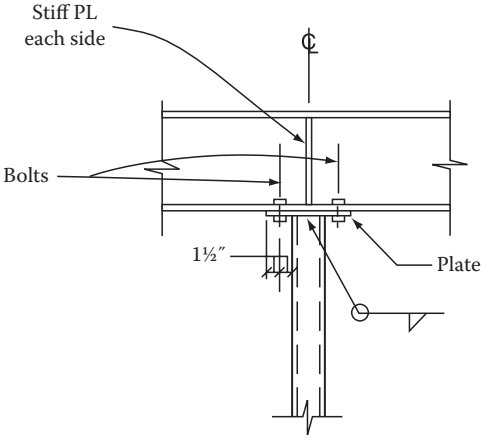


FIGURE 10.22 (BC-6) Beam continuous over HSS column.

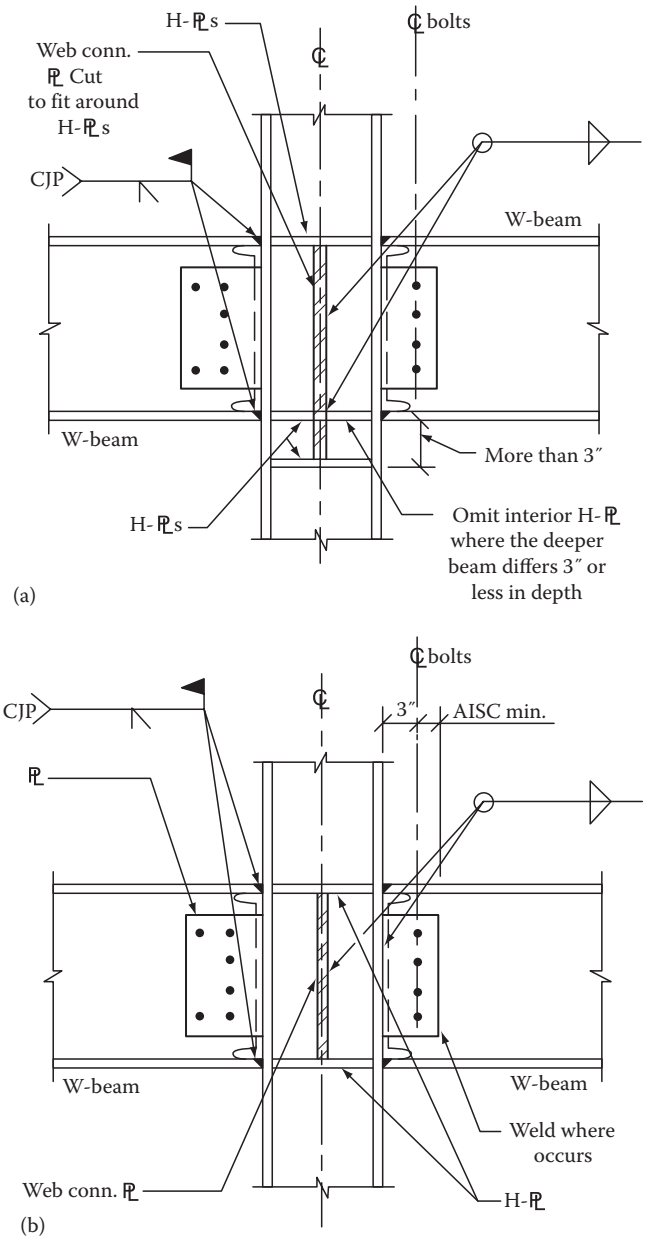


FIGURE 10.23 (BC-7) Moment connection to beam flange: (a) beams of unequal depth; (b) beams of equal depth.

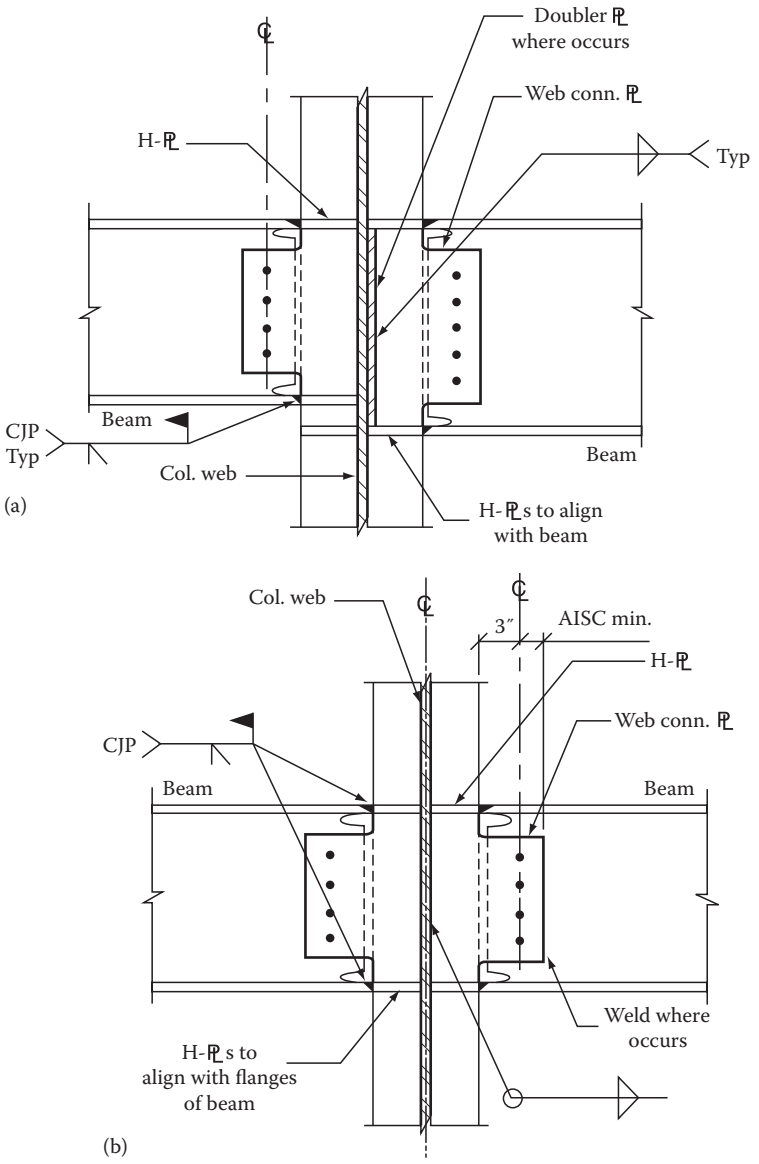


FIGURE 10.24 (BC-8) Moment connection to column web: (a) beams of unequal depth; (b) beams of equal depth.

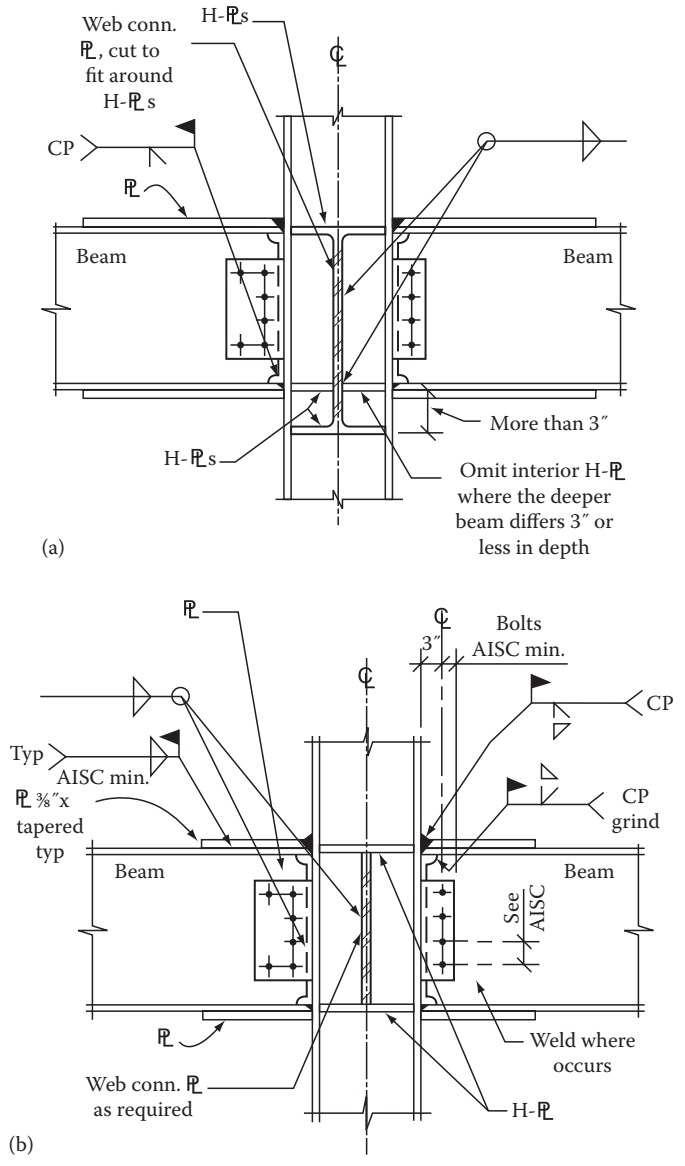


FIGURE 10.25 (BC-9) Moment connection to column flange, beam with plates: (a) beams of unequal depth; (b) beams of equal depth.

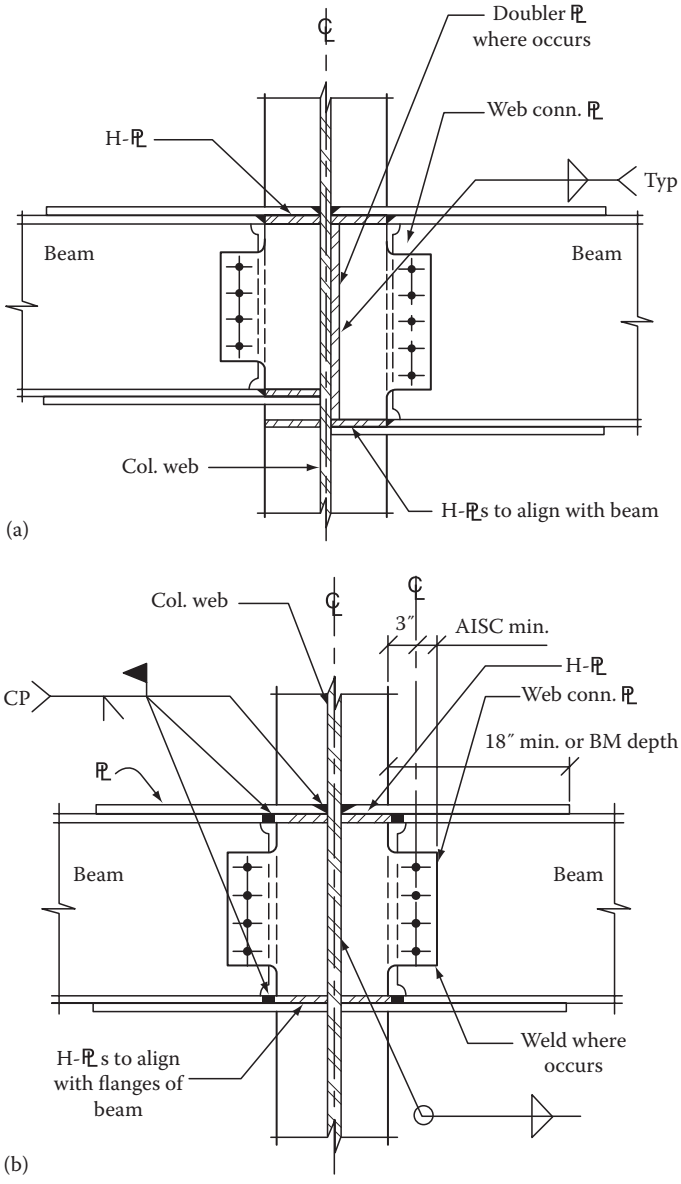


FIGURE 10.26 (BC-10) Moment connection to column web, beam with plates: (a) beams of unequal depth; (b) beams of equal depth.

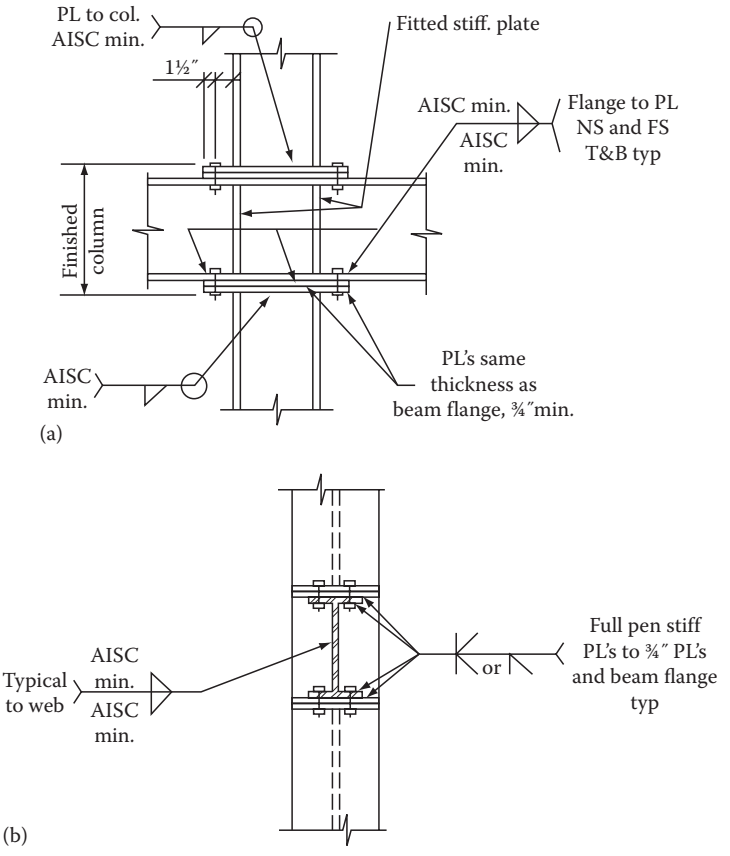


FIGURE 10.27 (BC-11) Interrupted column to continuous beam: (a) elevation; (b) section.

