A.M. Neville

Properties of Concrete

5th edition

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Properties of concrete

Fifth Edition

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Preface to the Fifth Edition

The format, organization and style of this edition are the same as those of the previous editions. The justification is that those features have shown themselves to be successful by the fact that sales have continued to be strong right up to the year 2011. The total sales in English, as well as in the 12 languages into which this book has been translated, exceed half-a-million copies over a period of nearly half-a-century.

With the passage of time, standards evolve, become modified, withdrawn and replaced. This produces a need for updating a technical book such as *Properties of Concrete* and can be accommodated by minor changes in new impressions of an existing edition, as was done in the 14 impressions of the fourth edition, which I intended to be final. This is still the case with American standards, where ASTM has a strict policy of periodic reviews, confirmation or replacement.

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On the other hand, the situation of the British standards is far more complex. Specifically, there exist now some new British standards, described as also European standards, denoted by BS EN. There continue to be in force some traditional British standards, denoted by BS. In some cases, the British standards are described as obsolete, obsolescent, and also as 'current, superseded'. All this is highly confusing but is perhaps an inevitable consequence of a piecemeal introduction of new standards, which do not simply replace the old ones on a one-to-one basis. There is a well-known saying that a camel is a horse designed by a committee. The European standards are designed by an international committee!

I have retained by way of tables and limits information contained in a number of the old British standards, even those that have been withdrawn, because they contribute to knowledge of what is desirable in the understanding of a relevant property. I believe that such an approach is valuable in a scientific book, encyclopaedic in

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character. This is especially so because a number of the new BS EN standards lay down how to measure some property of concrete and then to 'declare' the outcome but say nothing about the interpretation of the result. Such an approach does not contribute to knowledge of what is desirable, let alone to understanding of the relevant property.

The new standards have been introduced in this fifth edition of *Properties of Concrete* for the purpose of informing the reader about the approach to, or principles of, testing. However, given continuing evolution of standards, for specific use, the reader should refer to the text of the actual standards and follow them scrupulously. After all, this book is not intended to be a manual or handbook, let alone a cookbook.

Furthermore, I have not deleted references to earlier publications. I did so for two reasons. Firstly, this is a new edition of a successful book and not a new book. Secondly, the old references contain the development of our knowledge, much

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of it basic. On the other hand, many of the recent papers contain minutiae of specific behaviour under specific conditions, and contribute but little to the pool of knowledge capable of generalization.

It may be a reflection of my age but I find that our pool of knowledge of value to designers, contractors and suppliers is not greatly increased by a paper co-authored by six people without much coordination or generalization. Nor are papers describing the behaviour of concrete incorporating fly ash from a single source of value to the concrete community at large, the main benefit being commercial or personal.

This edition contains some additional topics: delayed ettringite formation, recycled concrete aggregate, self-compacting concrete, thaumasite sulfate attack, and of course augmentation and modification of various topics.

I have not included that topical subject, sustainability (which seems to be the flavour of the decade). As I see it, if sustainability of concrete as a material (distinct from a structure made of

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concrete) is to ensure durability, then of course, this is of great importance; for this reason chapters 10 and 11 are devoted to the durability of concrete.

However, durability does not mean a service life as long as possible. What we should aim at is a desired service life, and this is governed by the function of the structure: a garden shed is at the lower end of the scale, and a large bridge or dam are at the upper end. Dwellings are a good example of social needs changing with time, e.g. lifts (elevators) or bathrooms. Likewise, offices may be of the open plan type or they may consist of numerous separate rooms Where there is a social usage change, the 'old' style may be a disadvantage in that a modification of the structure may be more expensive than demolition followed by a new design. And paying in advance for a more expensive structure in the first place is uneconomical and may also discourage construction. But these issues are outside the scope of this

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book. So, if I do not enthuse over sustainability, it is not because of my ignorance.

In writing the fifth edition, and especially in including references to new standards, I have been greatly assisted by Robert Thomas, Manager, Library and Information Services of the Institution of Structural Engineers and by Rose Marney, Library Manager and Debra Francis, Librarian, of the Institution of Civil Engineers. Their exceptionally efficient and friendly help is gratefully acknowledged.

I am grateful to Simon Lake for progressing the contractual aspects of the fifth edition and to Patrick Bond, Robert Sykes and Helen Leech for looking after the production aspects of the book.

And, as always, I have to thank profoundly my lifelong technical collaborator and severe critic (i.e. wife), Dr. Mary Neville.

May I wish the reader (or in the 19th century parlance, ‘the gentle reader’) good concrete and durable concrete structures.

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Preface

Concrete and steel are the two most commonly used structural materials. They sometimes complement one another, and sometimes compete with one another so that structures of a similar type and function can be built in either of these materials. And yet, the engineer often knows less about the concrete of which the structure is made than about the steel.

Steel is manufactured under carefully controlled conditions; its properties are determined in a laboratory and described in a manufacturer's certificate. Thus, the designer need only specify the steel as complying with a relevant standard, and the site engineer's supervision is limited to the workmanship of the connections between the individual steel members.

On a concrete building site, the situation is totally different. It is true that the quality of cement is guaranteed by the manufacturer in a manner

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similar to that of steel and, provided suitable cementitious materials are chosen, it is hardly ever a cause of faults in a concrete structure. But it is concrete, and not cement, that is the building material. The structural members are more often than not made in situ, and their quality is almost exclusively dependent on the workmanship of concrete making and placing.

The disparity in the methods of steel and concrete making is, therefore, clear, and the importance of the control of the quality of concrete work on the site is apparent. Furthermore, as the trade of a concretor has not yet the training and the tradition of some of the other building trades, an engineer's supervision on the site is essential. These facts must be borne in mind by the designer, as careful and intricate design can be easily vitiated if the properties of the actual concrete differ from those assumed in the design calculations. Structural design is only as good as the materials used.

From the above it must not be concluded that making good concrete is difficult. 'Bad' concrete

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– often a substance of unsuitable consistency, hardening into a honeycombed, non-homogeneous mass – is made simply by mixing cement, aggregate and water. Surprisingly, the ingredients of a good concrete are exactly the same, and it is only the 'know-how', backed up by understanding, that is responsible for the difference.

What, then, is good concrete? There are two overall criteria: the concrete has to be satisfactory in its hardened state, and also in its fresh state while being transported from the mixer and placed in the formwork. The requirements in the fresh state are that the consistency of the mix be such that it can be compacted by the means desired without excessive effort, and also that the mix be cohesive enough for the methods of transporting and placing used so as not to produce segregation with a consequent lack of homogeneity of the finished product. The primary requirements of a good concrete in its hardened state are a satisfactory compressive strength and an adequate durability.

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All this has been valid since the first edition of this book appeared in 1963. In its three editions and the 12 languages in which translations have been published, the book seems to have served well those involved in concrete, which continues to be the most important and widespread construction material. However, very significant changes in knowledge and in practice have taken place in recent years, and this is why a fourth edition needed to be written. The extent of these changes has been such that a 'bolt-on' approach was not appropriate and, except for its fundamental core, this is, therefore, a new book. Its coverage has been greatly widened, and it gives a broad as well as a detailed view of concrete as a construction material. But there has been no change for change's sake. The form, style, approach, and organization of the material in the previous editions have been maintained so that those readers who are familiar with the earlier versions will have no difficulty in finding their way in the new book.

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The fourth edition contains much new material on cementitious materials, some of which were not used, or were little used, in the past. Knowledge of these materials should now form part of the engineer's stock-in-trade. Durability of concrete under various conditions of exposure, including carbonation and alkali–silica reaction, is treated fully. In particular, the behaviour of concrete under the extreme conditions existing in coastal areas of the hot parts of the world, where a great deal of construction nowadays takes place, is discussed. Other new topics are: high performance concrete, recently introduced admixtures, concrete under cryogenic conditions, and properties of the aggregate–matrix interface, to mention but the main ones.

It has to be admitted that the treatment of the various cementitious materials presented quite a challenge which has provoked the following digression. A very large number of papers on these materials and some other topics were published in the 1980s and continue in the 1990s. Many

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worthwhile papers have elucidated the behaviour of the various materials and their influence on the properties of concrete. But many more reported narrowly construed investigations which described the influence of a single parameter, with some other conditions kept unrealistically constant. Sometimes it is forgotten that, in a concrete mix, it is usually not possible to change one ingredient without altering some other property of the mix.

Generalized inferences from such piecemeal research are at best difficult and at worst dangerous. We do not need more of these little research projects, each one chalking up a 'publication' in the author's curriculum vitae. Nor do we need an endless succession of formulae, each derived from a small set of data. Some, seemingly impressive, analyses show an excellent correlation with the experimental data fed into the pool from which the expressions were derived in the first place: such correlation is not surprising. But then it should not be surprising either if the ex-

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pressions fail dismally when used to predict the behaviour in untried circumstances where there exist factors ignored in the original analysis.

A further comment can be made about the influences of various factors on the behaviour of concrete which have been determined by statistical analyses. While the use of statistics in the evaluation of test results and in establishing relationships is valuable, and often essential, a statistical relation alone, without a physical explanation, is not a sound basis for claiming that a true relation exists between two or more factors. Likewise, extrapolation of a valid relationship must not be assumed to be automatically valid. This is obvious but sometimes forgotten by an enthusiastic author who is under the impression that he or she has discovered a general ‘rule’.

Whereas we must consider available research, there is little value in collecting together a mass of research findings or giving a general review of each topic of research. Rather, this book has striven to integrate the various topics so as to

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show their interdependence in the making and using of concrete. An understanding of the physical and chemical phenomena involved is an essential basis for tackling the unfamiliar, in contrast to the *ad hoc* approach for picking up clues from past experience, which will work only so far, and sometimes may result in a catastrophe. Concrete is a patient material but, even so, avoidable flaws in the selection and proportioning of the mix ingredients should be avoided.

It has to be remembered that the various concrete mixes now used are derivatives and developments of the traditional concrete, so that knowledge of the basic properties of concrete continues to be essential. In consequence, a large part of the book is devoted to these fundamentals. The original work of the pioneers of the knowledge of concrete which explains the underlying behaviour of concrete on a scientific basis and the classical references have been retained: they allow us to have a proper perspective of our knowledge.

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The ultimate purpose of this book is to facilitate better construction in concrete. To achieve this, it is necessary to understand, to master, and to control the behaviour of concrete not only in the laboratory but also in actual structures. It is in this respect that an author with a structural background is at an advantage. Furthermore, experience in construction and in investigations of lack of durability and serviceability has been exploited.

Because the book is used in so many countries, it was thought best to use both the SI and the Imperial units of measurement, now paradoxically known as American. All the data, diagrams and tables are, therefore, conveniently presented for readers, progressive or traditionalist, in all countries.

This book was written in its entirety during the period of one year and it should therefore present a closely-knit explanation of the behaviour of concrete, rather than a series of somewhat disconnected chapters. This cohesion may

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be of benefit to readers who have often been obliged to consult collections of uncoordinated articles in a 'book' with a nominal editor or editors.

In a single volume, it is not possible to cover the whole field of concrete: specialized materials, such as fibre reinforced concrete, polymer concrete, or sulfur concrete, albeit useful, are not dealt with. Inevitably, the author selects what he considers most important or most interesting, or simply what he knows most about, even though the scope of his knowledge increases with age and experience. The emphasis in this book is on an integrated view of the properties of concrete and on underlying scientific reasons, for, as Henri Poincaré said, an accumulation of facts is no more a science than a heap of stones is a house.

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Acknowledgements

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don W1A 2BS, from where copies of the complete standards may be purchased. The late Professor J. F. Kirkaldy kindly provided the data of Table 3.7.

The full details of the sources can be found at the end of each chapter; the reference numbers appear with the captions to the illustrations and the headings to the tables.

I am grateful to my various clients in litigation and arbitration, and equally to their opposing parties, who enabled me to achieve a better understanding of the behaviour of concrete in service, often by way of observing its 'misbehaviour'.

Very considerable help in finding references was provided by the staff of the Library of the Institution of Civil Engineers, and especially by Mr Robert Thomas who was indefatigable in tracking down the various sources. Finally, I wish to put on record the enormous effort and achievement of Mary Hallam Neville in *cementing* the sources and references into a *cohesive* manuscript

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culminating in a *concrete* book. Without her prompting (a much better word than nagging) this book may not have anteceded the Author's decease.

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Chapter 1. Portland cement

Cement, in the general sense of the word, can be described as a material with adhesive and cohesive properties which make it capable of bonding mineral fragments into a compact whole. This definition embraces a large variety of cementing materials.

For constructional purposes, the meaning of the term ‘cement’ is restricted to the bonding materials used with stones, sand, bricks, building blocks, etc. The principal constituents of this type of cement are compounds of lime, so that in building and civil engineering we are concerned with calcareous cement. The cements of interest in the making of concrete have the property of setting and hardening under water by virtue of a chemical reaction with it and are, therefore, called hydraulic cements.

Hydraulic cements consist mainly of silicates and aluminates of lime, and can be classified

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broadly as natural cements, Portland cements, and high-alumina cements. The present chapter deals with the manufacture of Portland cement and its structure and properties, both when unhydrated and in a hardened state. The different types of Portland and other cements are considered in Chapter 2.