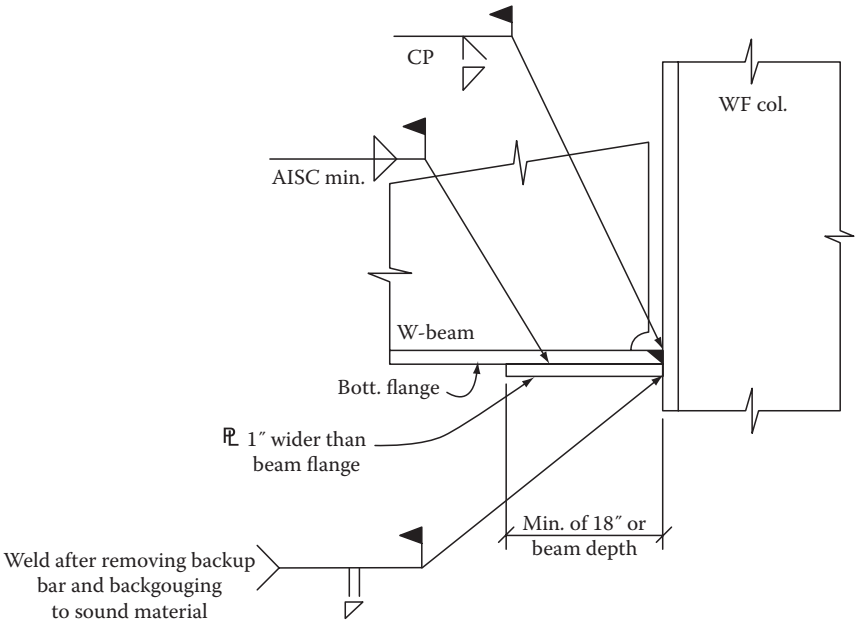


FIGURE 10.28 (BC-12) Bottom flange weld detail.

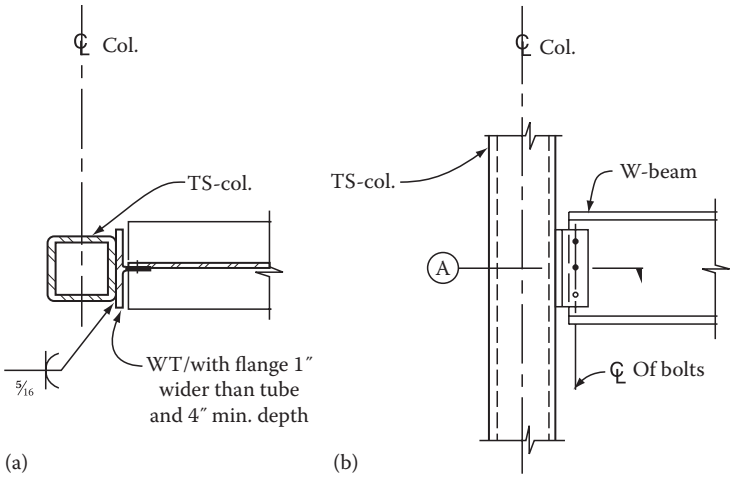


FIGURE 10.29 (BC-13) Wide-flange beam to TS-column: (a) plan section A; (b) elevation.

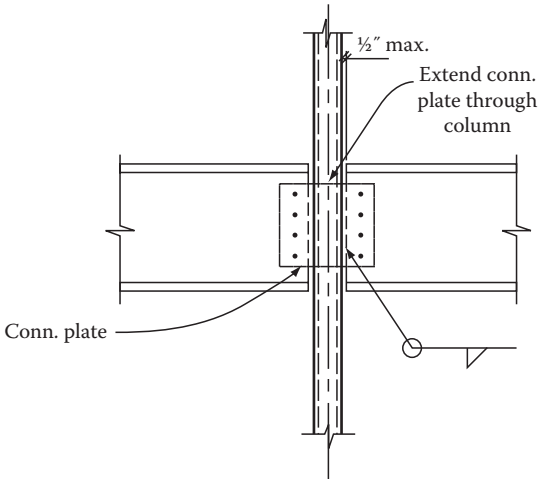


FIGURE 10.30 (BC-14) Wide-flange beam to pipe or TS column.

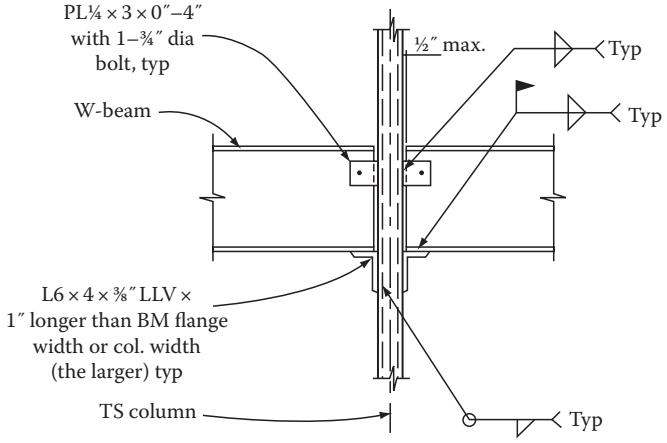


FIGURE 10.31 (BC-15) Wide-flange beam to TS column, unstiffened seated connection.

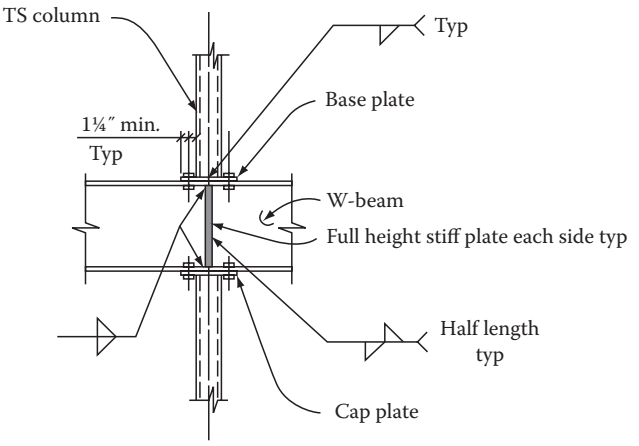


FIGURE 10.32 (BC-16) Interrupted TS column to continuous beam.

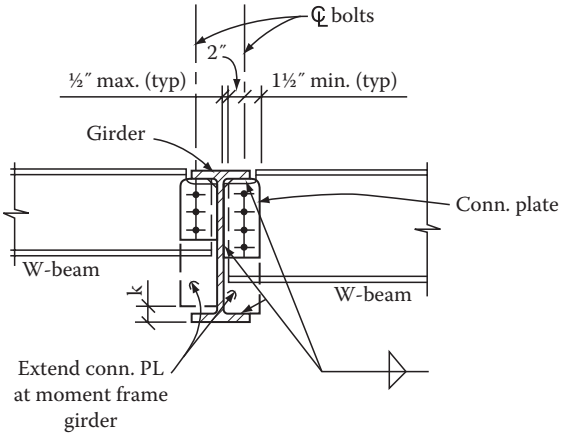


FIGURE 10.33 (BB-1) Typical beam.

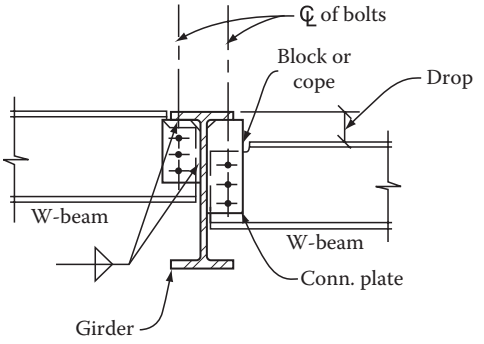


FIGURE 10.34 (BB-2) Dropped beam.

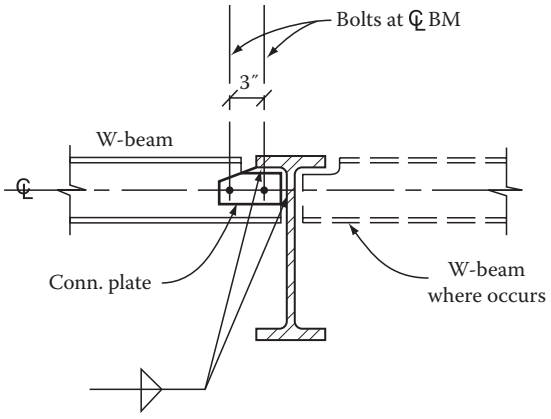


FIGURE 10.35 (BB-3) Beam 4–7 in. deep.

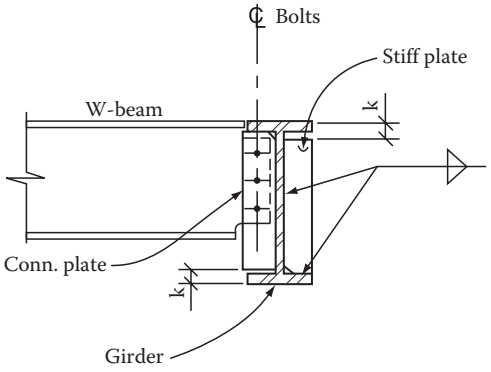


FIGURE 10.36 (BB-4) One-sided beam.

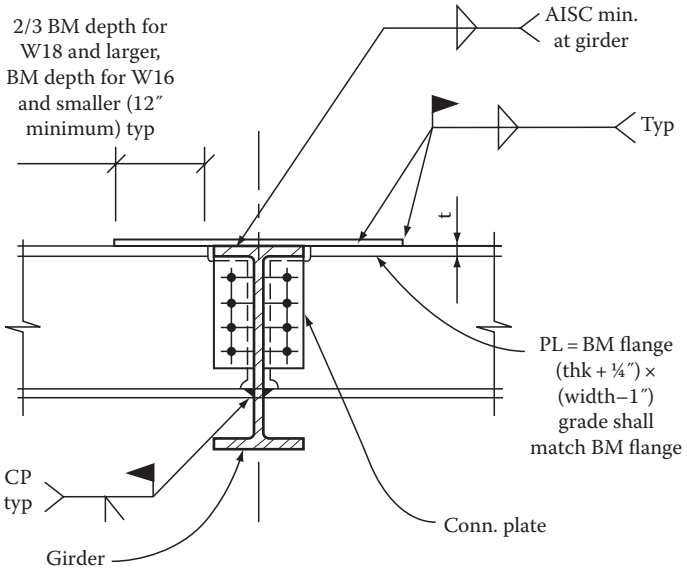


FIGURE 10.37 (BB-5) Moment connection, girder deeper than beams.

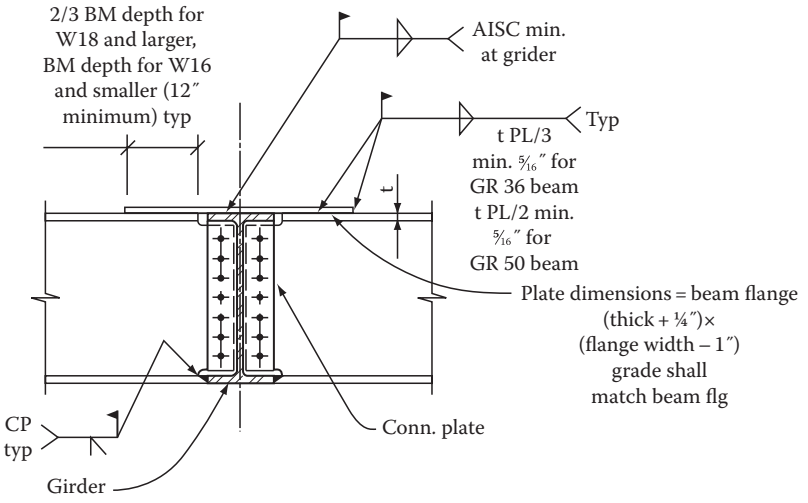


FIGURE 10.38 (BB-6) Moment connection, girder and beams of equal depth.

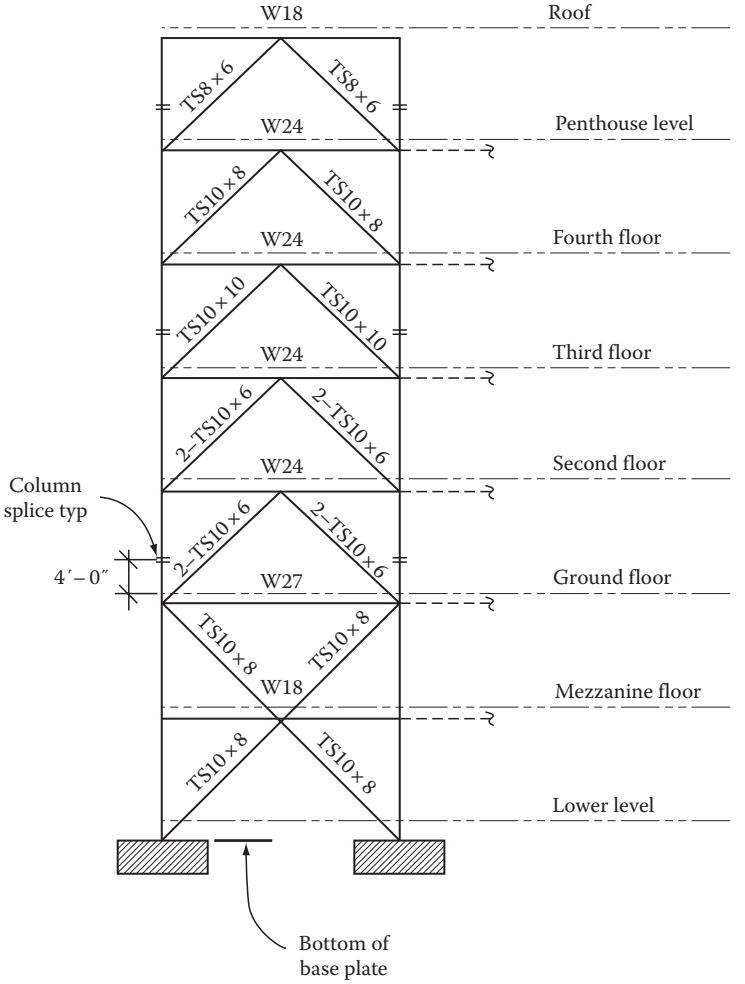


FIGURE 10.39 (BR-1) Schematic elevation of braced frame with story-high chevron inverted V-braces.