Historical note

The use of cementing materials is very old. The ancient Egyptians used calcined impure gypsum. The Greeks and the Romans used calcined limestone and later learned to add to lime and water, sand and crushed stone or brick and broken tiles. This was the first concrete in history. Lime mortar does not harden under water, and for construction under water the Romans ground together lime and a volcanic ash or finely ground burnt clay tiles. The active silica and alumina in the ash and the tiles combined with the lime to produce what became known as pozzolanic cement from the name of the village of Pozzuoli, near Vesuvi-

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us, where the volcanic ash was first found. The name ‘pozzolanic cement’ is used to this day to describe cements obtained simply by the grinding of natural materials at normal temperature. Some of the Roman structures in which masonry was bonded by mortar, such as the Coliseum in Rome and the Pont du Gard near Nîmes, and concrete structures such as the Pantheon in Rome, have survived to this day, with the cementitious material still hard and firm. In the ruins at Pompeii, the mortar is often less weathered than the rather soft stone.

The Middle Ages brought a general decline in the quality and use of cement, and it was only in the eighteenth century that an advance in the knowledge of cements occurred. John Smeaton, commissioned in 1756 to rebuild the Eddystone Lighthouse, off the Cornish coast, found that the best mortar was produced when pozzolana was mixed with limestone containing a considerable proportion of clayey matter. By recognizing the role of the clay, hitherto considered undesirable,

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Smeaton was the first to understand the chemical properties of hydraulic lime, that is a material obtained by burning a mixture of lime and clay.

There followed a development of other hydraulic cements, such as the 'Roman cement' obtained by James Parker by calcining nodules of argillaceous limestone, culminating in the patent for 'Portland cement' taken out by Joseph Aspdin, a Leeds bricklayer, stonemason, and builder, in 1824. This cement was prepared by heating a mixture of finely-divided clay and hard limestone in a furnace until CO_2 had been driven off; this temperature was much lower than that necessary for clinkering. The prototype of modern cement was made in 1845 by Isaac Johnson, who burnt a mixture of clay and chalk until clinkering, so that the reactions necessary for the formation of strongly cementitious compounds took place.

The name 'Portland cement', given originally due to the resemblance of the colour and quality of the hardened cement to Portland stone – a limestone quarried in Dorset – has remained

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throughout the world to this day to describe a cement obtained by intimately mixing together calcareous and argillaceous, or other silica-, alumina-, and iron oxide-bearing materials, burning them at a clinkering temperature, and grinding the resulting clinker. The definition of Portland cement in various standards is on these lines, recognizing that gypsum is added after burning; nowadays, other materials may also be added or blended (see p. 64).

Manufacture of Portland cement

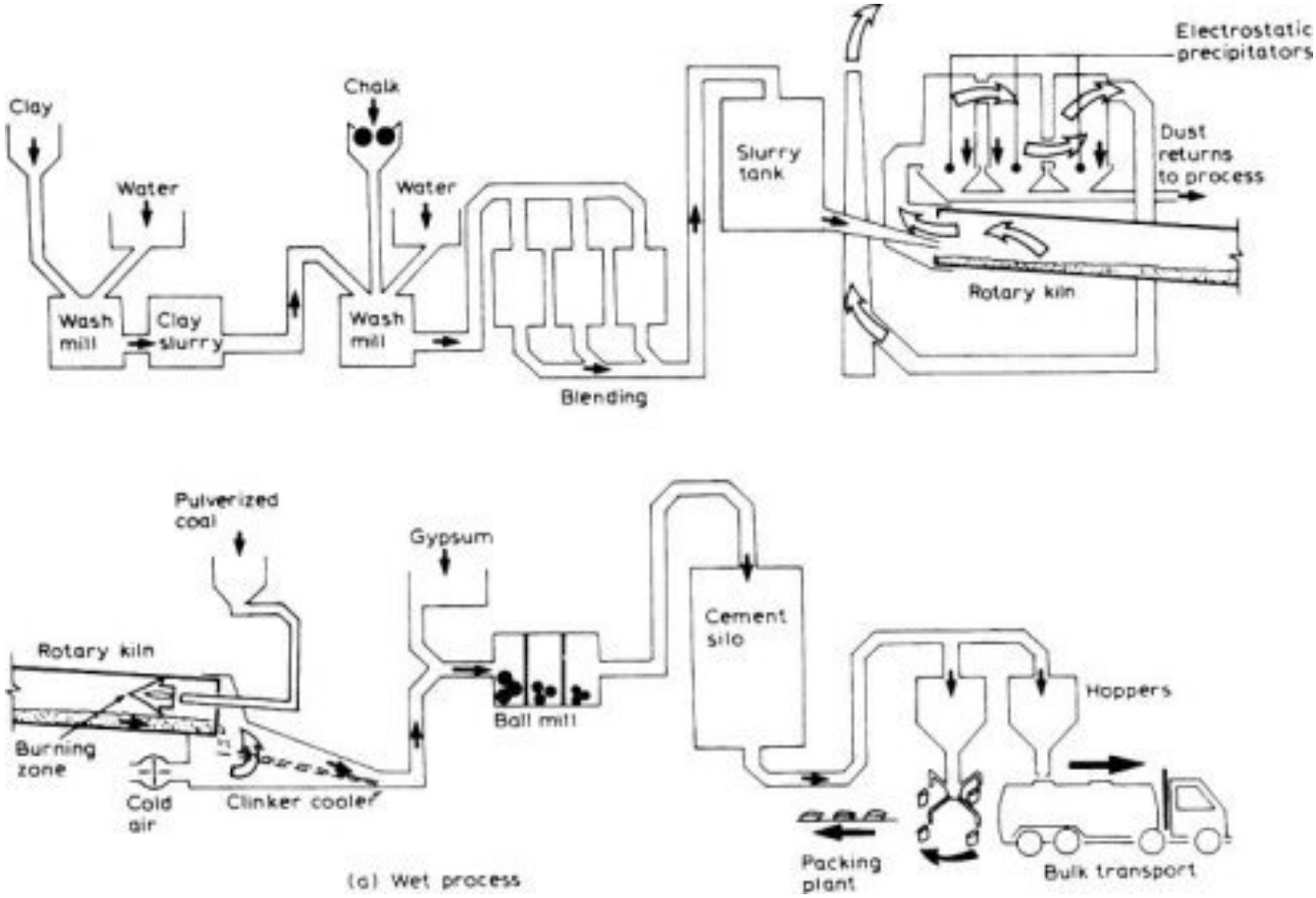
From the definition of Portland cement given above, it can be seen that it is made primarly from a calcareous material, such as limestone or chalk, and from alumina and silica found as clay or shale. Marl, a mixture of calcareous and argillaceous materials, is also used. Raw materials for the manufacture of Portland cement are found in nearly all countries and cement plants operate all over the world.

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The process of manufacture of cement consists essentially of grinding the raw materials, mixing them intimately in certain proportions and burning in a large rotary kiln at a temperature of up to about 1450 °C when the material sinters and partially fuses into balls known as clinker. The clinker is cooled and ground to a fine powder, with some gypsum added, and the resulting product is the commercial Portland cement so widely used throughout the world.

Some details of the manufacture of cement will now be given, and these can be best followed with reference to the diagrammatic representation of the process shown in Fig. 1.1.

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(a) Wet process

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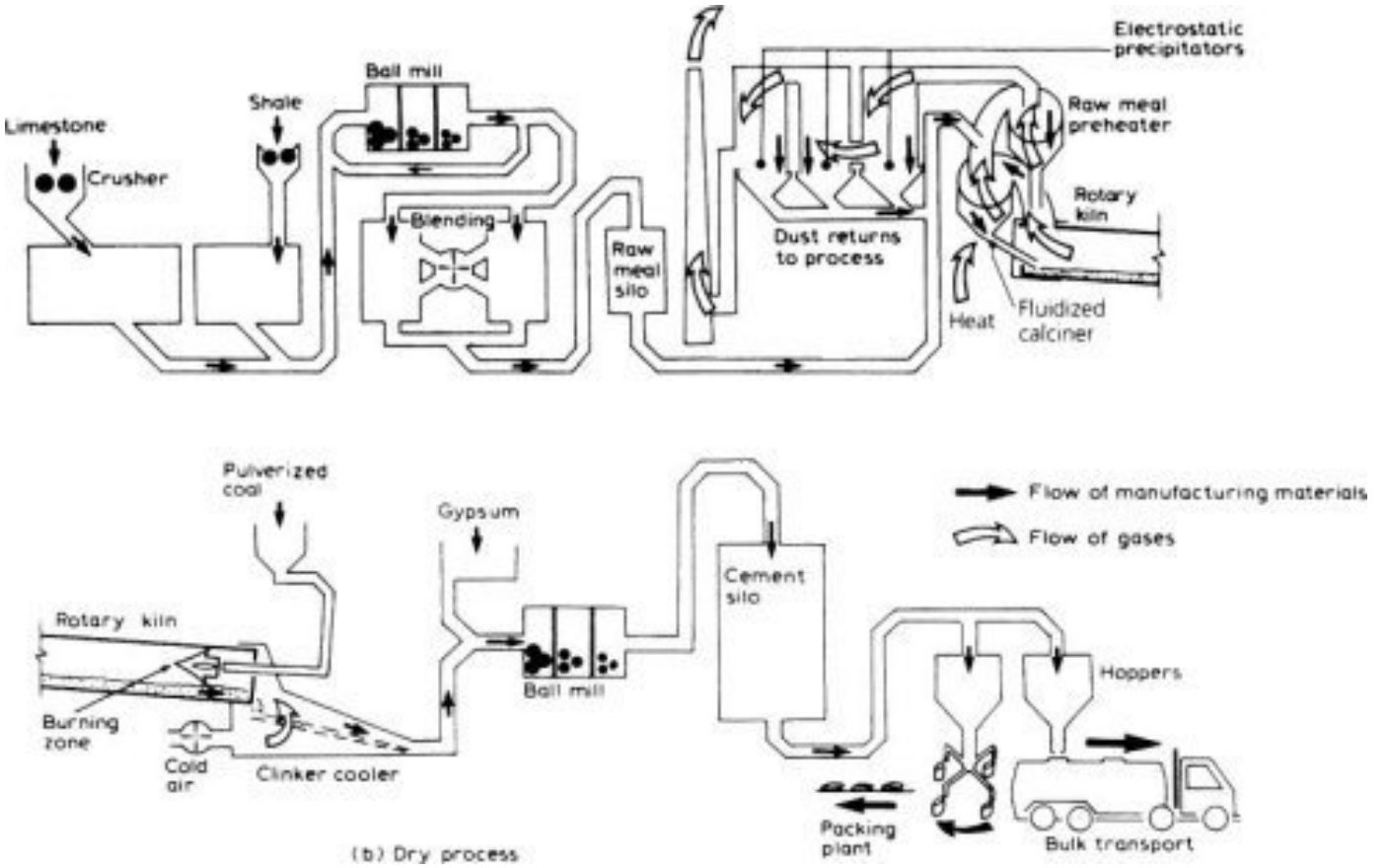


Fig. 1.1. Diagrammatic representation of: (*a*) the wet process and (*b*) the dry process of manufacture of cement

The mixing and grinding of the raw materials can be done either in water or in a dry condition; hence the names 'wet' and 'dry' processes. The actual methods of manufacture depend also on the hardness of the raw materials used and on their moisture content.

Let us consider first the wet process. When chalk is used, it is finely broken up and dispersed

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in water in a washmill; this is a circular pit with revolving radial arms carrying rakes which break up the lumps of solid matter. The clay is also broken up and mixed with water, usually in a similar washmill. The two mixtures are now pumped so as to mix in predetermined proportions and pass through a series of screens. The resulting cement slurry flows into storage tanks.

When limestone is used, it has to be blasted, then crushed, usually in two progressively smaller crushers, and then fed into a ball mill with the clay dispersed in water. There, the comminution of the limestone (to the fineness of flour) is completed, and the resultant slurry is pumped into storage tanks. From here onwards, the process is the same regardless of the original nature of the raw materials.

The slurry is a liquid of creamy consistency, with a water content of between 35 and 50 per cent, and only a small fraction of material – about 2 per cent – larger than a 90 μm (No. 170 ASTM) sieve size. There are usually a number of storage

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tanks in which the slurry is kept, the sedimentation of the suspended solids being prevented by mechanical stirrers or bubbling by compressed air. The lime content of the slurry is governed by the proportioning of the original calcareous and argillaceous materials, as mentioned earlier. Final adjustment in order to achieve the required chemical composition can be made by blending slurries from different storage tanks, sometimes using an elaborate system of blending tanks. Occasionally, for example in the world's northernmost plant in Norway, the raw material is a rock of such composition that it alone is crushed and no blending is required.

Finally, the slurry with the desired lime content passes into the rotary kiln. This is a large, refractory-lined steel cylinder, up to 8 m (or 26 ft) in diameter, sometimes as long as 230 m (or 760 ft), slowly rotating about its axis, which is slightly inclined to the horizontal. The slurry is fed in at the upper end while pulverized coal is blown in by an air blast at the lower end of the

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kiln, where the temperature reaches about 1450 °C. The coal, which must not have too high an ash content, deserves a special mention because typically 220 kg (500 lb) of coal is used to make one tonne of cement. This is worth bearing in mind when considering the price of cement. Oil (of the order of 125 litres (33 US gallons) per tonne of cement) or natural gas were also used, but since the 1980s most oil-fired plants have been converted to coal, which is by far the most common fuel used in most countries. It is worth noting that, because it is burnt in the kiln, coal with a high sulfur content can be used without harmful emissions.