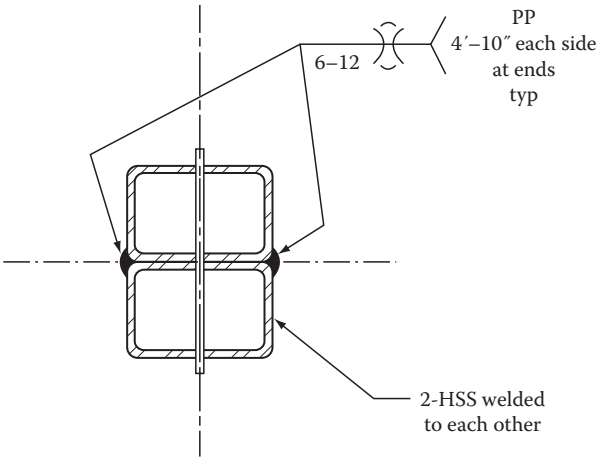


FIGURE 10.40 (BR-2) Built-up brace.

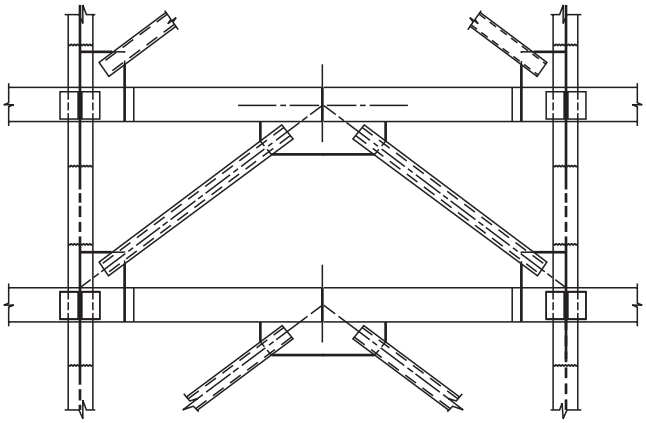


FIGURE 10.41 (BR-3) Schematic elevation of OCBF.

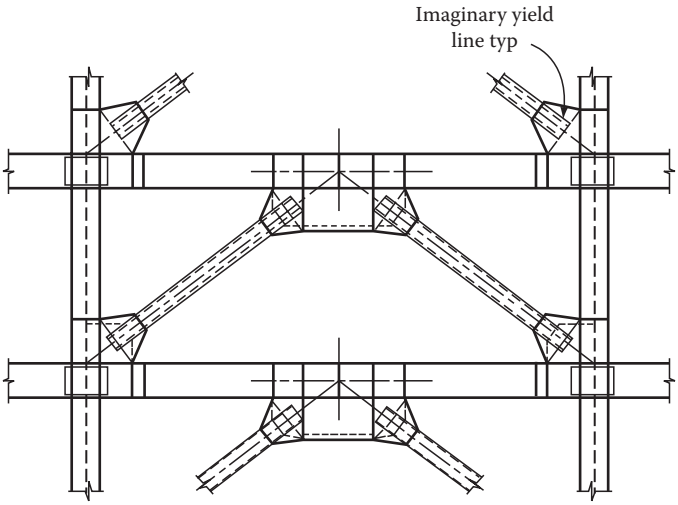


FIGURE 10.42 (BR-4) Schematic elevation of SCBF showing imaginary yield lines in gusset plates. Note that gusset plate sizes for SCBF are substantially larger than those for OCBF.

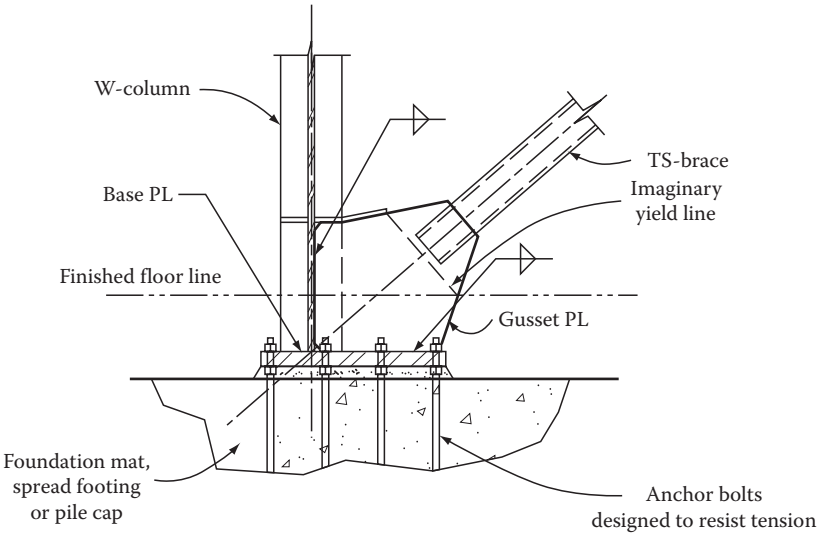


FIGURE 10.43 (BR-5) Brace connection at foundation. *Note*: Provisions for resisting horizontal forces not shown for clarity.

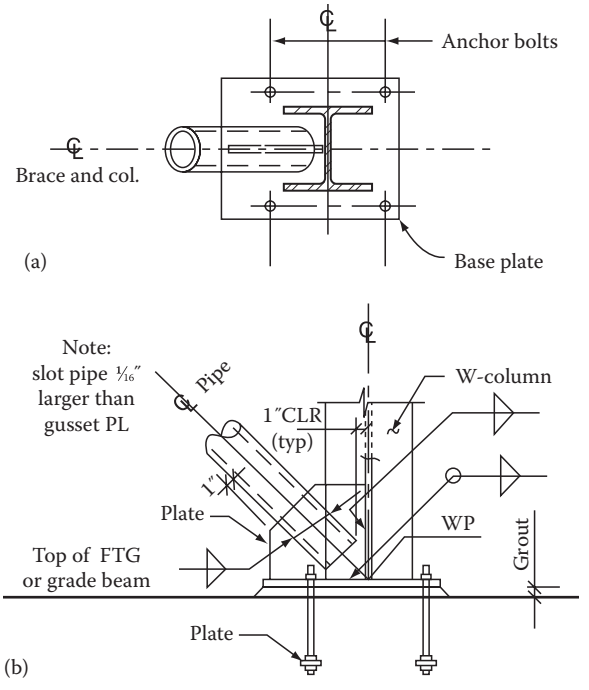


FIGURE 10.44 (BR-6) Brace attached to column web.

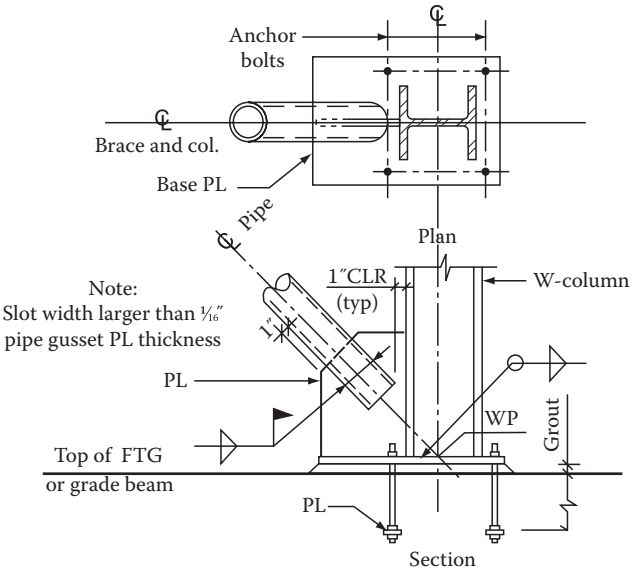


FIGURE 10.45 (BR-7) Brace attached to column flange.

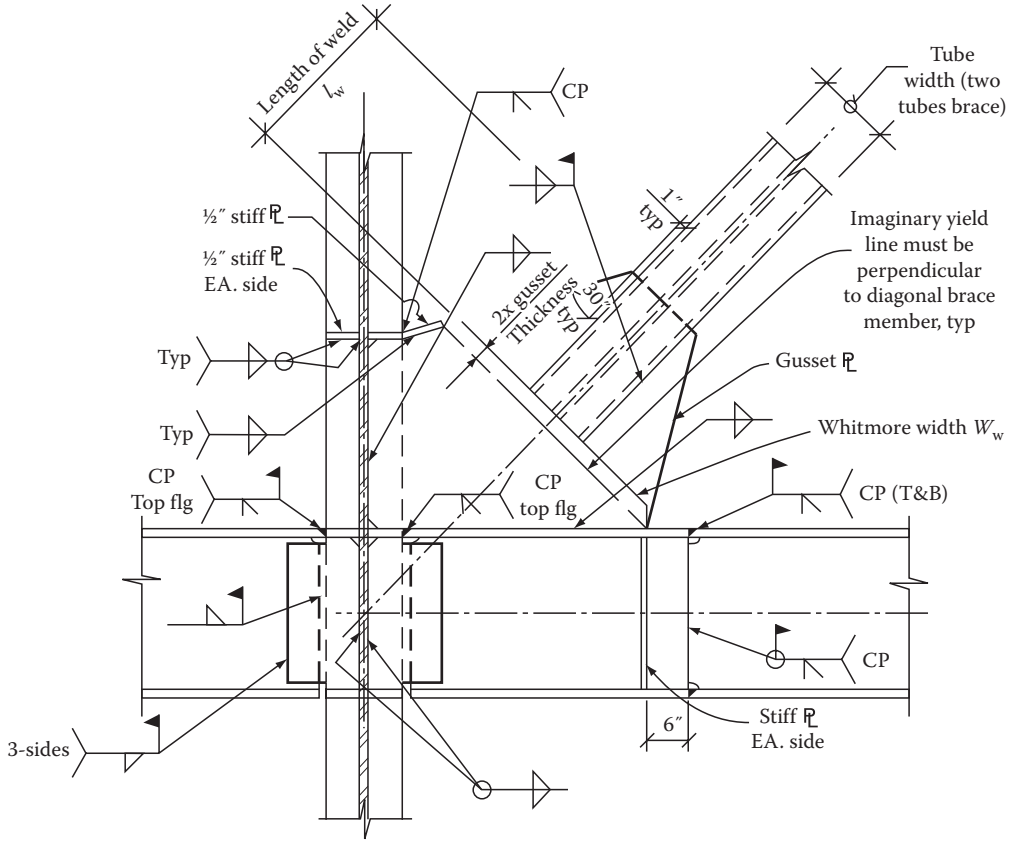


FIGURE 10.46 (BR-8) Brace-to-column connection, SCBF. *Notes*: 1. Yield line intersects free edge of gusset plate and not beam flange. 2. Whitmore effective width W_w = tube width + $2l_w$ (tan 30). 3. Welding of beam flanges is for resisting drag forces.

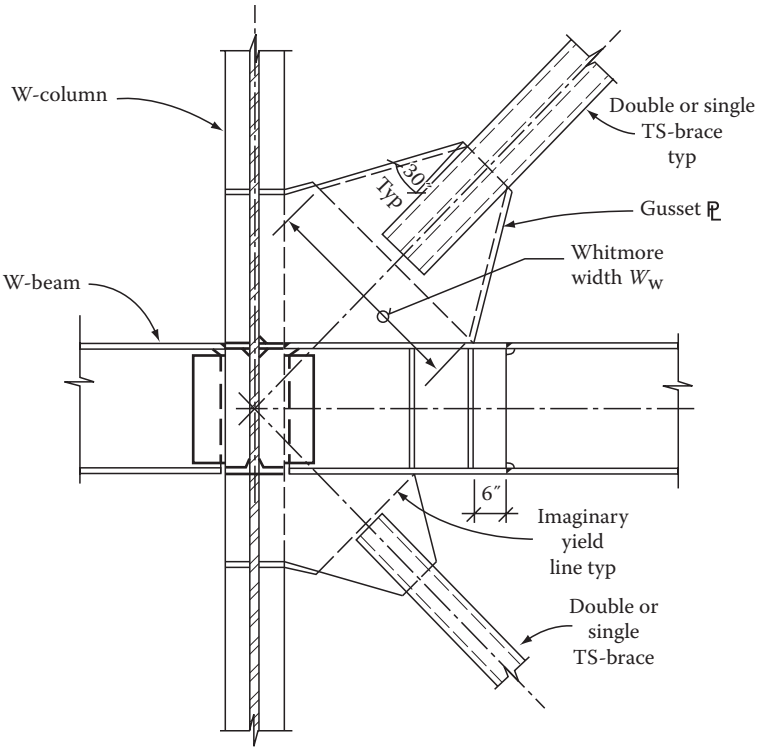


FIGURE 10.47 (BR-9) SCBF connection at column.