The slurry, in its movement down the kiln, encounters a progressively higher temperature. At first, the water is driven off and CO_2 is liberated; further on, the dry material undergoes a series of chemical reactions until finally, in the hottest part of the kiln, some 20 to 30 per cent of the material becomes liquid, and lime, silica and alumina recombine. The mass then fuses into balls, 3 to 25

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mm ($\frac{1}{8}$ to 1 in.) in diameter, known as clinker. The clinker drops into coolers, which are of various types and often provide means for an exchange of heat with the air subsequently used for the combustion of the pulverized coal. The kiln has to operate continuously in order to ensure a steady regime, and therefore uniformity of clinker, and also to reduce the deterioration of the refractory lining. It should be noted that the flame temperature reaches 1650 °C. The largest existing kiln in a wet-process plant produces 3600 tonnes of clinker a day. Because the manufacture of cement by the wet process is energy intensive, new wet-process plants are no longer built.

In the dry and semi-dry processes, the raw materials are crushed and fed in the correct proportions into a grinding mill, where they are dried and reduced in size to a fine powder. The dry powder, called raw meal, is then pumped to a blending silo, and final adjustment is now made in the proportions of the materials required for the manufacture of cement. To obtain a uniform

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and intimate mixture, the raw meal is blended, usually by means of compressed air inducing an upward movement of the powder and decreasing its apparent density. The air is pumped over one quadrant of the silo at a time, and this permits the apparently heavier material from the non-aerated quadrants to move laterally into the aerated quadrant. Thus the aerated material tends to behave almost like a liquid and, by aerating all quadrants in turn for a total period of about one hour, a uniform mixture is obtained. In some cement plants, continuous blending is used.

In the semi-dry process, the blended meal is now sieved and fed into a rotating dish called a granulator, water weighing about 12 per cent of the meal being added at the same time. In this manner, hard pellets about 15 mm ($\frac{1}{2}$ in.) in diameter are formed. This is necessary, as cold powder fed direct into a kiln would not permit the air flow and exchange of heat necessary for the chemical reactions of formation of cement clinker.

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The pellets are baked hard in a pre-heating grate by means of hot gases from the kiln. The pellets then enter the kiln, and subsequent operations are the same as in the wet process of manufacture. Since, however, the moisture content of the pellets is only 12 per cent as compared with the 40 per cent moisture content of the slurry used in the wet process, the semi-dry-process kiln is considerably smaller. The amount of heat required is also very much lower because only some 12 per cent of moisture has to be driven off, but additional heat has already been used in removing the original moisture content of the raw materials (usually 6 to 10 per cent). The process is thus quite economical, but only when the raw materials are comparatively dry. In such a case the total coal consumption can be as little as 100 kg (220 lb) per tonne of cement.

In the dry process (see Fig. 1b), the raw meal, which has a moisture content of about 0.2 per cent, is passed through a pre-heater, usually of a suspension type; that means that the raw meal

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particles are suspended in the rising gases. Here, the raw meal is heated to about 800 °C before being fed into the kiln. Because the raw meal contains no moisture to be driven off and because it is already pre-heated, the kiln can be shorter than in the wet process. The pre-heating uses the hot gas leaving the kiln. Because that gas contains a significant proportion of the rather volatile alkalis (see p. 9) and chlorides, a part of the gas may need to be bled off to ensure that the alkali content of the cement is not too high.

The major part of the raw meal can be passed through a fluidized calciner (using a separate heat source) introduced between the pre-heater and the kiln. The temperature in the fluidized calciner is about 820 °C. This temperature is stable so that the calcination is uniform and the efficiency of the heat exchange is high.

A part of the raw meal is fed direct into the kiln in the usual manner but, overall, the effect of the fluidized calciner is to increase the decarbonation (dissociation of $CaCO_3$) of the raw meal

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prior to entry into the kiln and thus greatly to increase the kiln throughput. What is probably the largest dry-process plant in the world produces 10 000 tonnes of clinker a day using a kiln 6.2 m (20 ft) in diameter and 105 m (345 ft) long. In the U.S. more than 80% of cement production uses the dry process.

It should be stressed that all processes require an intimate mixture of the raw materials because a part of the reactions in the kiln must take place by diffusion in solid materials, and a uniform distribution of materials is essential to ensure a uniform product.

On exit from the kiln, regardless of the type of process, the clinker is cooled, the heat being used to preheat the combustion air. The cool clinker, which is characteristically black, glistening, and hard, is interground with gypsum in order to prevent flash setting of the cement. The grinding is done in a ball mill consisting of several compartments with progressively smaller steel balls, sometimes preceded by passing through a roll

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press. In most plants, a closed-circuit grinding system is used: the cement discharged by the mill is passed through a separator, fine particles being removed to the storage silo by an air current, while the coarser particles are passed through the mill once again. Closed-circuit grinding avoids the production of a large amount of excessively fine material or of a small amount of too coarse material, faults often encountered with open-circuit grinding. Small quantities of grinding aids such as ethylene glycol or propylene glycol are used. Information about grinding aids is given by Massazza and Testolin.[1.90] The performance of a ball mill can be improved by pre-grinding the clinker in a horizontal impact crusher.

Once the cement has been satisfactorily ground, when it will have as many as 1.1×10^{12} particles per kg (5×10^{11} per lb), it is ready for transport in bulk. Less commonly, the cement is packed in bags or drums. However, some types of cement, such as white, hydrophobic, expansive, regulated-set, oil-well, and high-alumina, are

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always packed in bags or drums. A standard bag in the United Kingdom contains 50 kg (110 lb) of cement; a US sack weighs 94 lb (42.6 kg); other bag sizes are also used. Bags of 25 kg are becoming popular.

Except when the raw materials necessitate the use of the wet process, the dry process is used nowadays in order to minimize the energy required for burning. Typically, the burning process represents 40 to 60 per cent of the production cost, while the extraction of raw materials for the manufacture of cement represents only 10 per cent of the total cost of cement.

Around 1990, the average energy consumption in the United States for the production of 1 tonne of cement by the dry process was 1.6 MWh. In modern plants, this figure is much reduced, being below 0.8 MWh in Austria.[1.96] Electricity consumption, which accounts for some 6 to 8 per cent of total energy used, is typically of the order: 10 kWh for crushing the raw materials, 28 kWh in the raw meal preparation,

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24 kWh in burning, and 41 kWh in grinding.[1.18] The capital cost of installation of a cement plant is very high: nearly US$200 per tonne of cement produced per annum.

In addition to the main processes, there are also other processes of manufacture of cement, of which one, using gypsum instead of lime, perhaps deserves mention. Gypsum, clay and coke with sand and iron oxide are burnt in a rotary kiln, the end products being Portland cement and sulfur dioxide which is further converted into sulfuric acid.

In areas where only a small cement production is required or where investment capital is limited, a vertical kiln of the Gottlieb type can be used. This fires nodules of raw meal and fine coal powder combined, and produces agglomerated clinker which is then broken up. A single kiln, 10 m (33 ft) high, produces up to 300 tonnes of cement a day. China used several thousand of such kilns, but now has a very large modern cement

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industry, producing 1000 million tonnes per annum.

Chemical composition of Portland cement

We have seen that the raw materials used in the manufacture of Portland cement consist mainly of lime, silica, alumina and iron oxide. These compounds interact with one another in the kiln to form a series of more complex products and, apart from a small residue of uncombined lime which has not had sufficient time to react, a state of chemical equilibrium is reached. However, equilibrium is not maintained during cooling, and the rate of cooling will affect the degree of crystallization and the amount of amorphous material present in the cooled clinker. The properties of this amorphous material, known as glass, differ considerably from those of crystalline compounds of a nominally similar chemical composition. Another complication arises from the inter-

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action of the liquid part of the clinker with the crystalline compounds already present.

Nevertheless, cement can be considered as being in frozen equilibrium, i.e. the cooled products are assumed to reproduce the equilibrium existing at the clinkering temperature. This assumption is, in fact, made in the calculation of the compound composition of commercial cements: the 'potential' composition is calculated from the measured quantities of oxides present in the clinker as if full crystallization of equilibrium products had taken place.

Four compounds are usually regarded as the major constituents of cement: they are listed in Table 1.1, together with their abbreviated symbols. This shortened notation, used by cement chemists, describes each oxide by one letter, viz.: $CaO = C$; $SiO_2 = S$; $Al_2O_3 = A$; and $Fe_2O_3 = F$. Likewise, H_2O in hydrated cement is denoted by H, and SO_3 by $\bar{S}$.

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Table 1.1. Main Compounds of Portland Cement

Name of compound	*Oxide composition*	*Abbreviation*
Tricalcium silicate	$3CaO.SiO_2$	C_3S
Dicalcium silicate	$2CaO.SiO_2$	C_2S
Tricalcium aluminate	$3CaO.Al_2O_3$	C_3A
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	C_4AF

In reality, the silicates in cement are not pure compounds, but contain minor oxides in solid solution. These oxides have significant effects on the atomic arrangements, crystal form and hydraulic properties of the silicates.

The calculation of the potential composition of Portland cement is based on the work of R. H. Bogue and others, and is often referred to as 'Bogue composition'. Bogue's[1,2] equations for the percentages of main compounds in cement are given below. The terms in brackets represent the percentage of the given oxide in the total mass of cement.

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$$C_3S = 4.07(CaO) - 7.60(SiO_2) - 6.72(Al_2O_3) - 1.43(Fe_2O_3) - 2.85(SO_3)$$

$$C_2S = 2.87(SiO_2) - 0.75(3CaO.SiO_2)$$

$$C_3A = 2.65(Al_2O_3) - 1.69(Fe_2O_3)$$

$$C_4AF = 3.04(Fe_2O_3).$$

There are also other methods of calculating the composition,[1.1] but the subject is not considered to be within the scope of this book. We should note, however, that the Bogue composition underestimates the C_3S content (and overestimates C_2S) because other oxides replace some of the CaO in C_3S; as already stated, chemically pure C_3S and C_2S do not occur in Portland cement clinker.

A modification of the Bogue composition which takes into account the presence of substituent ions in the nominally pure main compounds has been developed by Taylor[1.84] for the rapidly cooled clinkers produced in modern cement plants.

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In addition to the main compounds listed in Table 1.1, there exist *minor compounds*, such as MgO, TiO_2, Mn_2O_3, K_2O and Na_2O; they usually amount to not more than a few per cent of the mass of cement. Two of the minor compounds are of particular interest: the oxides of sodium and potassium, Na_2O and K_2O, known as *the alkalis* (although other alkalis also exist in cement). They have been found to react with some aggregates, the products of the reaction causing disintegration of the concrete, and have also been observed to affect the rate of the gain of strength of cement.[1.3] It should, therefore, be pointed out that the term 'minor compounds' refers primarily to their quantity and not necessarily to their importance. The quantity of the alkalis and of Mn_2O_3 can be rapidly determined using a spectrophotometer.

The compound composition of cement has been established largely on the basis of studies of phase equilibria of the ternary systems C–A–S and C–A–F, and the quaternary system

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$C–C_2S–C_5A_3–C_4AF$, and others. The course of melting or crystallization was traced and the compositions of liquid and solid phases at any temperature were computed. In addition to the methods of chemical analysis, the actual composition of clinker can be determined by a microscope examination of powder preparations and their identification by the measurement of the refractive index. Polished and etched sections can be used both in reflected and transmitted light. Other methods include the use of X-ray powder diffraction to identify the crystalline phases and also to study the crystal structure of some of the phases, and of differential thermal analysis; quantitative analysis is also possible, but complicated calibrations are involved.[1.68] Modern techniques include phase analysis through a scanning electron microscope and image analysis through an optical microscope or a scanning electron microscope.

Estimating the composition of cement is aided by more rapid methods of determining the ele-

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mental composition, such as X-ray fluorescence, X-ray spectrometry, atomic absorption, flame photometry, and electron probe micro-analysis. X-ray diffractometry is useful in the determination of free lime, i.e. CaO as distinct from $Ca(OH)_2$, and this is convenient in controlling the kiln performance.[1.67]

C_3S, which is normally present in the largest amounts, occurs as small, equidimensional colourless grains. On cooling below 1250 °C, it decomposes slowly but, if cooling is not too slow, C_3S remains unchanged and is relatively stable at ordinary temperatures.

C_2S is known to have three, or possibly even four, forms. α-C_2S, which exists at high temperatures, inverts to the β-form at about 1450 °C. β-C_2S undergoes further inversion to γ-C_2S at about 670 °C but, at the rates of cooling of commercial cements, β-C_2S is preserved in the clinker. β-C_2S forms rounded grains, usually showing twinning.

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C_3A forms rectangular crystals, but C_3A in frozen glass forms an amorphous interstitial phase.

C_4AF is really a solid solution ranging from C_2F to C_6A_2F, but the description C_4AF is a convenient simplification.[1.4]

The actual proportions of the various compounds vary considerably from cement to cement, and indeed different types of cement are obtained by suitable proportioning of the raw materials. In the United States, an attempt was at one time made to control the properties of cements required for different purposes by specifying the limits of the four major compounds, as calculated from the oxide analysis. This procedure would cut out numerous physical tests normally performed, but unfortunately the calculated compound composition is not sufficiently accurate, nor does it take into account all the relevant properties of cement, and cannot therefore serve as a substitute for direct testing of the required properties.

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A general idea of the composition of cement can be obtained from Table 1.2, which gives the oxide composition limits of Portland cements. Table 1.3 gives the oxide composition of a typical cement of the 1960s and the calculated compound composition,[1.5] obtained by means of Bogue's equations on p. 9.