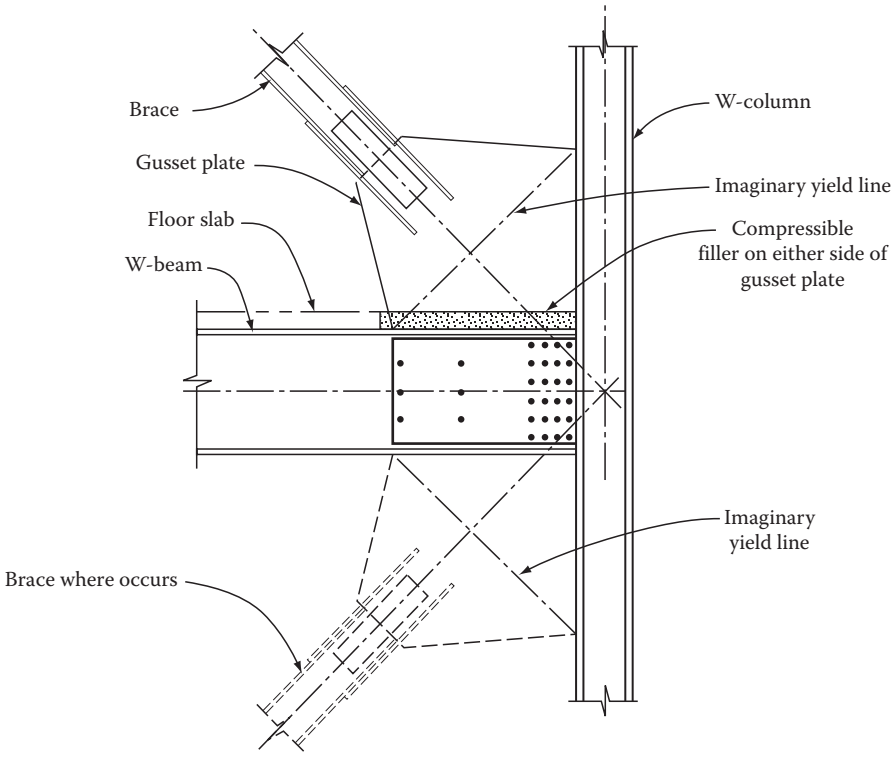


FIGURE 10.48 (BR-10) Gusset plate isolation from slab.

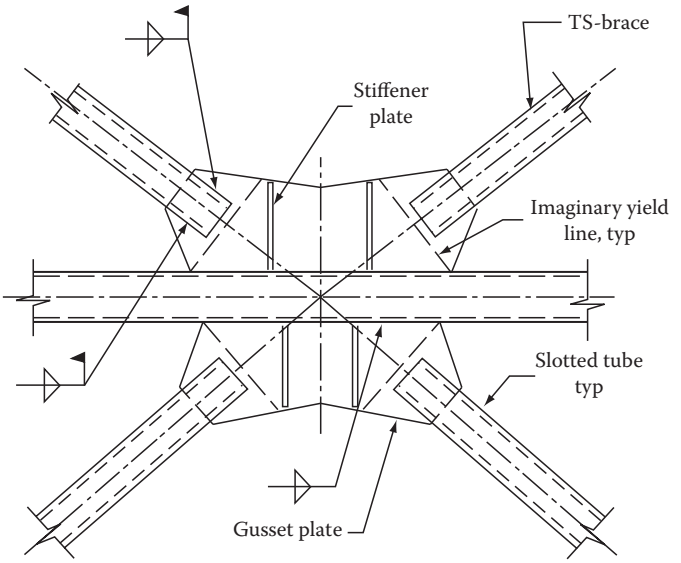


FIGURE 10.49 (BR-11) X-brace to beam connection.

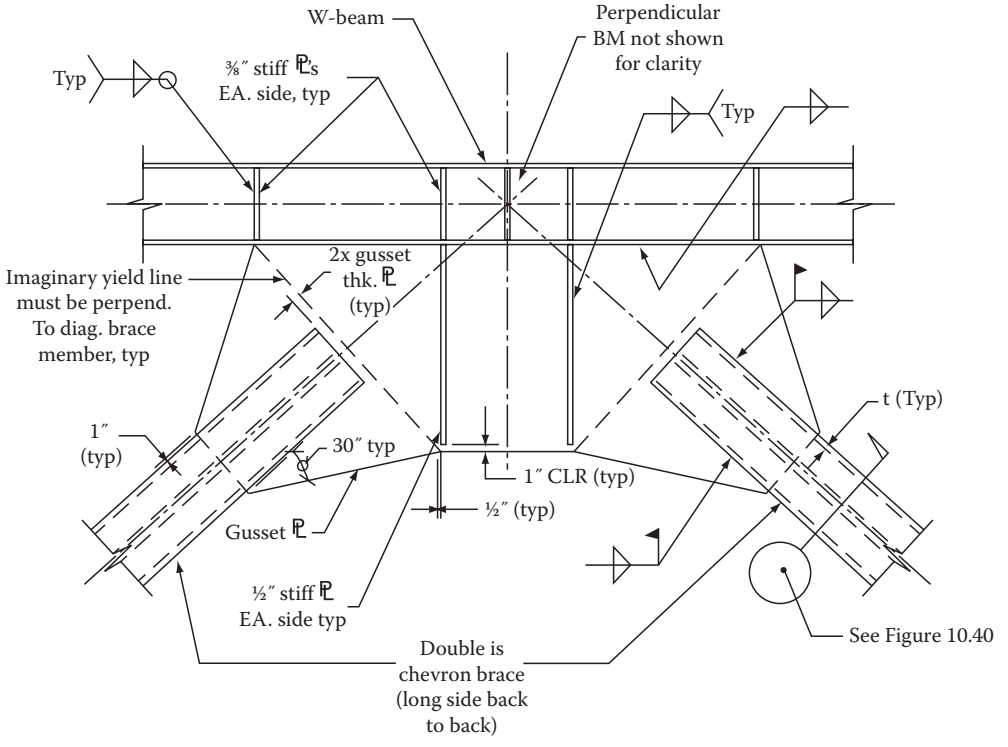


FIGURE 10.50 (BR-12) Chevron brace-to-beam connection. *Notes*: 1. Chevron braces designed to SCBF requirements need not be designed for the 1.5 load amplification factor required for OCBF. 2. Beam depth shown is not-to-scale.

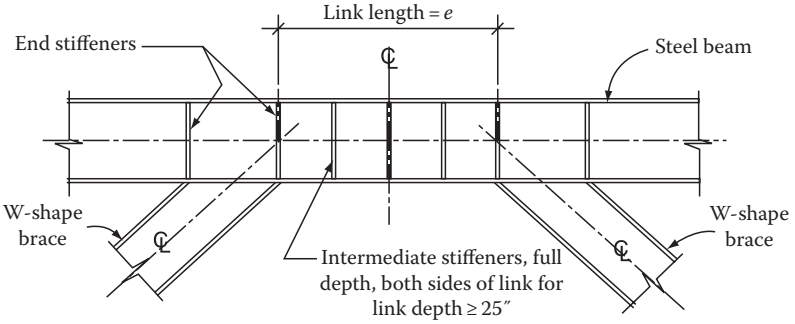


FIGURE 10.51 (EBF-1) Eccentric braced frame, EBF, with WF bracing.

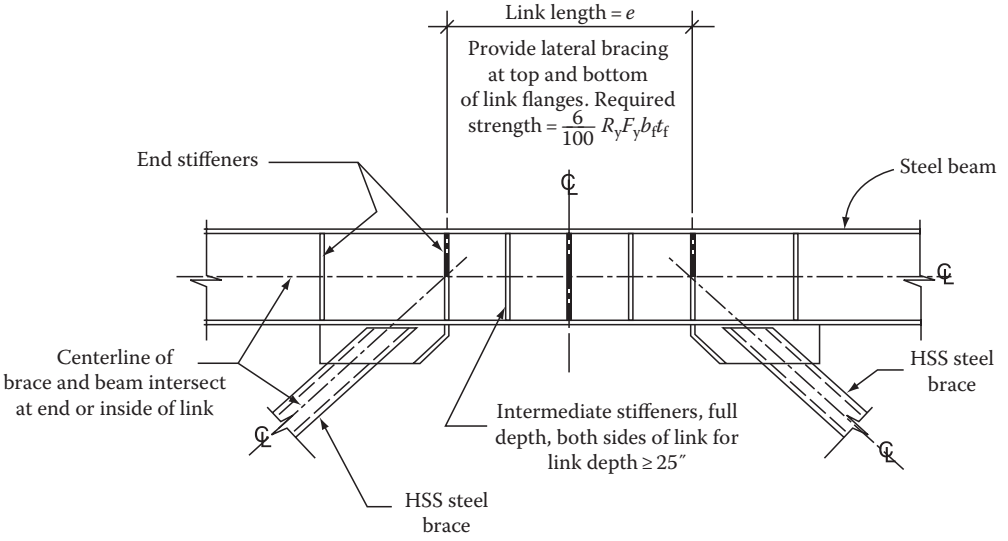


FIGURE 10.52 (EBF-2) EBF with HSS bracing.

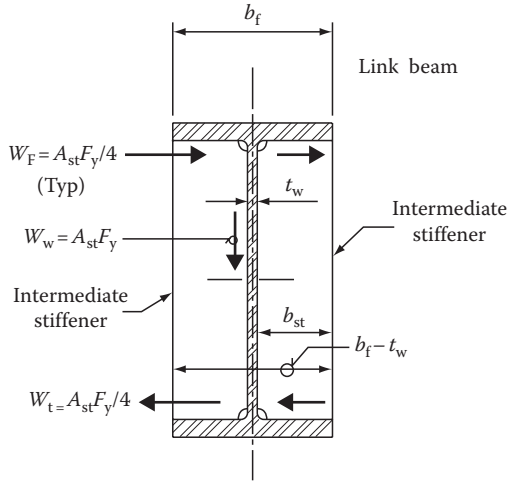


FIGURE 10.53 (EBF-3) Link beam intermediate web stiffeners. *Note*: Total strength of welds, W_F, connecting stiffeners to link flanges is equal to $A_{st}F_y$, where as strength of welds W_w connecting stiffener to link web is $A_{st}F_y$. Area of stiffener = A_{st}.

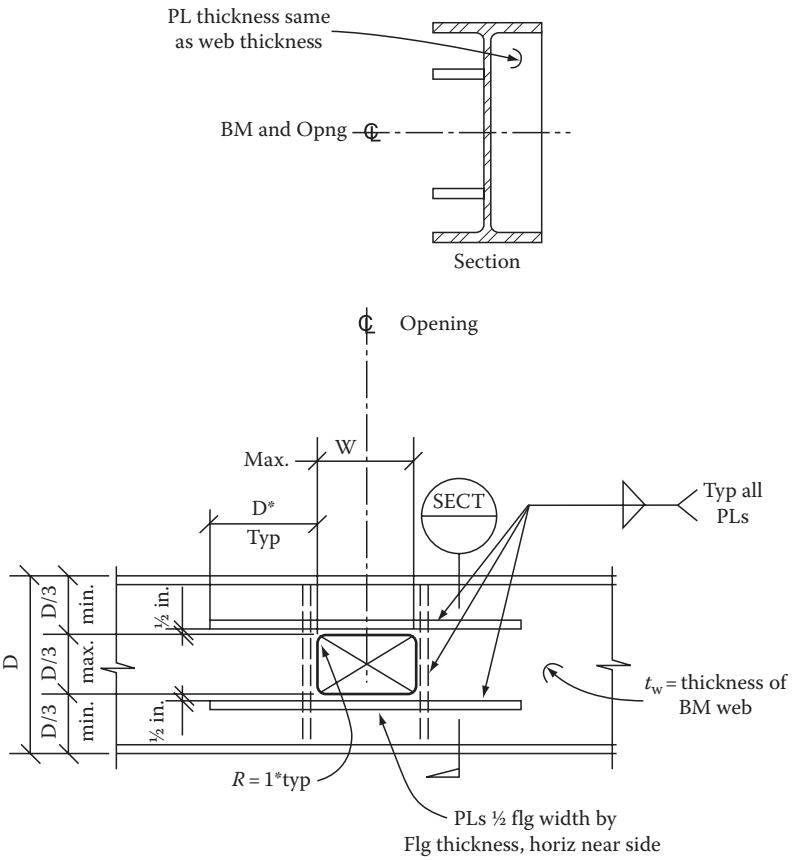


FIGURE 10.54 (M-1) Rectangular opening through beam web.

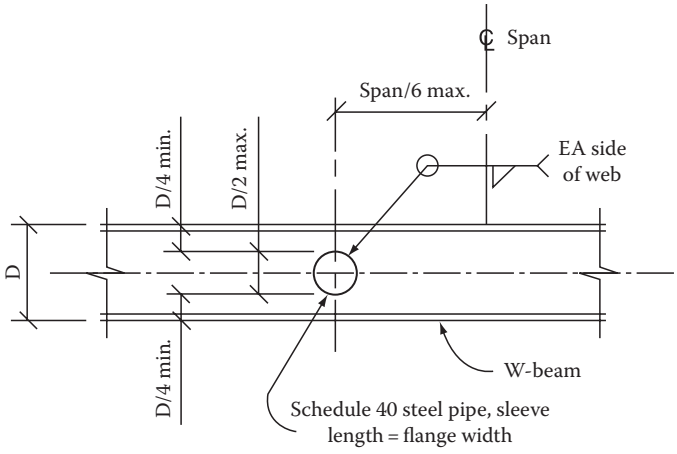


FIGURE 10.55 (M-2) Round opening through beam web. *Note*: Where round openings are D/6 or less, reinforcing shown is not required.

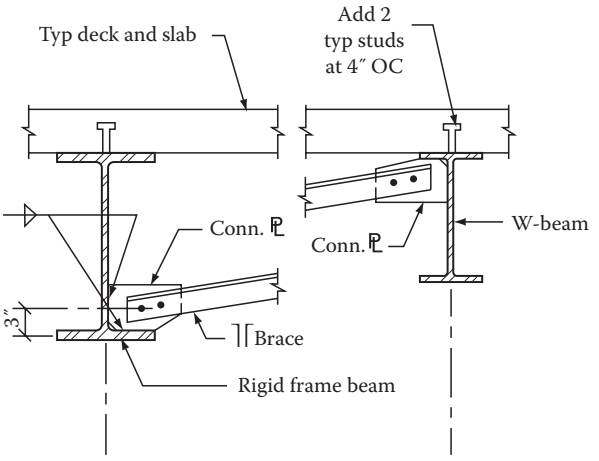


FIGURE 10.56 (M-3) Lateral bracing of frame beam.