Table 1.2. Usual Composition Limits of Portland Cement

Oxide	*Content, per cent*
CaO	60–67
SiO_2	17–25
Al_2O_3	3–8
Fe_2O_3	0.5–6.0
MgO	0.5–4.0
Alkalis (as Na_2O)	0.3–1.2
SO_3	2.0–3.5

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Table 1.3. Oxide and Compound Compositions of a Typical Portland Cement of the 1960s[1.5]

Typical oxide composition per cent		*Hence, calculated compound composition (using formulae of p. 9), per cent*	
CaO	63	C_3A	10.8
SiO_2	20	C_3S	54.1
Al_2O_3	6	C_2S	16.6
Fe_2O_3	3	C_4AF	9.1
MgO	$1\frac{1}{2}$	Minor compounds	—
SO_3	2		
K_2O, Na_2O	1		
Others	1		
Loss on ignition	2		
Insoluble residue	$\frac{1}{2}$		

Two terms used in Table 1.3 require explanation. The *insoluble residue*, determined by treating with hydrochloric acid, is a measure of adulteration of cement, largely arising from impurities in gypsum. British Standard BS 12 : 1996 (withdrawn) limits the insoluble residue to 1.5 per cent of the mass of cement. European Standard BS EN 197-1 : 2000, which allows a 5 per cent content of a filler (see p. 88), limits the insoluble residue to 5 per cent of the mass of the cement exclusive of the filler.

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The *loss on ignition* shows the extent of carbonation and hydration of free lime and free magnesia due to the exposure of cement to the atmosphere. The maximum loss on ignition (at 1000 °C) permitted by BS EN 197-1 : 2000 is 5 per cent and by ASTM C 150-09 is 3 per cent except for Type I cement (2.5 per cent); 4 per cent is acceptable for cements in the tropics. Because hydrated free lime is innocuous (see p. 51), for a given free lime content of cement, a greater loss on ignition is really advantageous. With cements containing a calcareous filler, a higher limit on the loss on ignition is necessary: 5 per cent of the mass of the cement nucleus is allowed by BS EN 197-1 : 2000.

It is interesting to observe the large influence of a change in the oxide composition on the compound composition of cement. Some data of Czernin's[1.5] are given in Table 1.4; column (1) shows the composition of a fairly typical rapid-hardening cement. If the lime content is decreased by 3 per cent, with corresponding in-

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creases in the other oxides (column (2)), a considerable change in the C_3S : C_2S ratio results. Column (3) shows a change of $1\frac{1}{2}$ per cent in the alumina and iron contents compared with the cement of column (1). The lime and silica contents are unaltered and yet the ratio of the two silicates, as well as the contents of C_3A and C_4AF, is greatly affected. It is apparent that the significance of the control of the oxide composition of cement cannot be over-emphasized. Within the usual range of ordinary and rapid-hardening Portland cements the sum of the contents of the two silicates varies only within narrow limits, so that the variation in composition depends largely on the ratio of CaO to SiO_2 in the raw materials.

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Table 1.4. Influence of Change in Oxide Composition on the Compound Composition[1.5]

	Percentage in Cement No.		
	(1)	*(2)*	*(3)*
Oxide			
CaO	66.0	63.0	66.0
SiO_2	20.0	22.0	20.0
Al_2O_3	7.0	7.7	5.5
Fe_2O_3	3.0	3.3	4.5
Others	4.0	4.0	4.0
Compound			
C_3S	65	33	73
C_2S	8	38	2
C_3A	14	15	7
C_4AF	9	10	14

In some countries in the European Union, there is a limit on soluble hexavalent chromium usually of 2 ppm of mass dry cement. Contact

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with excess chromium in fresh concrete may lead to dermatitis.

It may be convenient at this stage to summarize the pattern of formation and hydration of cement; this is shown schematically in Fig. 1.2.

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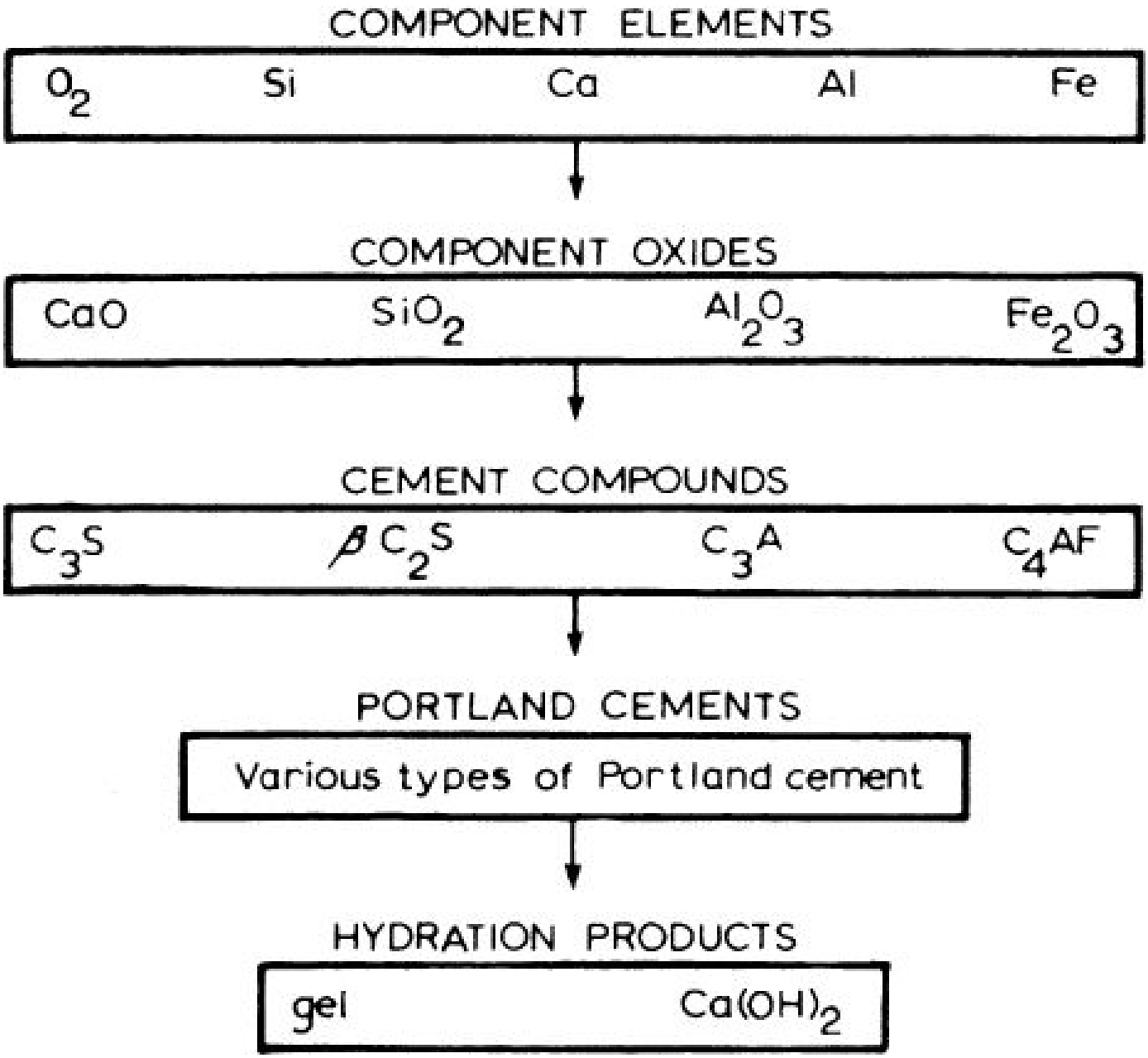


Fig. 1.2. Schematic representation of the formation and hydration of Portland cement

Hydration of cement

The reactions by virtue of which Portland cement becomes a bonding agent take place in a wa-

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ter–cement paste. In other words, in the presence of water, the silicates and aluminates listed in Table 1.1 form products of hydration which in time produce a firm and hard mass – the hydrated cement paste.

There are two ways in which compounds of the type present in cement can react with water. In the first, a direct addition of some molecules of water takes place, this being a true reaction of hydration. The second type of reaction with water is hydrolysis. It is convenient and usual, however, to apply the term hydration to all reactions of cement with water, i.e. to both true hydration and hydrolysis.

Le Chatelier was the first to observe, about 130 years ago, that the products of hydration of cement are chemically the same as the products of hydration of the individual compounds under similar conditions. This was later confirmed by Steinour[1.6] and by Bogue and Lerch,[1.7] with the proviso that the products of reaction may influence one another or may themselves interact with

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the other compounds in the system. The two calcium silicates are the main cementitious compounds in cement, and the physical behaviour of cement during hydration is similar to that of these two compounds alone.[1.8] The hydration of the individual compounds will be described in more detail in the succeeding sections.

The products of hydration of cement have a very low solubility in water as shown by the stability of the hydrated cement paste in contact with water. The hydrated cement bonds firmly to the unreacted cement, but the exact way in which this is achieved is not certain. It is possible that the newly produced hydrate forms an envelope which grows from within by the action of water that has penetrated the surrounding film of hydrate. Alternatively, the dissolved silicates may pass through the envelope and precipitate as an outer layer. A third possibility is for the colloidal solution to be precipitated throughout the mass after the condition of saturation has been reached,

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the further hydration continuing within this structure.

Whatever the mode of precipitation of the products of hydration, the rate of hydration decreases continuously, so that even after a long time there remains an appreciable amount of unhydrated cement. For instance, after 28 days in contact with water, grains of cement have been found to have hydrated to a depth of only 4 μm,[1.9] and 8 μm after a year. Powers[1.10] calculated that complete hydration under normal conditions is possible only for cement particles smaller than 50 μm, but full hydration has been obtained by grinding cement in water continuously for five days.

Microscopic examination of hydrated cement shows no evidence of channelling of water into the grains of cement to hydrate selectively the more reactive compounds (e.g. C_3S) which may lie in the centre of the particle. It would seem then, that hydration proceeds by a gradual reduction in the size of the cement particle. In fact, un-

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hydrated grains of coarse cement were found to contain C_3S as well as C_2S at the age of several months,[1.11] and it is probable that small grains of C_2S hydrate before the hydration of large grains of C_3S has been completed. The various compounds in cement are generally intermixed in all grains, and some investigations have suggested that the residue of a grain after a given period of hydration has the same percentage composition as the whole of the original grain.[1.12] However, the composition of the residue does change throughout the period of cement hydration,[1.49] and especially during the first 24 hours selective hydration may take place.

The main hydrates can be broadly classified as calcium silicate hydrates and tricalcium aluminate hydrate. C_4AF is believed to hydrate into tricalcium aluminate hydrate and an amorphous phase, probably $CaO.Fe_2O_3.aq$. It is possible also that some Fe_2O_3 is present in solid solution in the tricalcium aluminate hydrate.

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The progress of hydration of cement can be determined by different means, such as the measurement of: (a) the amount of $Ca(OH)_2$ in the paste; (b) the heat evolved by hydration; (c) the specific gravity of the paste; (d) the amount of chemically combined water; (e) the amount of unhydrated cement present (using X-ray quantitative analysis); and (f) also indirectly from the strength of the hydrated paste. Thermogravimetric techniques and continuous X-ray diffraction scanning of wet pastes undergoing hydration[1.50] can be used in studying early reactions. The microstructure of hydrated cement paste can also be studied by back-scattered electron imaging in a scanning electron microscope.

Calcium silicate hydrates

The rates of hydration of C_3S and C_2S in a pure state differ considerably, as shown in Fig. 1.3. When the various compounds are present all together in cement, their rates of hydration are affected by compound interactions. In commercial

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cements, the calcium silicates contain small impurities of some of the oxides present in the clinker. The 'impure' C_3S is known as *alite* and the 'impure' C_2S as *belite*. These impurities have a strong effect on the properties of the calcium silicate hydrates (see p. 48).

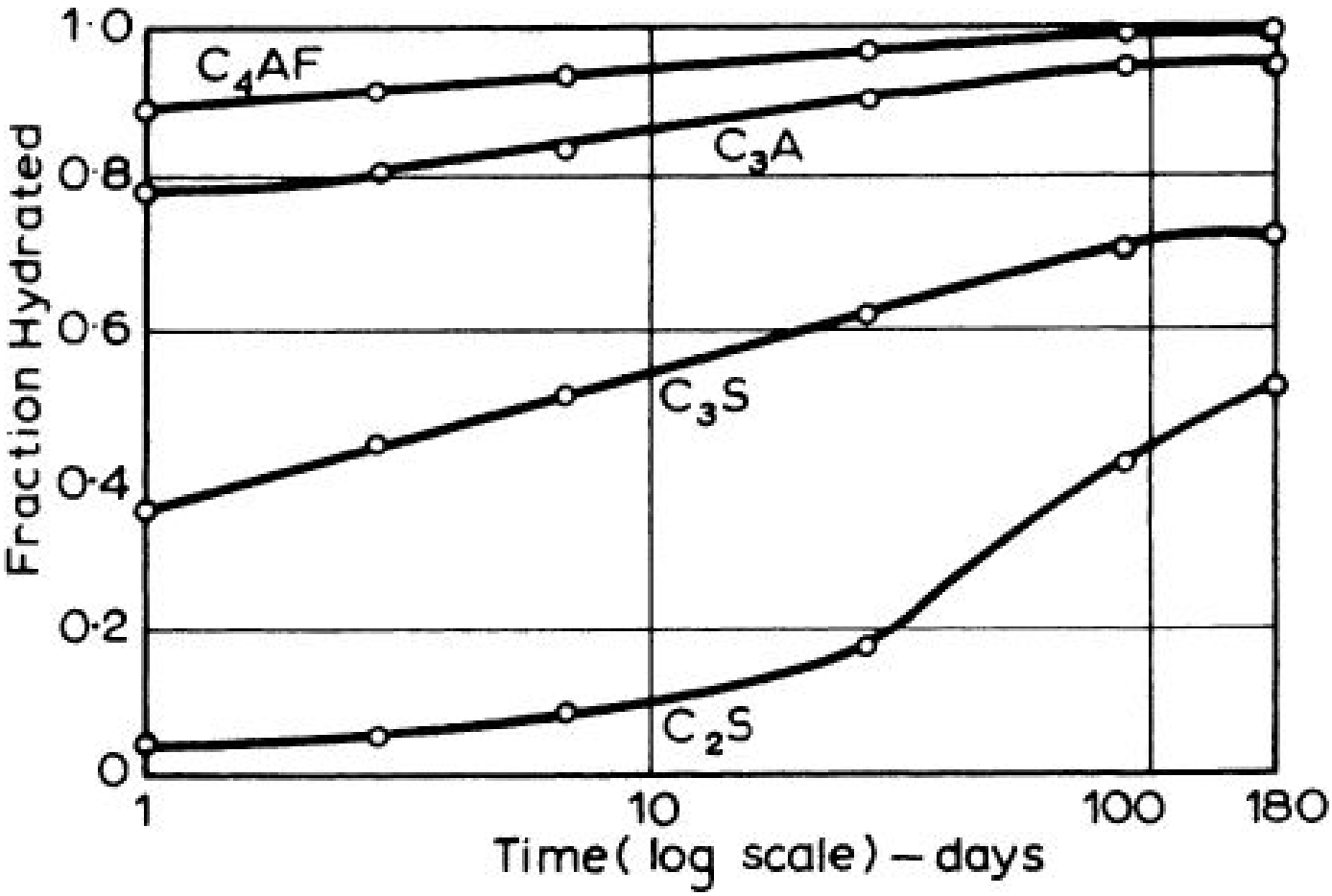


Fig. 1.3. Typical development of hydration of pure compounds[1.47]

When hydration takes place in a limited amount of water, as is the case in cement paste,

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in mortar or in concrete, C_3S is believed to undergo hydrolysis producing a calcium silicate of lower basicity, ultimately $C_3S_2H_3$, with the released lime separating out as $Ca(OH)_2$. There exists, however, some uncertainty as to whether C_3S and C_2S result ultimately in the same hydrate. It would appear to be so from considerations of the heat of hydration[1.6] and of the surface area of the products of hydration,[1.13] but physical observations indicate that there may be more than one – possibly several – distinct calcium silicate hydrates. The C : S ratio would be affected if some of the lime were absorbed or held in solid solution, and there is strong evidence that the ultimate product of hydration of C_2S has a lime/silica ratio of 1.65. This may be due to the fact that the hydration of C_3S is controlled by the rate of diffusion of ions through the overlying hydrate films while the hydration of C_2S is controlled by its slow intrinsic rate of reaction.[1.14] Furthermore, temperature may affect the products of hydration

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of the two silicates because the permeability of the gel is affected by temperature.

The C : S ratio has not been unequivocally determined because different test methods yield different results.[1.74] The variation can be as wide as 1.5 by chemical extraction and 2.0 by thermogravimetric method.[1.66] Electron-optical measurements also yield low values of the C : S ratio.[1.72] The ratio also varies with time and is influenced by the presence of other elements or compounds in the cement. Nowadays, the calcium silicate hydrates are broadly described as C–S–H, and the C : S ratio is believed to be probably near 2.[1.19] Because the crystals formed by hydration are imperfect and extremely small, the mole ratio of water to silica need not be a whole number. C–S–H usually contains small amounts of Al, Fe, Mg, and other ions. At one time, C–S–H was referred to as tobermorite gel because of a structural similarity to a mineral of this name, but this may not be correct,[1.60] and this description is now rarely used.

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Making the approximate *assumption* that $C_3S_2H_3$ is the final product of hydration of both C_3S and C_2S, the reactions of hydration can be written (as a guide, although not as exact stoichiometric equations) as follows.

For C_3S:

$$2C_3S + 6H \rightarrow C_3S_2H_3 + 3Ca(OH)_2.$$

The corresponding masses involved are:

$$100 + 24 \rightarrow 75 + 49.$$

For C_2S:

$$2C_2S + 4H \rightarrow C_3S_2H_3 + Ca(OH)_2.$$

The corresponding masses are:

$$100 + 21 \rightarrow 99 + 22.$$

Thus, on a mass basis, both silicates require approximately the same amount of water for their

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hydration, but C_3S produces more than twice as much $Ca(OH)_2$ as is formed by the hydration of C_2S.

The physical properties of the calcium silicate hydrates are of interest in connection with the setting and hardening properties of cement. These hydrates appear amorphous but electron microscopy shows their crystalline character. It is interesting to note that one of the hydrates believed to exist, denoted by Taylor[1.15] as CSH(I), has a layer structure similar to that of some clay minerals, e.g. montmorillonite and halloysite. The individual layers in the plane of the *a* and *b* axes are well crystallized while the distances between them are less rigidly defined. Such a lattice would be able to accommodate varying amounts of lime without fundamental change – a point relevant to the varying lime/silica ratios mentioned above. In fact, powder diagrams have shown that lime in excess of one molecule per molecule of silica is held in a random manner.[1.15] Steinour[1.16] de-

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scribed this as a merger of solid solution and adsorption.

Calcium silicates do not hydrate in the solid state but the anhydrous silicates probably first pass into solution and then react to form less soluble hydrated silicates which separate out of the supersaturated solution.[1.17] This is the type of mechanism of hydration first envisaged by Le Chatelier in 1881.

Studies by Diamond[1.60] indicate that the calcium silicate hydrates exist in a variety of forms: fibrous particles, flattened particles, a reticular network, irregular grains, all rather difficult to define. However, the predominant form is that of fibrous particles, possibly solid, possibly hollow, sometimes flattened, sometimes branching at the ends. Typically, they are 0.5 μm to 2 μm long and less than 0.2 μm across. This is not a precise picture, but the structure of calcium silicate hydrates is too disordered to be established by the existing techniques, including a combination of the scan-

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ning electron microscope and energy dispersive X-ray spectrometer.

The hydration of C_3S to a large extent characterizes the behaviour of cement and a description of the latter may be appropriate. Hydration does not proceed at a steady rate or even at a steadily changing rate. The initial rapid release of calcium hydroxide into the solution leaves an outer layer of calcium silicate hydrate, perhaps 10 nm thick.[1.61] This layer impedes further hydration so that, for some time thereafter, very little hydration takes place.

As the hydration of cement is an exothermic reaction, the rate of evolution of heat is an indication of the rate of hydration. This shows that there are three peaks in the rate of hydration in the first three days or so, from the time when the dry cement first comes into contact with water. Figure 1.4 shows a plot of the rate of evolution of heat against time.[1.81] We can see the first peak, which is very high, and which corresponds to the initial hydration at the surface of the cement particles,

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largely involving C_3A. The duration of this high rate of hydration is very short, and there follows a so-called *dormant period*, sometimes called also an induction period, during which the rate is very low. This period lasts one or two hours during which the cement paste is workable.

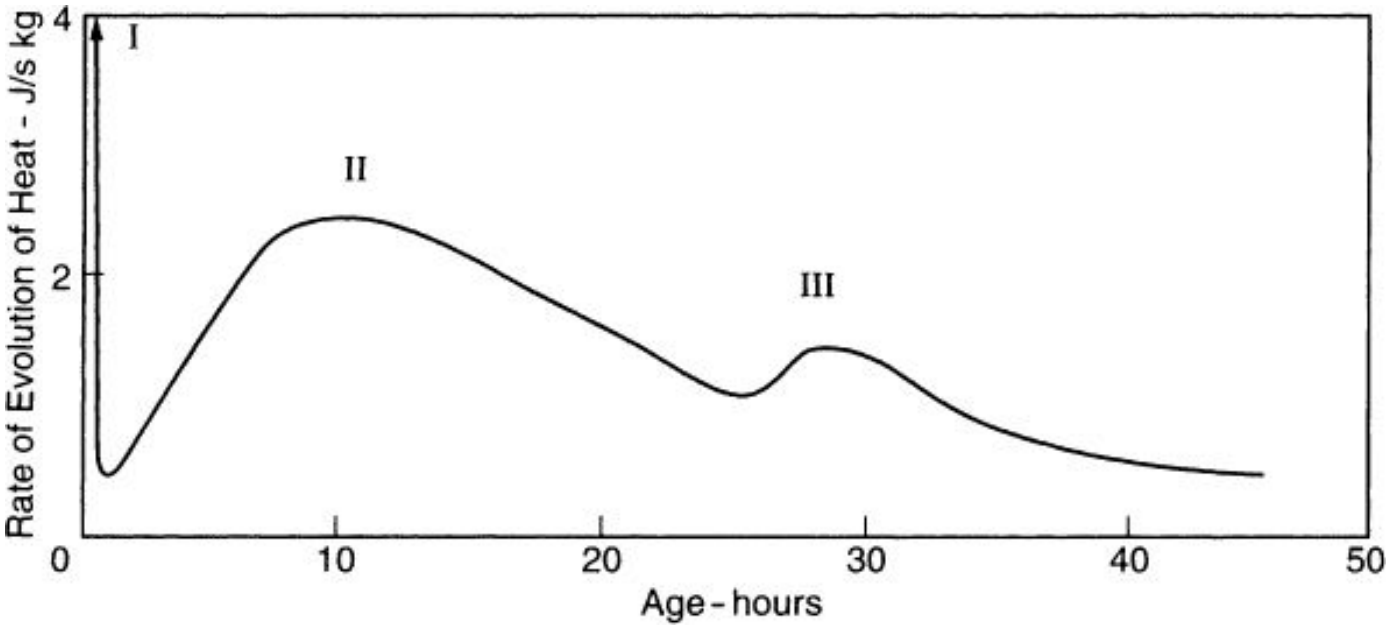


Fig. 1.4. Rate of evolution of heat of Portland cement with a water/cement ratio of 0.4.[181] The first peak of 3200 J/s kg is off the diagram

Eventually, the surface layer is broken down, possibly by an osmotic mechanism or by the growth of the crystals of calcium hydroxide. The

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rate of hydration (and therefore of heat evolution) increases fairly slowly and the products of hydration of individual grains come into contact with one another; setting then occurs. The rate of heat evolution reaches a second peak, typically at the age of about 10 hours, but sometimes as early as 4 hours.

Following this peak, the rate of hydration slows down over a long period, the diffusion through the pores in the products of hydration becoming the controlling factor.[1.62] With most, but not all, cements, there is a renewed increase in the rate of hydration up to a third, lower, peak at the age of between 18 and 30 hours. This peak is related to a renewed reaction of C_3A, following the exhaustion of gypsum.

The advent of the second peak is accelerated by the presence of the alkalis, by a higher fineness of the cement particles, and by an increase in temperature.

Because of the similarity in the progress of hydration of neat calcium silicates and of commer-

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cial Portland cements, they show similar strength development.[1.20] A considerable strength is possessed long before the reactions of hydration are complete and it would thus seem that a small amount of the hydrate binds together the unhydrated remainder; further hydration results in little increase in strength.

$Ca(OH)_2$ liberated by the hydrolysis of the calcium silicates forms thin hexagonal plates, often tens of micrometres across, but later they merge into a massive deposit.[1.60]

Tricalcium aluminate hydrate and the action of gypsum

The amount of C_3A present in most cements is comparatively small but its behaviour and structural relationship with the other phases in cement make it of interest. The tricalcium aluminate hydrate forms a prismatic dark interstitial material, possibly with other substances in solid solution, and is often in the form of flat plates individually surrounded by the calcium silicate hydrates.

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The reaction of pure C_3A with water is very violent and leads to immediate stiffening of the paste, known as *flash set*. To prevent this from happening, gypsum ($CaSO_4.2H_2O$) is added to cement clinker. Gypsum and C_3A react to form insoluble calcium sulfoaluminate ($3CaO.Al_2O_3.3CaSO_4.32H_2O$), but eventually tricalcium aluminate hydrate is formed, although this is preceded by a metastable $3CaO.Al_2O_3.CaSO_4.12H_2O$, produced at the expense of the original high-sulfate calcium sulfoaluminate.[1.6] As more C_3A comes into solution, the composition changes, the sulfate content decreasing continuously. The rate of reaction of the aluminate is high and, if this readjustment in composition is not rapid enough, direct hydration of C_3A is likely. In particular, the first peak in the rate of heat development, normally observed within five minutes of adding water to cement, means that some calcium aluminate hydrate is formed directly during that period, the conditions

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for the retardation by gypsum not yet having been established.

Instead of gypsum, other forms of calcium sulfate can be used in the manufacture of cement: hemihydrate ($CaSO_4.\frac{1}{2}H_2O$) or anhydrite ($CaSO_4$).

There is some evidence that the hydration of C_3A is retarded by $Ca(OH)_2$ liberated by the hydrolysis of C_3S.[1.62] This occurs due to the fact that $Ca(OH)_2$ reacts with C_3A and water to form C_4AH_{19}, which forms a protective coating on the surface of unhydrated grains of C_3A. It is also possible that $Ca(OH)_2$ decreases the concentration of aluminate ions in the solution, thus slowing down the rate of hydration of C_3A.[1.62]

The stable form of the calcium aluminate hydrate ultimately existing in the hydrated cement paste is probably the cubic crystal C_3AH_6, but it is possible that hexagonal C_4AH_{12} crystallizes out first and later changes to the cubic form. Thus the final form of the reaction can be written:

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$$C_3A + 6H \rightarrow C_3AH_6.$$

This again is an approximation and not a stoichiometric equation.