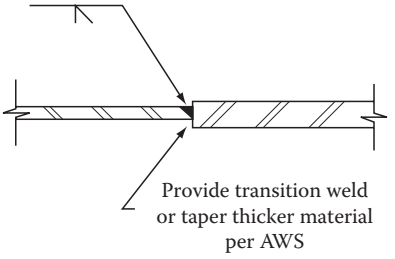


FIGURE 10.57 (M-4) Transition between sections of different thickness.

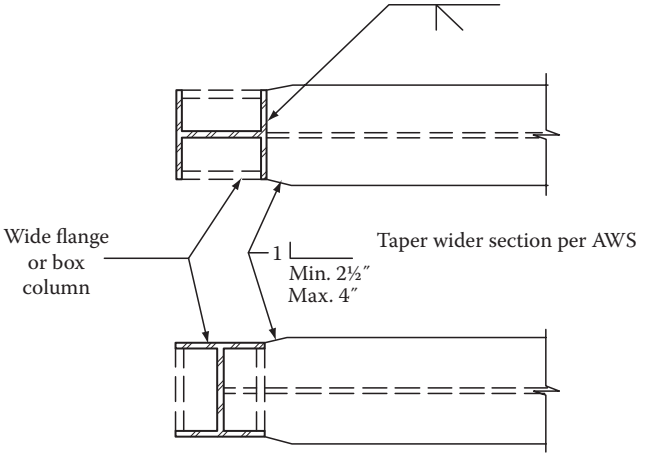


FIGURE 10.58 (M-5) Transition between sections of different widths.

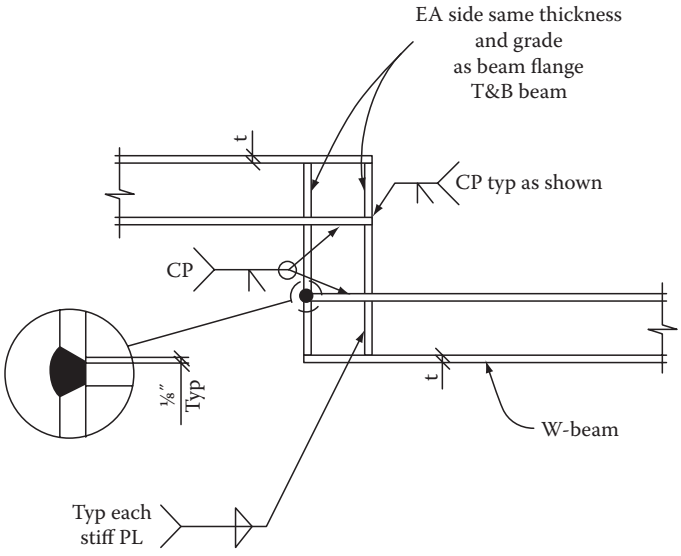


FIGURE 10.59 (M-6) Dog-leg beam.

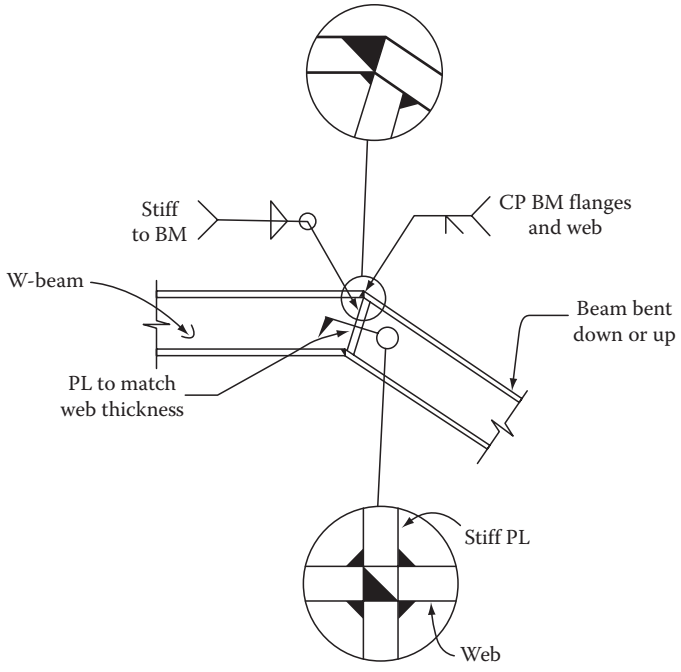
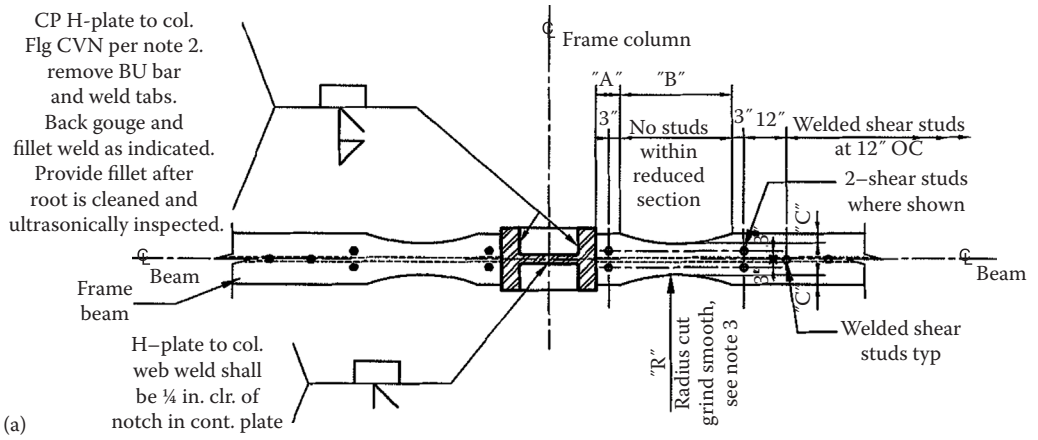
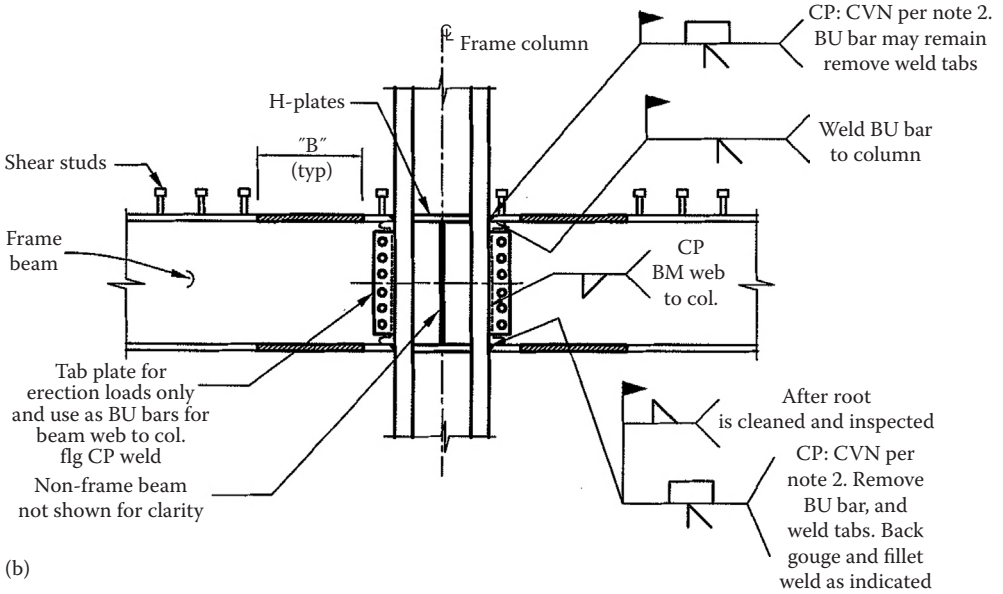


FIGURE 10.60 (M-7) Bent beam.





(c)

Beam	"A"	"B"	"C"	"R"
W24 × 76	4½"	13⅝"	1⅞"	17$\frac{3}{16}$"
W24 × 84	4½"	13¾"	1⅞"	17½"
W27 × 94	5"	17½"	2"	13½"
W30 × 104	5¼"	18½"	2⅛"	22⅝"
W30 × 116	5¼"	18½"	2⅛"	22⅝"

FIGURE 10.61 (PQC-1) Reduced beam section, RBS, connection. (a) plan; (b) elevation; (c) geometric parameters for typical beams. *Notes*: 1. (a) Full ultrasonic inspection shall be provided on all complete penetration groove welds contained at joints and splices (tested 100%). (b) Base metal thicker than 1½ in., when subjected to through thickness weld shrinkage strains, shall be ultrasonically inspected for discontinuities, and shall be accepted or rejected on the basis of the defect rating in accordance with the (larger reflector) criteria of the uniform building code standard. 2. All groove weld electrodes must be rated for CVN of at least 20 ft-lb at –20˚F, and 40 ft-lb at 70˚F. 3. Beam flange radius edges shall be ground in the direction of flange length to have a surface roughness value less than or equal to 1000 as defined in ANSI/ASME B46.1. All corners shall be rounded to minimize notch effects.

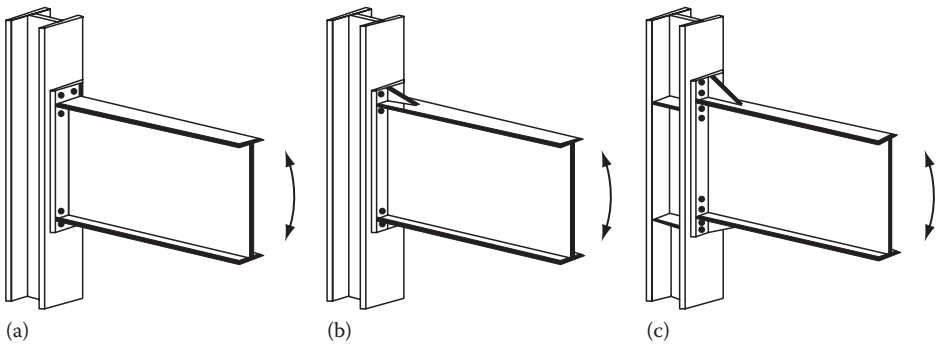


FIGURE 10.62 (PQC-3) Extended end plate moment connections: (a) four bolt unstiffened; (b) four bolt stiffened; (c) eight bolt stiffened.

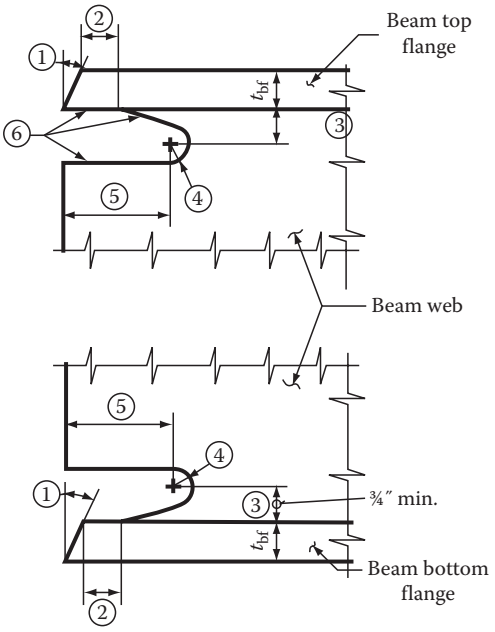


FIGURE 10.63 Special seismic weld access hole for WUF-W moment connection. *Notes*: 1. Bevel as required for selected groove weld. 2. Larger of t_{bf} or ½ in. (plus ½t_{bf} or minus ¼t_{bf}). 3. ¾t_{bf} to t_{bf}¾ in minimum (±¼ in.). 4. ⅜ in. minimum radius. 5. 3t_{bf} (±½ in.). Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25″.

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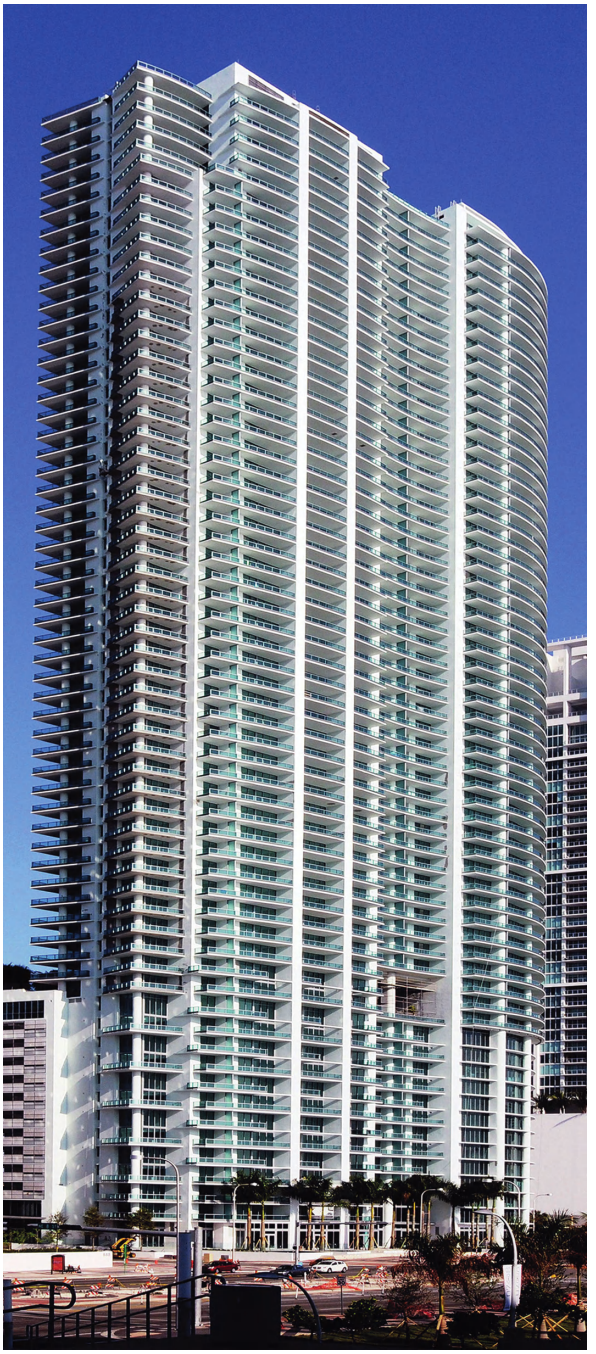
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The welfare of the earth and its inhabitants is a defining theme for the 21st century. Structural engineering has a significant role to play as the world faces these challenges. In fulfilling this function, building engineers continue the age-old human endeavor to provide society with structures that protect, serve, and inspire mankind. From pioneering new systems for better buildings with economy, to ensuring the safety of human life from nature's wrath, to stabilizing implausible forms to defy gravity and lateral loads, building engineers venture to create livable space from humanity's dreams and ideas. Building engineering requires a comprehensive understanding of building assembly and an appreciation of how forces are resisted within the structure and eventually by the earth.