The molecular weights show that 100 parts of C_3A react with 40 parts of water by mass, which is a much higher proportion of water than that required by the silicates.

The presence of C_3A in cement is undesirable: it contributes little or nothing to the strength of cement except at early ages and, when hardened cement paste is attacked by sulfates, expansion due to the formation of calcium sulfoaluminate from C_3A may result in a disruption of the hardened paste. However, C_3A acts as a flux and thus reduces the temperature of burning of clinker and facilitates the combination of lime and silica; for these reasons, C_3A is useful in the manufacture of cement. C_4AF also acts as a flux. It may be noted that if some liquid were not formed during burning, the reactions in the kiln would progress much more slowly and would

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probably be incomplete. On the other hand, a higher C_3A content increases the energy required to grind the clinker.

A positive effect of C_3A is its binding capacity of chlorides (see p. 571).

Gypsum reacts not only with C_3A: with C_4AF it forms calcium sulfoferrite as well as calcium sulfoaluminate, and its presence may accelerate the hydration of the silicates.

The amount of gypsum added to the cement clinker has to be very carefully watched; in particular, an excess of gypsum leads to an expansion and consequent disruption of the set cement paste. The optimum gypsum content is determined by observation of the generation of the heat of hydration. As already mentioned, the first peak in the rate of heat evolution is followed by a second peak some 4 to 10 hours after the water had been added to cement, and with the correct amount of gypsum there should be little C_3A available for reaction after all the gypsum has combined, and no further peak in the heat liberation should oc-

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cur. Thus, an optimum gypsum content leads to a desirable rate of early reaction and prevents local high concentration of products of hydration (see p. 362). In consequence, the size of pores in hydrated cement paste is reduced and strength is increased.[1.78]

The amount of gypsum required increases with the C_3A content and also with the alkali content of the cement. Increasing the fineness of cement has the effect of increasing the quantity of C_3A available at early stages, and this raises the gypsum requirement. A test for the optimum SO_3 content in Portland cement was prescribed by ASTM C 543-84 (discontinued). The optimization is based on a 1-day strength, which usually also produces the lowest shrinkage.

The amount of gypsum added to cement clinker is expressed as the mass of SO_3 present; this is limited by European Standard BS EN 197-1 : 2000 to a maximum of 3.5 per cent, but in some cases higher percentages are permitted. The chemically relevant SO_3 is the soluble sulfate

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contributed by gypsum and not that from high-sulfur fuel, which is bound in the clinker; this is why the current total SO_3 limit is higher than in the past. The maximum values of SO_3 laid down in ASTM C 150-09 depend on the content of C_3A, and are higher in rapid-hardening cement.

Setting

This is the term used to describe the stiffening of the cement paste, although the definition of the stiffness of the paste which is considered set is somewhat arbitrary. Broadly speaking, setting refers to a change from a fluid to a rigid stage. Although, during setting, the paste acquires some strength, for practical purposes it is important to distinguish setting from hardening, which refers to the gain of strength of a set cement paste.

In practice, the terms initial set and final set are used to describe arbitrarily chosen stages of setting. The method of measurement of these setting times is described on p. 49.

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It seems that setting is caused by a selective hydration of cement compounds: the two first to react are C_3A and C_3S. The flash-setting properties of the former were mentioned in the preceding section but the addition of gypsum delays the formation of calcium aluminate hydrate, and it is thus C_3S that sets first. Pure C_3S mixed with water also exhibits an initial set but C_2S stiffens in a more gradual manner.

In a properly retarded cement, the framework of the hydrated cement paste is established by the calcium silicate hydrate, while, if C_3A were allowed to set first, a rather porous calcium aluminate hydrate would form. The remaining cement compounds would then hydrate within this porous framework and the strength characteristics of the cement paste would be adversely affected.

Apart from the rapidity of formation of crystalline products, the development of films around cement grains and a mutual coagulation of components of the paste have also been suggested as factors in the development of set.

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At the time of the final set, there is a sharp drop in the electrical conductivity of the cement paste, and attempts have been made to measure setting by electrical means.

The setting time of cement decreases with a rise in temperature, but above about 30 °C (85 °F) a reverse effect may be observed.[1.1] At low temperatures setting is retarded.

False set

False set is the name given to the abnormal premature stiffening of cement within a few minutes of mixing with water. It differs from *flash set* in that no appreciable heat is evolved, and remixing the cement paste without addition of water restores plasticity of the paste until it sets in the normal manner and without a loss of strength.

Some of the causes of false set are to be found in the dehydration of gypsum when interground with too hot a clinker: hemihydrate ($CaSO_4.\frac{1}{2}H_2O$) or anhydrite ($CaSO_4$) are formed and when the cement is mixed with water these hydrate

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to form needle-shaped crystals of gypsum. Thus what is called 'plaster set' takes place with a resulting stiffening of the paste.

Another cause of false set may be associated with the alkalis in the cement. During storage they may carbonate, and alkali carbonates react with $Ca(OH)_2$, liberated by the hydrolysis of C_3S, to form $CaCO_3$. This precipitates and induces a rigidity of the paste.

It has also been suggested that false set can be due to the activation of C_3S by aeration at moderately high humidities. Water is adsorbed on the grains of cement, and these freshly activated surfaces can combine very rapidly with more water during mixing: this rapid hydration would produce false set.[1.21]

Laboratory tests at cement plants generally ensure that cement is free from false set. If, however, false set is encountered, it can be dealt with by remixing the concrete without adding any water. Although this is not easy, workability will

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be improved and the concrete can be placed in the normal manner.

Fineness of cement

It may be recalled that one of the last steps in the manufacture of cement is the grinding of clinker mixed with gypsum. Because hydration starts at the surface of the cement particles, it is the total surface area of cement that represents the material available for hydration. Thus, the rate of hydration depends on the fineness of the cement particles and, for a rapid development of strength, high fineness is necessary (see Fig. 1.5); the long-term strength is not affected. A higher early rate of hydration means, of course, also a higher rate of early heat evolution.

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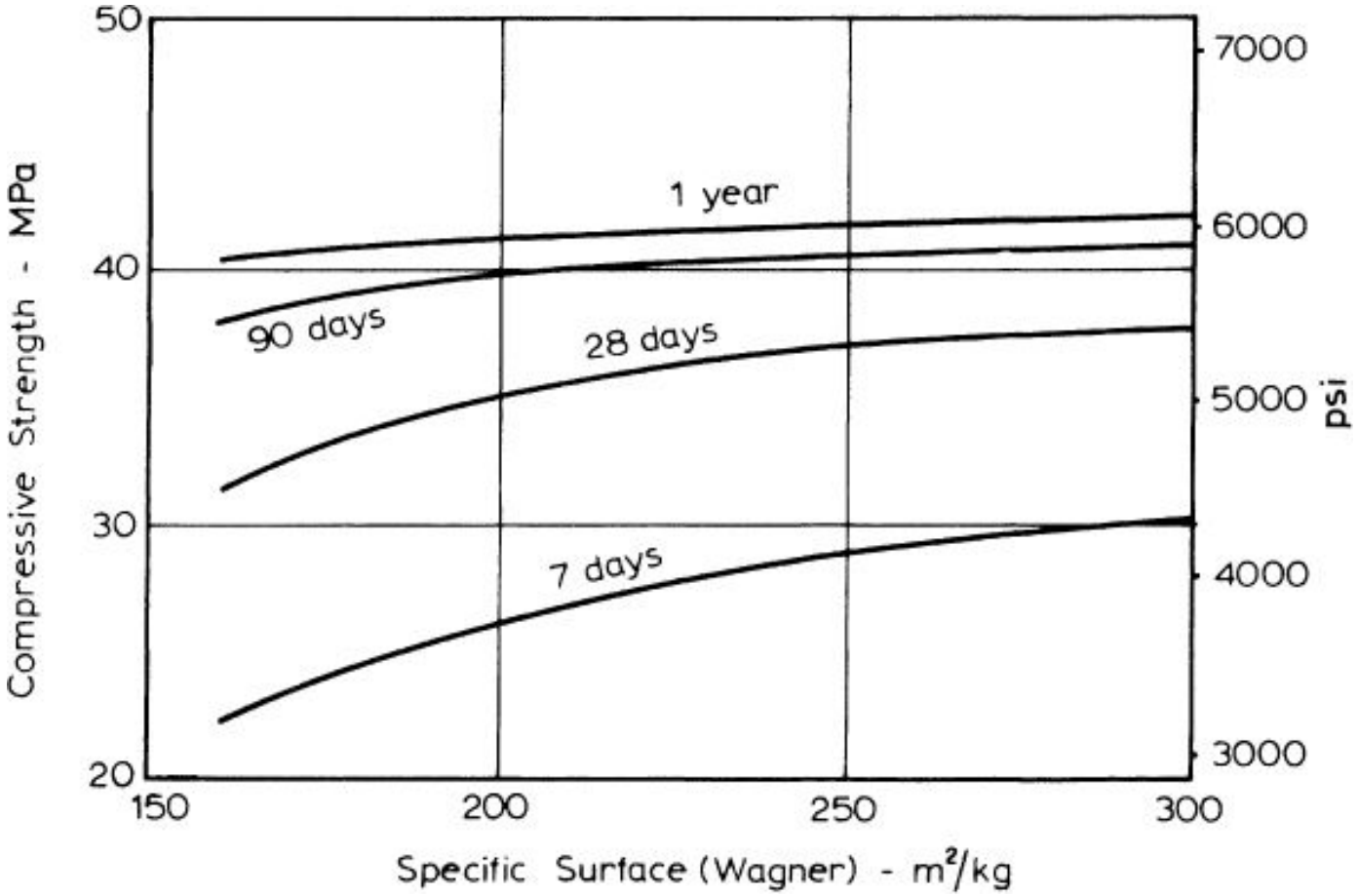


Fig. 1.5. Relation between strength of concrete at different ages and fineness of cement[1.43]

On the other hand, the cost of grinding to a higher fineness is considerable, and also the finer the cement the more rapidly it deteriorates on exposure to the atmosphere. Finer cement leads to a stronger reaction with alkali-reactive aggregate,[1.44] and makes the cement paste, though not necessarily concrete, exhibit a higher shrinkage

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and a greater proneness to cracking. However, fine cement bleeds less than a coarser one.

An increase in fineness increases the amount of gypsum required for proper retardation because, in a finer cement, more C_3A is available for early hydration. The water content of a paste of standard consistency is greater the finer the cement, but conversely an increase in fineness of cement slightly improves the workability of a concrete mix. This anomaly may be due partly to the fact that the tests for consistency of cement paste and workability measure different properties of fresh paste; also, accidental air affects the workability of cement paste, and cements of different fineness may contain different amounts of air.

We can see then that fineness is a vital property of cement and has to be carefully controlled. The fraction of cement retained on a 45 μm (No. 325 ASTM) test sieve can be determined using ASTM C 430-08. (For size of openings of different sieves see Table 3.14.) This would ensure

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that the cement does not contain an excess of large grains which, because of their comparatively small surface area per unit mass, would play only a small role in the process of hydration and development of strength.

However, the sieve test gives no information on the size of grains smaller than 45 μm (No. 325 ASTM) sieve, and it is the finer particles that play the greatest part in the early hydration.

For this reason, modern standards prescribe a test for fineness by determination of the specific surface of cement expressed as the total surface area in square metres per kilogram. A direct approach is to measure the particle size distribution by sedimentation or elutriation: these methods are based on the dependence of the rate of free fall of particles on their diameter. Stokes' law gives the terminal velocity of fall under gravity of a spherical particle in a fluid medium; the cement particles are, in fact, not spherical. This medium must of course be chemically inert with respect to cement. It is also important to achieve a satisfact-

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ory dispersion of cement particles as partial flocculation would produce a decrease in the apparent specific surface.

A development of these methods is the Wagner turbidimeter used in the United States (ASTM C 115-10). In this test, the concentration of particles in suspension at a given level in kerosene is determined using a beam of light, the percentage of light transmitted being measured by a photocell. The turbidimeter gives generally consistent results, but an error is introduced by assuming a uniform size distribution of particles smaller than 7.5 μm. It is precisely these finest particles that contribute most to the specific surface of cement and the error is especially significant with the finer cements used nowadays. However, an improvement on the standard method is possible if the concentration of particles 5 μm in size is determined and a modification of calculations is made.[1.51] A typical curve of particle size distribution is shown in Fig. 1.6, which gives also the corresponding contribution

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of these particles to the total surface area of the sample. As mentioned on p. 7, the particle size distribution depends on the method of grinding and varies, therefore, from plant to plant.