In the following pages, projects showcasing the diversity of modern challenges faced by structural engineers are described. Structural engineering for the illustrated projects is by DeSimone Consulting Engineers.





900 BISCAYNE BOULEVARD

At 65 stories and 750 ft tall, this project contains approximately 1,800,000 sf of space, including 516 luxury condominium units on 47 floors, 70,000 sf of office space on 11 floors, 15,000 sf of retail in the building's base, and garage parking for 815 cars. The residential lobby is 18 ft aboveground and there is a separate lobby and porte-cochere for the office spaces. Penthouse apartments are located on the top five floors. The building also features a lagoon pool and recreation deck, as well as a 7000 sf state-of-the-art fitness center. Foundations are 122 ft deep auger cast piles. The structure consists of prestressed slabs and large, highly efficient shear walls.

Location
Miami, Florida

Client
TERRA

Architect
Revuelta Vega Leon

Structural Engineer
DeSimone Consulting Engineers





1450 BRICKELL AVENUE

This 580,000 sf project creates a 35-story office tower located on the northwest corner of Brickell Avenue and S.E. 15th Road in downtown Miami, Florida. The design includes a ground floor lobby, retail areas, 11 stories of parking, and 21 stories of office space. A recreational deck is provided on the 11th level, south of the parking levels. A double-height office space was utilized at this floor to keep plate levels uniform. The building's exterior design involves the use of a complex interlocking curtain wall system. This project is the first LEED Gold Class A Office Tower undertaken in Miami.

Location
Miami, Florida

Client
Rilea Group

Architect
Nichols Brosch Wurst Wolfe & Associates

Structural Engineer
DeSimone Consulting Engineers



AKRON ART MUSEUM





The expansion of the museum involves a 50,000 sf addition to the existing facility; a 14,000 sf Italian Renaissance building built in 1899 as a post office. The architect proposed to intersect the new and the old building in a dynamic way, by creating a zone of leakage allowing the city to flow into the building and art to flow out of the building. The building is broken up into three primary components: the Crystal, the Gallery-Box, and the Roof Cloud. The Crystal signifies main entry and operates as a space for orientation. It encompasses a 150-seat auditorium, classrooms, library, café, and bookstore. The Gallery Box is an independent space, with long span structure that can be built out freely based on the needs of each exhibit. The Roof Cloud hovers above the building creating a blurred envelope for the museum, and also operates as a horizontal landmark in the city.

Location
Akron, Ohio

Client
Akron Art Museum

Architect
Coop Himmelblau
Westlake Reed Leskosky

Structural Engineer
DeSimone Consulting Engineers



ART GALLERY OF ALBERTA





Located on Sir Winston Churchill Square, the Art Gallery of Alberta (formerly the Edmonton Art Gallery) is a provocative and inviting visual arts center for the city of Edmonton. An existing facility was located on the site, a quarter of which was demolished to allow for the design and construction of this new addition, which creates 27,000 sf of floor space to the facility. The design infills and expands vertically a new steel structure housing gallery spaces, administrative functions, and rooftop gardens. Through materiality and strategic repositioning on the site, the gallery formally and philosophically extends out to the community. The project also utilizes a wide array of "green" features and intelligent engineering strategies. In total, the project provides the gallery with 83,000 sf of usable space.

Location
Edmonton, Canada

Client
Art Gallery of Alberta

Architect
Randall Stout Architects

Structural Engineer
DeSimone Consulting Engineers



ATLANTIS, PHASE I–III (© ANDY ANDERSON.)





The two million square feet project was completed in three phases. Phase I involved a design and upgrade to the resort's existing support structures. Phase II expansion added a new 50,000 sf casino and a 24-story, 1200-room hotel. The hotel component is the tallest structure in the Caribbean. The expansion added another three million-gallon open-air aquarium to the resort grounds. Combined with Phase I, the outdoor aquarium habitat is the largest in the world. Phase III work was completed in 2007. The project added a new 600-key condo hotel, a timeshare resort "Harbourside at Atlantis," and a 250,000 sf convention center featuring the largest ballroom in the Caribbean. The project also expanded the water park and added a Marina Village.

Location
Paradise Island, Bahamas

Client
Kernzer International, Ltd.

Architect
WATG
HKS, Inc.
EDSA

Structural Engineer
DeSimone Consulting Engineers



THE COSMOPOLITAN RESORT AND CASINO



Located on a 7 ac site with minimal strip frontage, this 6.5 million square feet project is adjacent to the Jockey Club and between the Bellagio and MGM CityCenter. The limited site area forced a nonconventional vertically stacked solution. Parking was pushed below grade, slurry walls were utilized, and the mat foundation was precompressed using micro-piles to prevent soil rebound. The five-story podium was structured using steel and contains convention and ballroom space, a 1800-seat theatre, retail space, and casino areas. Lateral stability at these levels is provided by steel shear wall panels. Above the podium are two 50-story towers containing hotel guest rooms and residential units. The towers are structured using 9 in. post-tensioned flat slabs and full-width shear walls between room demise walls. Tower levels transfer loads to lot-line shear walls via a 7 ft thick sky mat located at the top podium level.

Location
Las Vegas, Nevada

Client
The Related Companies

Architect
Arquitectonica
Friedmutter Group

Structural Engineer
DeSimone Consulting Engineers









FISHER CENTER (© ESTO PHOTOGRAPHICS INC.)

This performing arts center houses Bard College's opera, dance, and musical productions as well as the theatrical teaching facilities. The 860-seat, 64,000 sf performance auditorium and rigging tower are made of tall, slender, cast-in-place concrete walls. The first and second balcony seating sections cantilever from these walls and are carefully balanced with the hallway floors outside the auditorium. The 420-seat teaching theater is housed in a reinforced concrete masonry auditorium and rigging tower. The exterior of the buildings, which envelop the functional boxes, embodies the signature Frank Gehry metal "skin." The complex, compound, sinuous curves, which are computer generated, define the form to which the thin stainless steel sheathing follows. Behind the "skin" are custom built steel members placed at 10 ft increments. The final product is a unique harmony of steel and concrete with the versatility to house almost any size and type of performance.

Location
Annandale-on-Hudson, New York

Client
Bard College

Architect
Gehry Partners

Structural Engineer
DeSimone Consulting Engineers





FOUR SEASONS HOTEL AND TOWER (© MARK SURLOFF PHOTOGRAPHY.)

At 800 ft tall and two million square feet, the Four Seasons Hotel, Miami, is the tallest building in the United States south of New York City and the fifth tallest residential building in the world. This mixed-use structure consists of a 65-story tower that contains office, hotel, and condominium space, and a 500,000 sf podium that accommodates a 1,000-space parking garage. Using a reinforced concrete structure, the design incorporates core shear walls, post-tensioned slabs and perimeter columns. Spans between columns reach 40 ft. Engineering services for this building included an extensive wind tunnel study, sophisticated computer lateral analysis, state-of-the-art foundation load test, creep and shrinkage analysis, settlement analysis, and post-tension slab design.

Location
Miami, Florida

Client
Millennium Partners

Architect
Handel Architects
Bermello, Ajamil & Partners

Structural Engineer
DeSimone Consulting Engineers





FOUR SEASONS TOWER (© MARK SURLOFF PHOTOGRAPHY.)

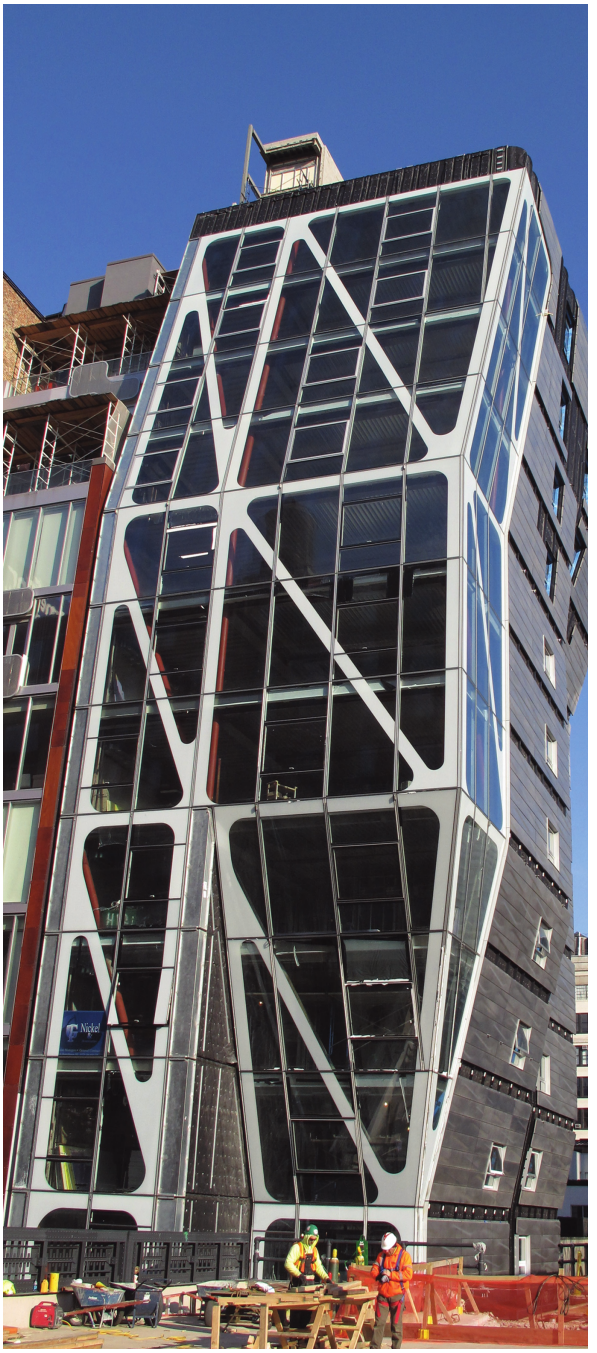
A 40-story mixed-use structure located just off Union Square in the heart of San Francisco, the building contains a five-star hotel, with 277 guest rooms and 142 luxury condominiums. The podium level houses both retail and parking. The building is located along a major fault line in one of the highest category seismic zones in the United States. The innovative and irregular structure raised many engineering challenges. DeSimone performed a dynamic analysis on the structure as a whole, as well as its subsections. Nonlinear viscous dampers were used to limit wind impact. A limit state design approach was utilized to resist seismic forces associated with a Maximum Credible Seismic Event.

Location
San Francisco, California

Client
Millennium Partners
WDG Ventures

Architect
Handel Architects
Del Maro Campo

Structural Engineer
DeSimone Consulting Engineers





HL 23

Located in the Chelsea section of Manhattan, the site sits next to and under the High Line historic railway district, which bisects the site. The project contains 14 floors and 45,000 sf of high-end residential space that features generous ceiling heights and expansive column-free zones. The building will have a single-story basement level, a ground floor gallery space, and loft floors above. Above the fourth floor, the structure begins to project outward, creating an undulating façade that cantilevers over the High Line. As well, the building's south façade features an expressed structural diagonal that serves to "hang" the cantilevered floors from above from the interior columns. The project seeks a Gold LEED certification.

Location
New York, New York

Client
Alf Naman Real Estate

Architect
Neil M. Denari Architects

Structural Engineer
DeSimone Consulting Engineers





IAC/INTERACTIVECORP HEADQUARTERS (© ROYCE DOUGLAS/ EMPORIS.COM)



This 10-story, 190,000 sf office building houses public areas and restaurants in addition to basic office functions. Reminiscent of boat sails, the billowing exterior design produces dramatic angles. The complex exterior was created using sculpted reinforced concrete structure delicately clad with an all-glass curtain wall system. A highly integrated environment, the form is broken down into five, two-tier modules, twisted to create the building's dynamic form. Concrete provided the ideal structural solution and an innovative way to achieve the complex set requirements. The sculptural glass façade was created using complex, multidirectionally curved curtain wall panels, supported by structural slabs. The slabs cantilever varying distances beyond the perimeter columns and are sloped to conform to the design. Interior spans exceed 35 ft in many locations to accommodate the open office environment.

Location
New York, New York

Client
The Georgetown Company

Architect
Gehry Partners
Adamson Associates

Structural Engineer
DeSimone Consulting Engineers



LAS VEGAS CITY HALL

Located in the downtown Union Park neighborhood, the new eight-story City Hall facility will boast a modest area of approximately 268,000 sf and includes 70 restricted underground parking spaces. The new LEED Silver facility will become the home for the mayor's office, the city council, and other governmental offices.