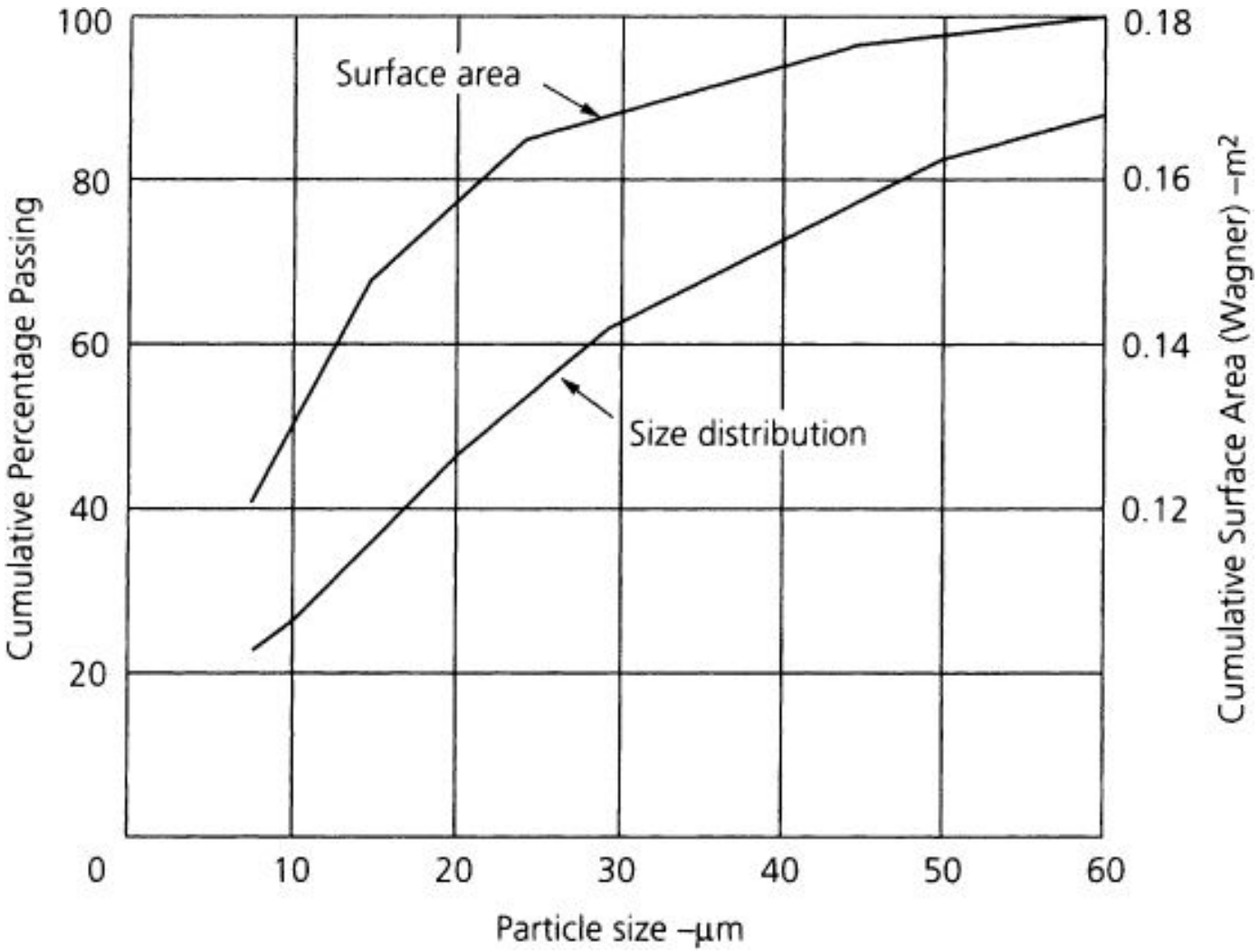


Fig. 1.6. Example of particle size distribution and cumulative surface area contributed by particles up to any given size for 1 g of cement

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It must be admitted, however, that it is not quite clear what is a 'good' grading of cement: should all the particles be of the same size or should their distribution be such that they are able to pack densely? It is now believed that, for a given specific surface of cement, early strength development is better if at least 50 per cent of the particles lie between 3 and 30 μm, with correspondingly fewer very fine and fewer very coarse particles. An even higher proportion of particles in the range of 3 to 30 μm, up to 95 per cent, is believed to lead to an improved early strength and also to a good ultimate strength of concrete made with such a cement. To achieve such a controlled particle size distribution it is necessary to use high-efficiency classifiers in closed-circuit grinding of clinker. These classifiers reduce the amount of energy used in grinding.[1.80]

The reason for the beneficial effect of middle-size particles may be found in the test results of Aïtcin et al.[1.91] who found that grinding of cement results in a certain amount of compound se-

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gregation. Specifically, particles smaller than 4 μm are very rich in SO_3 and rich in the alkalis; particles coarser than 30 μm contain a large proportion of C_2S, while the particles between 4 and 30 μm are rich in C_3S.

It should be noted, however, that there is no simple relation between strength and cement particle size distribution: for example, weathered, partly hydrated clinker, after grinding, results in cement with a misleadingly high apparent surface area.

The specific surface of cement can also be determined by the air permeability method, using an apparatus developed by Lea and Nurse. The method is based on the relation between the flow of a fluid through a granular bed and the surface area of the particles comprising the bed. From this, the surface area per unit mass of the bed material can be related to the permeability of a bed of a given porosity, i.e. containing a fixed volume of pores in the total volume of the bed.

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The permeability apparatus is shown diagrammatically in Fig. 1.7. Knowing the density of cement, the mass required to make a bed of porosity of 0.475 and 10 mm thick can be calculated. This amount of cement is placed in a cylindrical container, a stream of dry air is passed through the cement bed at a constant velocity, and the resulting pressure drop is measured by a manometer connected to the top and bottom of the bed. The rate of airflow is measured by a flowmeter consisting of a capillary placed in the circuit and a manometer across its ends.

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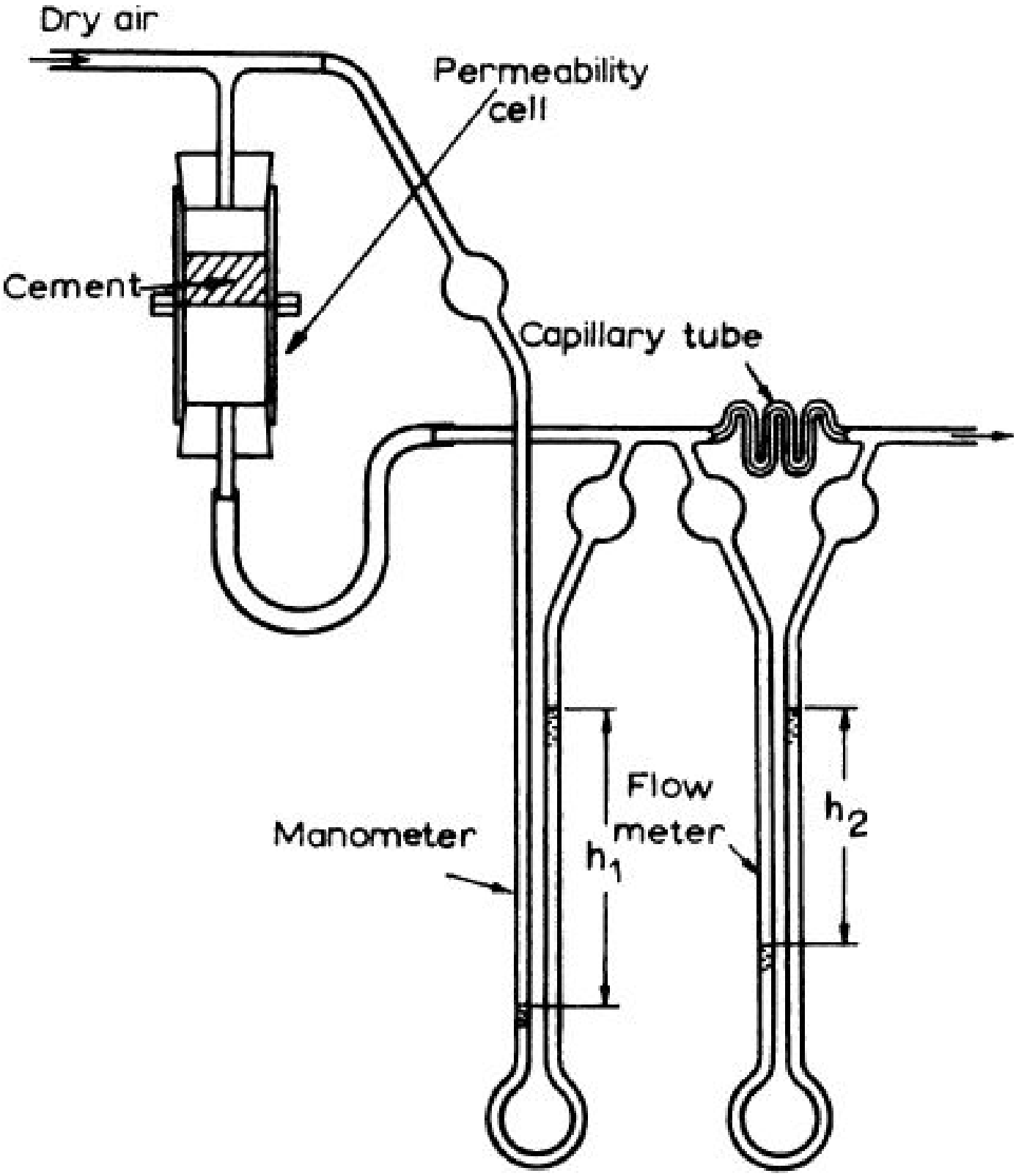


Fig. 1.7. Lea and Nurse permeability apparatus

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An equation developed by Carman gives the specific surface in square centimetres per gram as

$$S_w = \frac{14}{\rho(1-\varepsilon)}\sqrt{\frac{\varepsilon^3 A h_1}{KLh_2}},$$

where ρ = density of cement (g/cm^3)

ε = porosity of cement bed (0.475 in the BS test)

A = cross-sectional area of the bed (5.066 cm^2)

L = height of the bed (1 cm)

h_1 = pressure drop across the bed

h_2 = pressure drop across the flowmeter capillary (between 25 and 55 cm of kerosene)

and K = the flowmeter constant.

For a given aparatus and porosity the expression simplifies to

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$$S_w = \frac{K_1}{\rho}\sqrt{\frac{h_1}{h_2}},$$

where K_1 is a constant.

In the United States and nowadays in Europe, a modification of the Lea and Nurse method, developed by Blaine, is used; the method is prescribed by ASTM C 204-07 and by BS EN 196-6 : 2010. Here, the air does not pass through the bed at a constant rate but a known volume of air passes at a prescribed average pressure, the rate of flow diminishing steadily. The time t for the flow to take place is measured, and for a given apparatus and a standard porosity of 0.500, the specific surface is given by

$$S = K_2\sqrt{t},$$

where K_2 is a constant.

The Lea and Nurse and the Blaine methods give values of specific surface in close agreement with one another but very much higher than the value obtained by the Wagner method. This is

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due to Wagner's assumptions about the size distribution of particles below 7.5 μm, mentioned earlier. The actual distribution in this range is such that the average value of 3.75 μm, assumed by Wagner, underestimates the surface area of these particles. In the air permeability method, the surface area of *all* particles is measured directly, and the resulting value of the specific surface is about 1.8 times higher than the value calculated by the Wagner method. The actual range of the conversion factor varies between 1.6 and 2.2, depending on the fineness of the cement and its gypsum content.

Either method gives a good picture of the *relative* variation in the fineness of cement, and for practical purposes this is sufficient. The Wagner method is somewhat more informative in that it gives an indication of the particle size distribution. An absolute measurement of the specific surface can be obtained by the nitrogen adsorption method, based on the work of Brunauer, Emmett and Teller.[1.45] While in the air-permeability

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methods only the continuous paths through the bed of cement contribute to the measured area, in the nitrogen adsorption method the 'internal' area is also accessible to the nitrogen molecules. For this reason, the measured value of the specific surface is considerably higher than that determined by the air permeability methods. Some typical values are given in Table 1.5.

Table 1.5. Specific Surface of Cement Measured by Different Methods[1.1]

Cement	*Specific surface, m^2/kg measured by:*		
	Wagner method	*Lea and Nurse method*	*Nitrogen adsorption*
A	180	260	790
B	230	415	1000

The surface area of powders much finer than Portland cement, such as silica fume and fly ash, cannot be determined by the air permeability method but requires the use of gas adsorption methods, such as the nitrogen adsorption method. The latter is time consuming, and it may be preferable to use mercury intrusion porosi-

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metry;[1.69] however, this technique has not yet been generally accepted.

Modern specifications no longer lay down minimum values of the specific surface of Portland cement, this being indirectly controlled by the early strength requirement where appropriate. It may be useful to state, however, that a typical ordinary Portland cement would have a specific surface of about 350 or 380 m^2/kg; the specific surface of rapid-hardening Portland cement is typically higher.

Structure of hydrated cement

Many of the mechanical properties of hardened cement and concrete appear to depend not so much on the chemical composition of the hydrated cement as on the physical structure of the products of hydration, viewed at the level of colloidal dimensions. For this reason it is important to have a good picture of the physical properties of the cement gel.

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Fresh cement paste is a plastic network of particles of cement in water but, once the paste has set, its apparent or gross volume remains approximately constant. At any stage of hydration, the hardened paste consists of very poorly crystallized hydrates of the various compounds, referred to collectively as gel, of crystals of $Ca(OH)_2$, some minor components, unhydrated cement, and the residue of the water-filled spaces in the fresh paste. These voids are called capillary pores but, within the gel itself, there exist interstitial voids, called gel pores. The nominal diameter of gel pores is about 3 nm while capillary pores are one or two orders of magnitude larger. There are thus, in hydrated paste, two distinct classes of pores represented diagrammatically in Fig. 1.8.