Location
Las Vegas, Nevada

Client
City of Las Vegas

Architect
Elkus Manfredi Architects
JMA Architecture Studios

Structural Engineer
DeSimone Consulting Engineers



MANDARIN ORIENTAL (© NESTOR PEREZ/EMPORIS.COM)





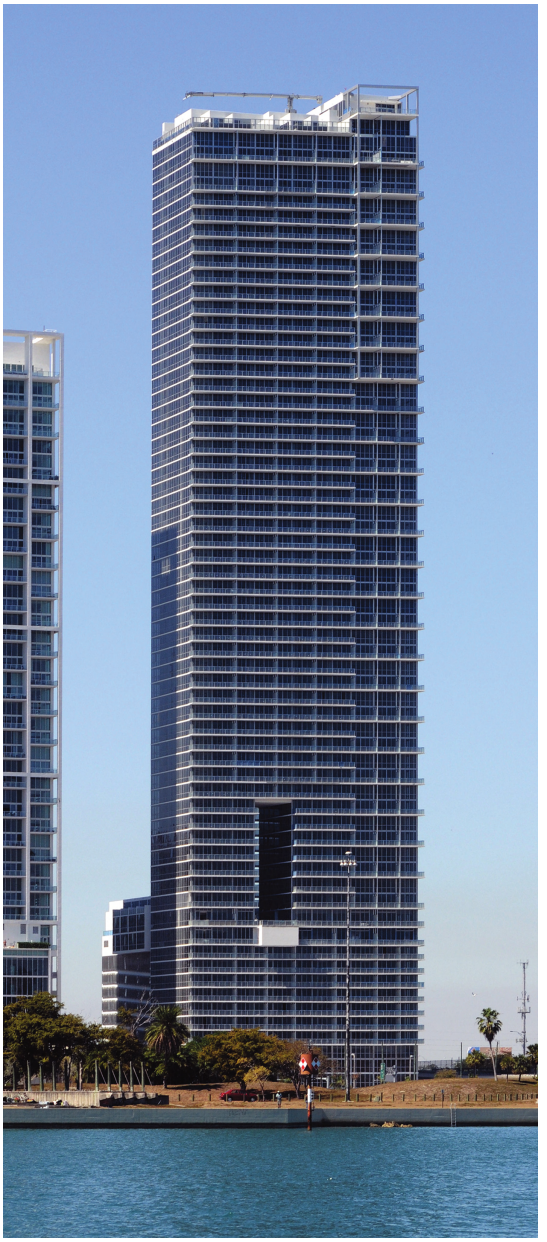
Located on Brickell Key waterfront, this six-star hotel contains 329 guest rooms, a business center, ballroom, restaurant, and health club facilities. The building's roofline is gradually terraced, ranging in height from 17 to 21 stories, to abstractly resemble the company's Mandarin sampan logo. The ballroom facility is located in a separate structure, situated across from the main entry porte-cochere, connecting back to the main complex by a bridge at the second story. The building's concrete structure utilizes both conventional and post-tensioned slabs, and shear walls for lateral support. A pile foundation was required for support, as the building is located on reclaimed land.

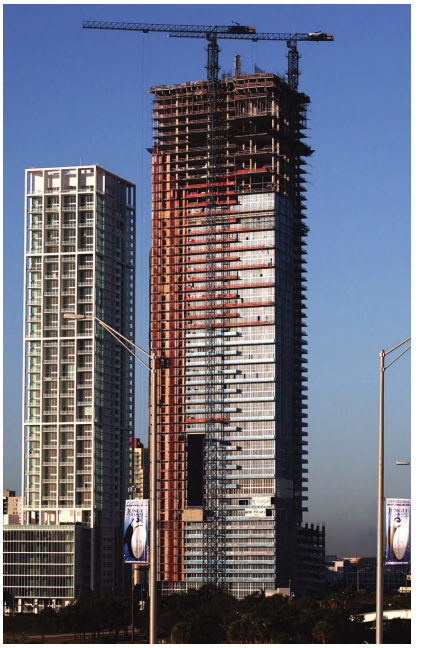
Location
Miami, Florida

Client
Swire Properties
Mandarin Oriental Hotel Group

Architect
RTKL Associates

Structural Engineer
DeSimone Consulting Engineers





MARQUIS

This 67-story, 704 ft tall, one million square feet mixed-use residential development contains 306 luxury condominium units, including 14 townhouses lining the pool deck, a 48-room boutique hotel tower in addition to a street level retail component. Each podium floor contains 34,000 sf while the tower floors above contain 13,000 sf. The superstructure consists primarily of posttension slabs poured monolithically with columns and shear walls. The building is supported on an augercast pile foundation system.

Location
Miami, Florida

Client
Leviev Boymelgreen

Architect
Arquitectonica

Structural Engineer
DeSimone Consulting Engineers







METREON (© JOHN SLAUGHTER/ EMPORIS.COM)



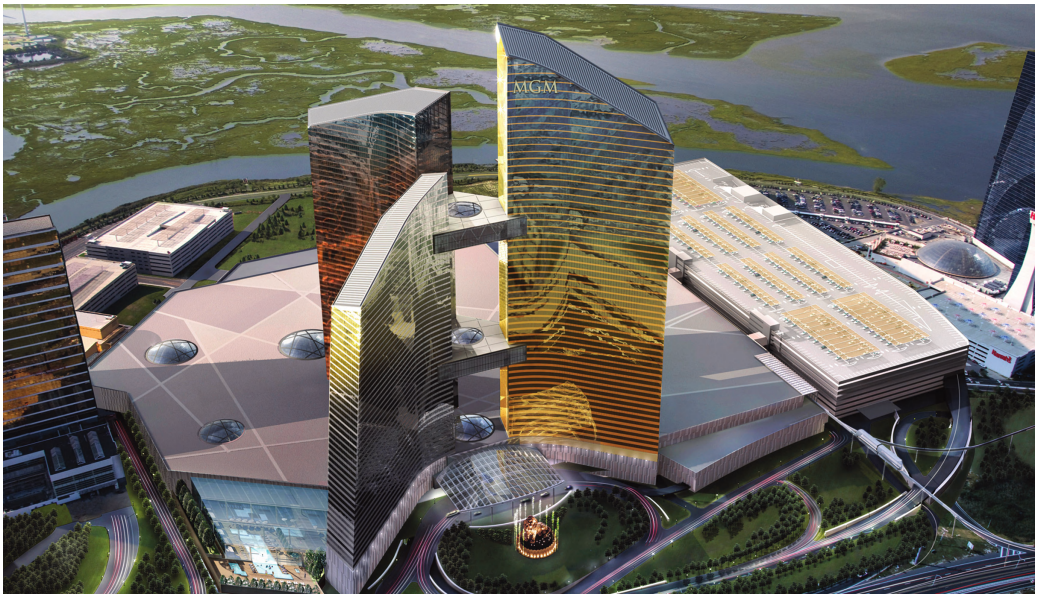
This four-story, 350,000 sf complex accommodates a variety of retail and entertainment functions, and includes space for large volume retailers, theme stores, small local merchants, indoor restaurants, outdoor cafes, and virtual reality attractions. The facility also contains a performance hall, and a 16-theater Sony cinema with IMAX theater. Constructed on top of an underground conference hall, the structure utilizes 140 ft long clear span plate girders. The lateral force resisting system is a special moment resisting frame, which ties into and transfers forces to the existing structure. The project has been designed to meet California's Zone 4 Seismic Requirements.

Location
San Francisco, California

Client
Millennium Partners

Architect
SMWM Architects
Handel Architects

Structural Engineer
DeSimone Consulting Engineers



MGM MIRAGE RENAISSANCE POINTE





This 8.2 million square feet, mixed-use complex creates a premier gaming and resort facility. A two-level, 2.5 million square feet podium spans the site and includes gaming spaces, restaurants, meeting and assembly rooms, retail areas, an event center, theaters, showrooms, a spa/health club, and back-of-house facilities. Above the podium, the project is separated into three hotel towers. Tower A stands 800 ft tall and contains 1.03 million square feet. Tower B stands 679 ft tall and contains 1.08 million square feet. Tower C is 596 ft tall and contains 881,500 sf. In total, the hotels create 2,986,000 sf. The project also includes 3,780,000 sf of structured parking garage, surface parking, and valet parking.

Location
Atlantic City, New Jersey

Client
MGM Grand Development

Architect
Kohn Pedersen Fox Associates

Structural Engineer
DeSimone Consulting Engineers



MIAMI INTERNATIONAL AIRPORT SOUTH TERMINAL EXPANSION



This multiyear upgrade to Miami International Airport expanded the South Terminal. The new terminal includes a 14-gate, 800,000 sf concourse and houses the airport's international terminal. The five-story building is framed with structural steel, concrete shear walls, and composite steel moment frames that act as the wind resisting system. Roofs are curved and wing-shaped, accommodating extensive clerestory glazing, which bathes the terminal with an abundance of natural illumination. The Immigrations and Customs area features an open area with ceiling heights of 35 ft. The design also allows for adequate vertical clearance to accommodate a modern luggage system, people movers, and drive ramps. Other key design elements include a 65 ft window wall and oversize mural/art supporting walls.

Location
Miami, Florida

Client
Dade County Aviation Department
Kerzner International, Ltd.

Architect
Borrelli & Associates
Mateu Carreno Rizo & Partners

Structural Engineer
DeSimone Consulting Engineers





MILLENNIUM PLACE (© STEVEN BROOKE STUDIOS.)

Spanning two city blocks, Millennium Place brings together an exciting hub for shopping, entertainment, fitness, and residential living in one development. The complex features: a 300-room five-star hotel (The Ritz-Carlton, Boston Common), a state-of-the-art Loews Theatre multiplex, a Reebok Sports Club and spa facility, retail space, 350 luxury condominiums, 125 corporate apartments, and 1000 parking spaces. The 3.4 ac site provides 1.8 million sf of usable space. The diversity of programs within the complex required creative structural solutions to accommodate varying column grid and clearance requirements. The 35-story steel-framed south tower was built using top-down construction techniques. A perimeter slurry wall encloses the five-level underground parking garage while serving to protect adjacent buildings and the Orange Line tunnel of the Massachusetts Bay Transportation Authority (MBTA). The north tower is framed in cast-in-place concrete.



Location
Boston, Massachusetts

Client
Macomber Development & Associates

Architect
Handel Architects
CBT Architects

Structural Engineer
DeSimone Consulting Engineers





MILLENNIUM TOWER

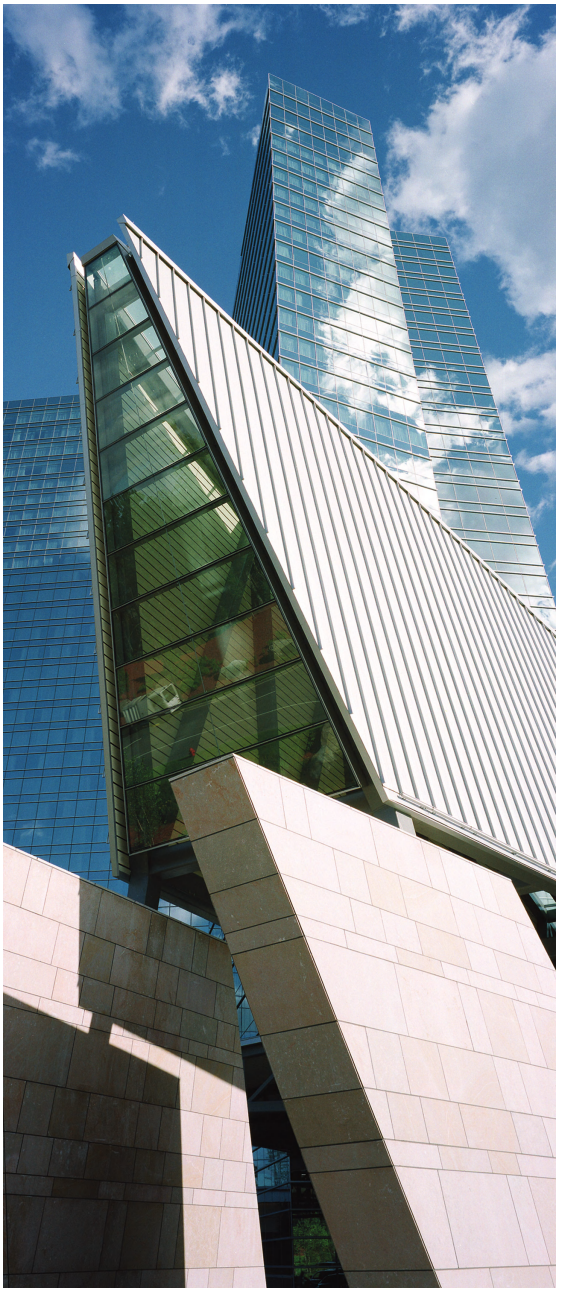
This project is the fourth tallest structure in the city and the tallest reinforced concrete structure in the Western United States. The building contains a 60-story residential tower and an adjacent 12-story residential structure connected by a podium structure. The 645 ft high tower contains 970,500 sf of luxury condominiums and the adjacent building contains 180,000 sf of condominiums and recreational amenities including a swimming pool. Four levels of parking are provided below grade. The tower's foundation mat rests on 950 130 ton piles, each approximately 80 ft in length. The subterranean levels utilize an internally braced soil cement slurry wall system with soldier piles located 5 ft on center. The tower's lateral system comprises a 24 in. thick concrete shear wall core and a partial perimeter moment frame. Outrigger trusses are provided at three intermediate levels to control lateral deflections.

Location
San Francisco, California

Client
Millennium Partners

Architect
Handel Architects

Structural Engineer
DeSimone Consulting Engineers





MOHEGAN SUN (© TIM GRIFFITH.)

The largest private development project on the East Coast, Mohegan Sun's expansion, "Project Sunburst," established the resort/casino as the Northeast's premier entertainment, meeting, and gaming destination. Among the highlights of the four million square feet expansion are: a 34-story, 1,200-room luxury hotel, 100,000 sf of meeting and function space including the Northeast's largest ballroom, 175,000 sf of retail shopping, nine new restaurants, a 10,000-seat arena, 115,000 additional sf of gaming space, a 300-seat cabaret, a world-class spa, a business center, a 16,000 sf indoor pool, and a 12,500 sf outdoor terrace.

Location
Uncasville, Connecticut

Client
Mohegan Tribal Building Department
Trading Cove Associates

Architect
Kohn Pedersen Fox Architects
Rockwell Group

Structural Engineer
DeSimone Consulting Engineers





ONE BAYFRONT PLAZA (© WOODRUFF/ BROWN.)

Located on Biscayne Boulevard in Miami, the project comprises one tower sitting atop a 22-story podium building. The podium building contains lobbies, retail, parking, and office space. The 23rd floor contains a landscape roof deck with pool and spa amenities. The tower is 70 stories tall and accommodates office space, hotel amenities, and guestrooms, while the top two floors provide for penthouse units. In total, the project contains 2.8 million square feet and stands 1,180 ft tall. When completed, the building will be the tallest structure in South Florida.

Location
Miami, Florida

Client
Florida East Coast Realty, Inc.

Architect
Terra Architects

Structural Engineer
DeSimone Consulting Engineers



OUR LUCAYA BEACH AND GOLF RESORT





Completed at the end of 2000, this $360 million resort facility provides 1600 guestrooms. Three distinct hotel buildings are on the site. Two buildings were renovated and one was demolished and replaced with a new 700-room, serpentine-shaped guestroom building. A signature arrival building of approximately 50,000 sf welcomes guests to the hotel complex. Reef Village is the product of a renovation and alteration of the old Grand Bahama Beach Hotel. The old Lucayan Beach Hotel is now the convention center hotel with the existing casino converted into ballroom space. Numerous site buildings and structures are provided including four site restaurants and bars, a 40 ft tall sugar mill with a giant spiral water slide, an elevated aqueduct, a complex-shaped building designed to entertain children, a central services building, landscape maintenance, and security buildings. An Isle of Capri Casino building was part of the retail portion of the project.

Location
Grand Bahama Island, Bahamas

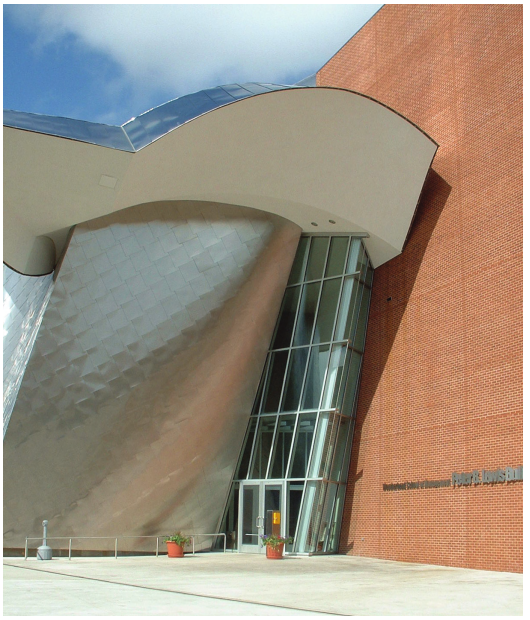
Client
Hutchinson Lucayan, Ltd.

Architect
RTKL Associates, Inc.

Structural Engineer
DeSimone Consulting Engineers



PETER B. LEWIS BUILDING



Located at the Weatherhead School of Management at Case Western Reserve University, the building is a composite structure of concrete and steel and contains 145,000 sf of space and includes classrooms, offices, a library and a café. Cast-in-place concrete flat slab construction was used for the primary floor supporting elements. A 36 in. concrete transfer slab and 8 ft deep curved concrete beams were also utilized. The building features a 65,000 sf steel clad roof structure. The stainless steel-clad surfaces were framed using 4 in. standard pipe steel. The steel portion of the building rests upon the concrete and is therefore not load bearing. However, the structural steel elements do provide additional shear resistance. The design includes light-gage framing, which supports the curved drywall that clads the interior atrium space. This is the first Gehry project designed to have all of its structural steel detailing completed in Computer Aided Three-dimensional Interactive Application (CATIA).

Location
Cleveland, Ohio

Client
Case Western Reserve University

Architect
Gehry Partners

Structural Engineer
DeSimone Consulting Engineers



SANDS CASINO RESORT, BETHLEHEM



This casino is one of 13 new casinos planned for the Keystone State. Located on the former Bethlehem Steel site south of the city, the gaming facility opened in May 2009. Phase I is a 257,000 sf project containing casino and hotel components. Adjacent to the Casino and connected under the Minsi l'iail Bridge is a 10-story hotel encompassing 204,000 sf. Bordering the hotel is a 47,000 sf multipurpose space, 204,000 sf of retail space, and over 1 million square feet of garage parking for 6,000 cars. The new casino will build upon the structural and architectural environment of the Bethlehem Steel Plant. The roof of the Casino uses sawtooth monitors and there are exposed structural theming elements located at the building's facades.

Location
Bethlehem, Pennsylvania

Client
Sands Bethworks Gaming, LLC

Architect
RTKL Associates, Inc.
Walsh Bishop Associates

Structural Engineer
DeSimone Consulting Engineers





SHANGRI-LA, NEW YORK

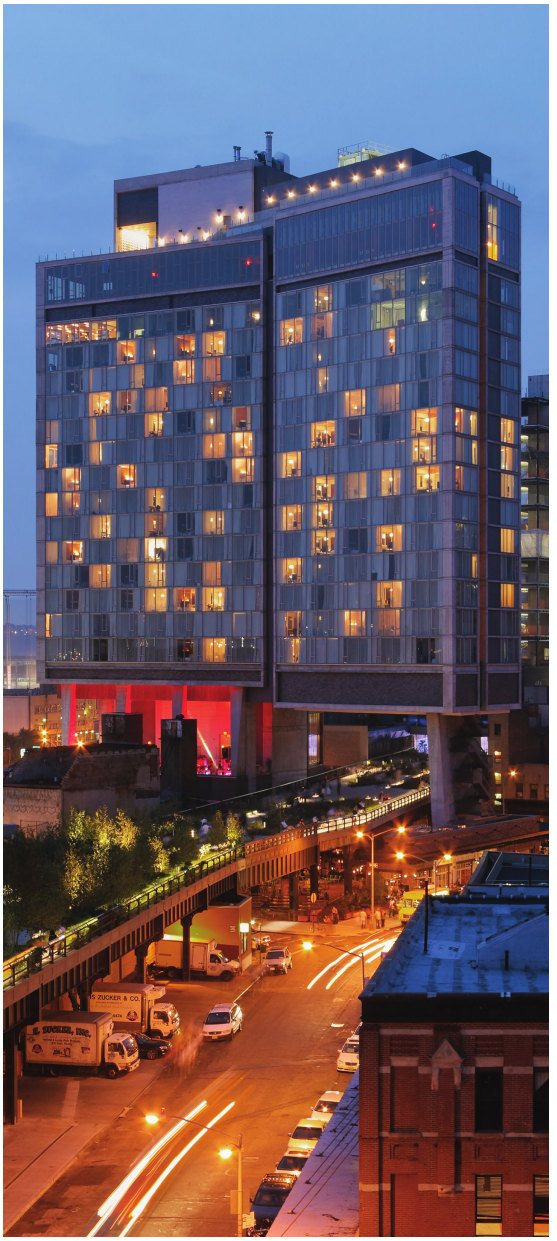
This 300,000 sf, 67-story, luxury condominium hotel contains three below grade levels, a 10-story tower located at the east end of the site, and a 67-story tower covering the west side of the site. The ground floor features a 100 ft atrium and there will be a pool with retractable roof at the ninth level. Limited to only a 4000 sf tower footprint, the building will rise 709 ft, giving the structure a very slender aspect ratio of 16.5:1. To overcome this challenge, the design implements the first use of posttension shear walls in New York City. In addition, a two-story rigid link is located at the top podium level, thereby reducing the effective cantilever length of the tower. Dual symmetric dampers are also utilized to control the building's lateral response.

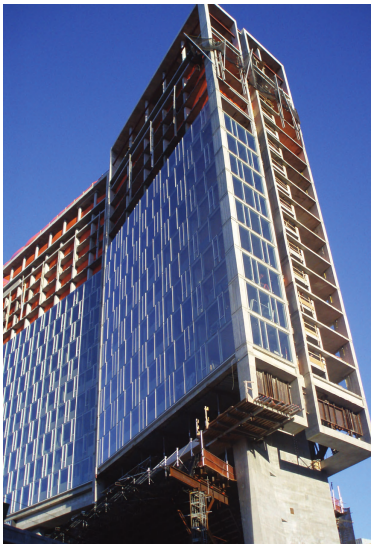
Location
New York, New York

Client
RFR Realty
Hines Development

Architect
Foster & Partners
SLCE Architects

Structural Engineer
DeSimone Consulting Engineers





THE STANDARD

This new 22-story, 204,500 sf luxury boutique hotel is erected over the Highline, a historic elevated railroad/landmark district, which bisects the site. The building's structure consists of cast-in-place concrete. A transfer system spanning the Highline is located at the fifth floor and enables the above floor construction. This transfer system requires the use of grade 65 steel to accommodate the 114 ft length. The truss sits five levels above grade and 30 ft above the Highline. The project's elegant East Pier is a 5 ft thick, 57 ft tall architectural exposed concrete member. The pier is also a technological challenge as it was created in a single 16 h pour. Poor soil conditions require a deep foundation system, and the proximity of ground water close to street level necessitates the implementation of a heavily reinforced bathtub-type foundation.

Location
New York, New York

Client
Hotels AB
The John Buck Company

Architect
Polshek Partnership Architects

Structural Engineer
DeSimone Consulting Engineers



TAUBMAN MUSEUM OF ART (© JEFF GOLDBERG/ESTO.)



This 85,000 sf complex houses the museum's permanent collection of nineteenth and early twentieth century American paintings, African art, Southern decorative arts, and Contemporary art. The exterior design of the building draws from the surrounding landscape with an undulating roofline of stainless steel and glass combined with angular exterior stone walls. The facility houses considerably larger exhibition and education spaces than the original, adjoining museum, including an expanded Art Venture, a children's interactive gallery and art center as well as a flexible programming/theater space, an IMAX theater, a multipurpose auditorium, a research library, a book and gift shop, a studio classroom, a works on paper study room, and a café with indoor and outdoor seating. A spacious atrium provides a meeting place for the community, as well as a premier venue for special events.

Location
Roanoke, Virginia

Client
Taubman Museum of Art

Architect
Randall Stout Architects

Structural Engineer
DeSimone Consulting Engineers





VDARA TOWER (© TIMOTHY HURSLEY).

Located in Las Vegas on a 66 acre downtown site, this new, 57-story mixed-use project contains 1.5 million square feet. The project is the second phase of work to occur at the MGM CityCenter site and creates a 1500-unit condominium hotel tower, retail areas, spa facilities, restaurants, back-of-house areas, and parking.

Location
Las Vegas, Nevada

Client
MGM Mirage Design Group

Architect
Rafael Viñoly Architects
Leo A. Daly Company

Structural Engineer
DeSimone Consulting Engineers





VISIONAIRE

Located on Site 3 in Battery Park City, this new project contains approximately 486,000 sf. The building is composed of three primary components: a 32-story residential condominium building, a 4-story maintenance facility, and 4000 sf of ground floor retail space. The 437,000 sf residential tower contains 250 condominium units as well as a health club, support, and public spaces. The 42,500 sf maintenance facility provides space for the Battery Park City Parks Conservancy. The project also incorporates a full-basement parking garage, which utilizes a pressure slab and ramp access. This project was fast-tracked and designed in accordance with LEED Platinum Rating Guideline.

Location
New York, New York

Client
The Albanese Organization

Architect
Pelli Clarke Pelli Associates
SLCE Architects

Structural Engineer
DeSimone Consulting Engineers





WESTIN DIPLOMAT RESORT AND SPA (© MARSHALL GEROMETTA/EMPORIS.COM)



This 41-story, 2.8 million square feet resort contains a hotel, convention center, and condominium complex, with a parking garage, energy center, health spa and fitness center, and retail spaces. The residential and hospitality functions are contained within the tower, which sits atop a four-story podium structure. Located in a Federal Emergency Management Agency (FEMA) flood zone, the complex is built on an auger-cast pile foundation. The building uses conventionally reinforced slab construction; however, the tower sits atop two discrete structures. A 15 ft deep composite truss connects the two legs at the 10th floor, supporting the single tower above. The building occupies the entire site footprint at ground level and gradually steps back from the perimeter as it ascends. Sloped columns were utilized to pick up the long cantilevers created by the step backs. A curved space frame skylight forms the lobby roof structure.

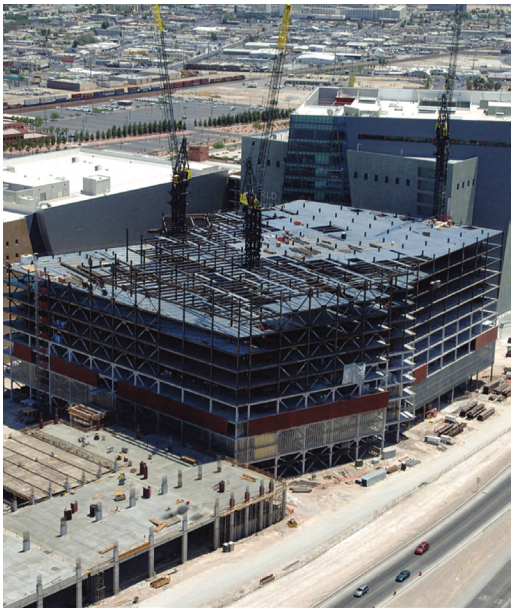
Location
Miami, Florida

Client
Diplomat Properties, Ltd.

Architect
Nichols Brosch Wurst Wolfe & Associates

Structural Engineer
DeSimone Consulting Engineers





WORLD MARKET CENTER BUILDING C

World Market Center is a dynamic showcase for the home and hospitality contract furnishings industry. Located in the heart of Las Vegas, this new, 15-story commercial building will add 2.1 million square feet to the existing World Market Center. Building C is the largest and most diversified showroom structure on the 12 million sf home furnishings campus. The building totals 100,000 sf and each level offers an open, easy to walk floor plan with optimum racetrack circulation as well as sky bridges that connect to other buildings.

Location
Las Vegas, Nevada

Client
The Related Companies

Architect
Jerde Partnership International
JMA Architecture Studios

Structural Engineer
DeSimone Consulting Engineers