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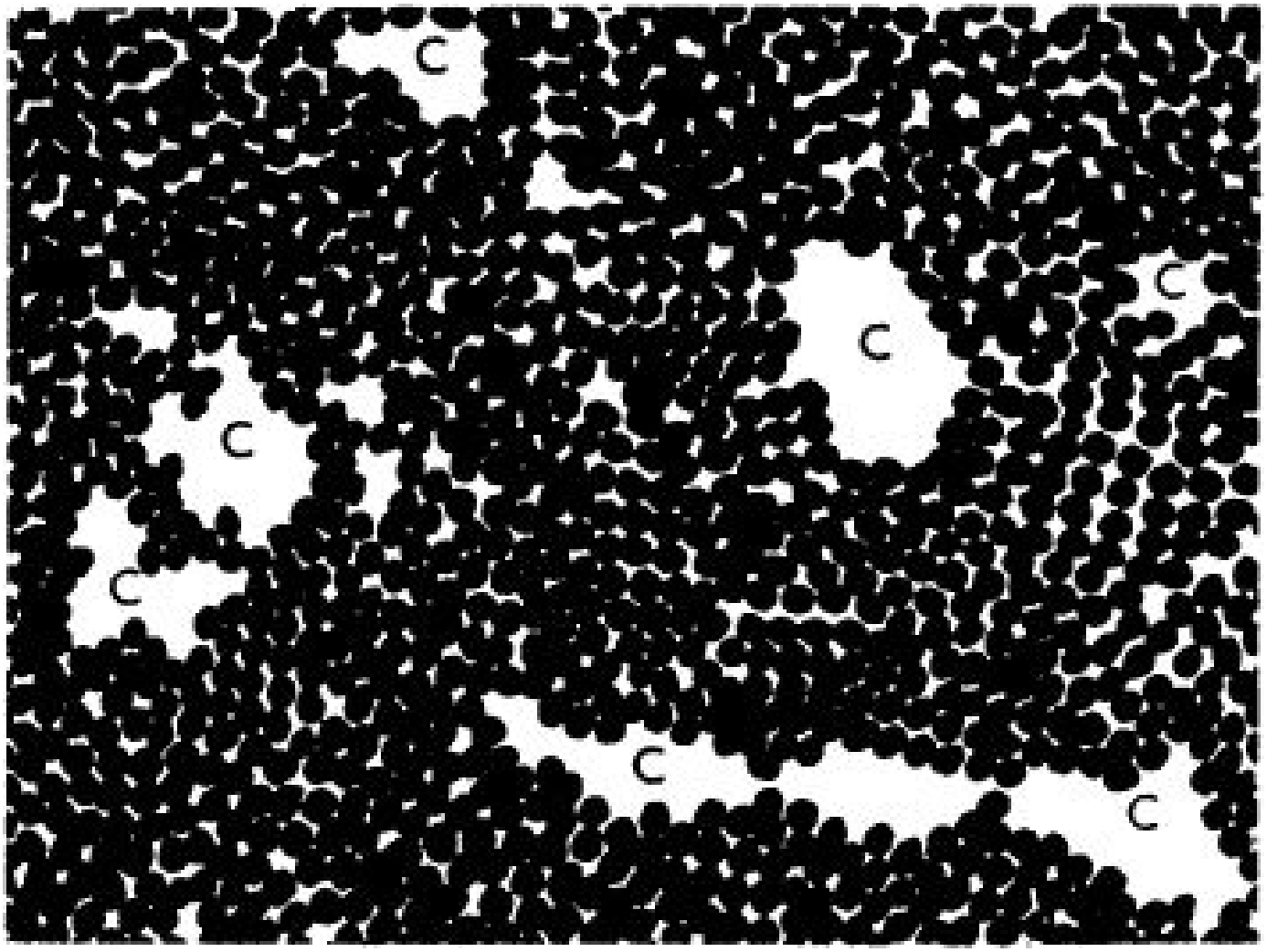


Fig. 1.8. Simplified model of paste structure.[1.22] Solid dots represent gel particles; interstitial spaces are gel pores; spaces such as those marked C are capillary pores. Size of gel pores is exaggerated

Because most of the products of hydration are colloidal (the ratio of calcium silicate hydrates to $Ca(OH)_2$ being 7 : 2 by mass[1.60]) during hydration the surface area of the solid phase increases

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enormously, and a large amount of free water becomes adsorbed on this surface. If no water movement to or from the cement paste is permitted, the reactions of hydration use up the water until too little is left to saturate the solid surfaces, and the relative humidity within the paste decreases. This is known as *self-desiccation*. Because gel can form only in water-filled space, self-desiccation leads to a lower hydration compared with a moist-cured paste. However, in self-desiccated pastes with water/cement ratios in excess of 0.5, the amount of mixing water is sufficient for hydration to proceed at the same rate as when moist-cured.

Volume of products of hydration

The gross space available for the products of hydration consists of the absolute volume of the dry cement together with the volume of water added to the mix. The small loss of water due to bleeding and the contraction of the paste while still plastic will be ignored at this stage. The water

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bound chemically by C_3S and C_2S was shown to be very approximately 24 and 21 per cent of the mass of the two silicates, respectively. The corresponding figures for C_3A and C_4AF are 40 and 37 per cent, respectively. The latter value is calculated on the assumption that the final reaction of hydration of C_4AF is, in approximate terms,

$$C_4AF + 2Ca(OH)_2 + 10H \rightarrow C_3AH_6 + C_3FH_6.$$

As mentioned earlier, these figures are not accurate because our knowledge of stoichiometry of the products of hydration of cement is inadequate to state the amounts of water combined chemically. It is preferable, therefore, to consider non-evaporable water as determined by a given method (see p. 36). This water, determined under specified conditions,[1.48] is taken as 23 per cent of the mass of anhydrous cement (although in Type II cement the value may be as low as 18 per cent).

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The specific gravity of the products of hydration of cement is such that they occupy a greater volume than the absolute volume of unhydrated cement but smaller than the sum of volumes of the dry cement and the non-evaporable water by approximately 0.254 of the volume of the latter. An average value of specific gravity of the products of hydration (including pores in the densest structure possible) in a saturated state is 2.16.

As an example, let us consider the hydration of 100 g of cement. Taking the specific gravity of dry cement as 3.15, the absolute volume of unhydrated cement is 100/3.15 = 31.8 ml. The non-evaporable water is, as we have said, about 23 per cent of the mass of cement, i.e. 23 ml. The solid products of hydration occupy a volume equal to the sum of volumes of anhydrous cement and water less 0.254 of the volume of non-evaporable water, i.e.

$$31.8 + 0.23 \times 100(1 - 0.254) = 48.9 \text{ ml.}$$

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Because the paste in this condition has a characteristic porosity of about 28 per cent, the volume of gel water, w_g, is given by

$$\frac{w_g}{48.9 + w_g} = 0.28,$$

whence $w_g = 19.0$ ml, and the volume of hydrated cement is $48.9 + 19.0 = 67.9$ ml.

Summarizing, we have:

Mass of dry cement	=	100.0 g
Absolute volume of dry cement	=	31.8 ml
Weight of combined water	=	23.0 g
Volume of gel water	=	19.0 ml
Total water in the mix	=	42.0 ml
Water/cement ratio by mass	=	0.42
Water/cement ratio by volume	=	1.32
Volume of hydrated cement	=	67.9 ml
Original volume of cement and water	=	73.8 ml
Decrease in volume due to hydration	=	5.9 ml
Volume of products of hydration of 1 ml of dry cement	=	2.1 ml

It should be noted that the hydration was assumed to take place in a sealed test tube with no water movement to or from the system. The

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volumetric changes are shown in Fig. 1.9. The ‘decrease in volume’ of 5.9 ml represents the empty capillary space distributed throughout the hydrated cement paste.

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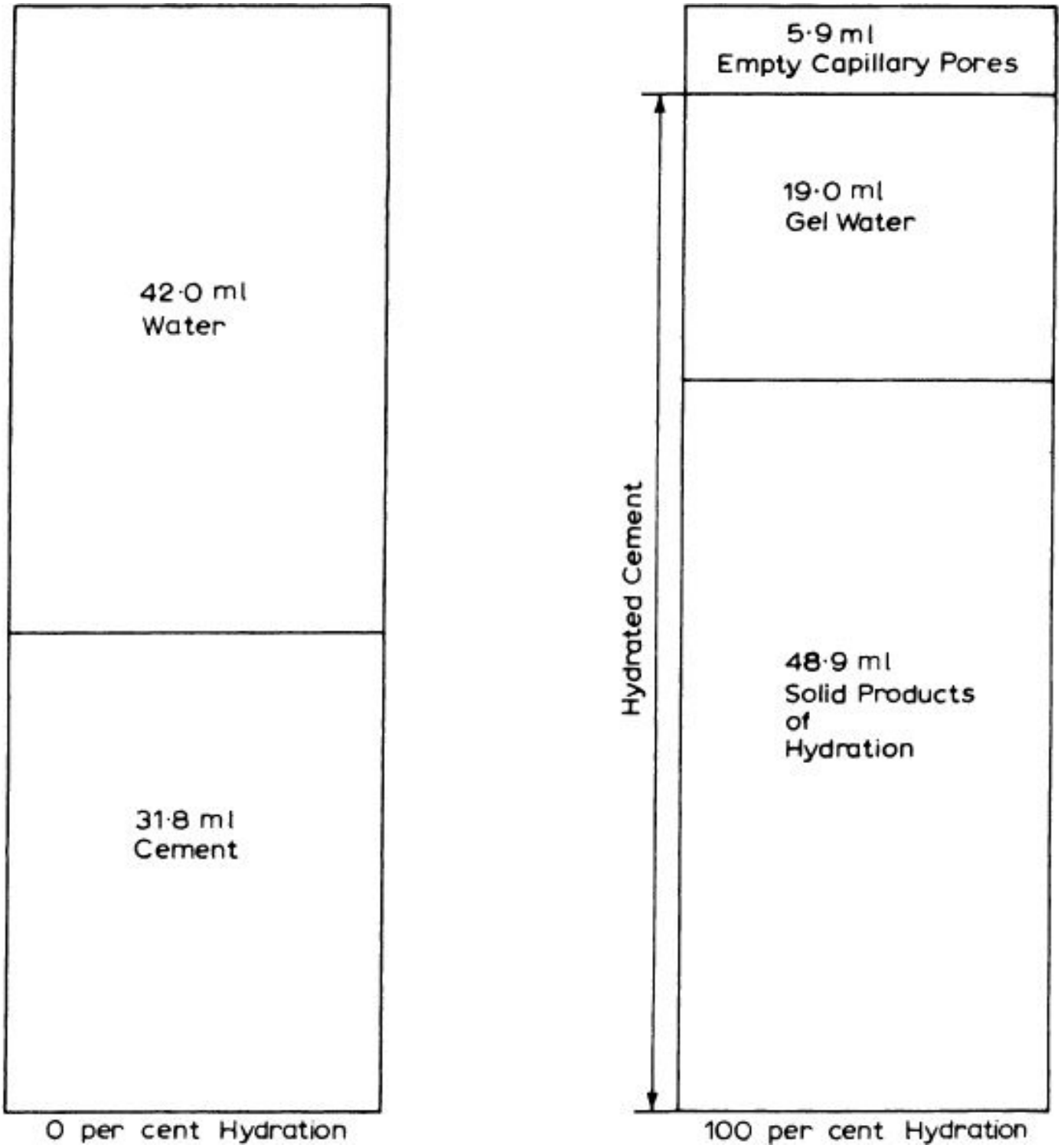


Fig. 1.9. Diagrammatic representation of volume changes on hydration of cement paste with a water/cement ratio of 0.42

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The values given above are only approximate but, had the total amount of water been lower than about 42 ml, it would have been inadequate for full hydration as gel can form only when sufficient water is available both for the chemical reactions *and* for the filling of the gel pores being formed. The gel water, because it is held firmly, cannot move into the capillaries so that it is not available for hydration of the still unhydrated cement.

Thus, when hydration in a sealed specimen has progressed to a stage when the combined water has become about one-half of the original water content, no further hydration will take place. It follows also that full hydration in a sealed specimen is possible only when the mixing water is at least twice the water required for chemical reaction, i.e. the mix has a water/cement ratio of about 0.5 by mass. In practice, in the example given above, the hydration would not in fact have progressed to completion because hydration stops even before the capillaries have become empty. It

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has been found that hydration becomes very slow when the water vapour pressure falls below about 0.8 of the saturation pressure.[1.23]

Let us now consider the hydration of a cement paste cured under water so that water can be imbibed as some of the capillaries become emptied by hydration. As shown before, 100 g of cement (31.8 ml) will, on full hydration, occupy 67.9 ml. Thus, for no unhydrated cement to be left and no capillary pores to be present, the original mixing water should be approximately (67.9 – 31.8) = 36.1 ml. This corresponds to a water/cement ratio of 1.14 by volume or 0.36 by mass. From other work, values of about 1.2 and 0.38, respectively, have been suggested.[1.22]

If the actual water/cement ratio of the mix, allowing for bleeding, is less than about 0.38 by mass, complete hydration is not possible as the volume available is insufficient to accommodate all the products of hydration. It will be recalled that hydration can take place only in water within the capillaries. For instance, if we have a mix

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of 100 g of cement (31.8 ml) and 30 g of water, the water would suffice to hydrate x g of cement, given by the following calculations.

Contraction in volume on hydration is:

$$0.23x \times 0.254 = 0.0585x.$$

Volume occupied by solid products of hydration is:

$$\frac{x}{3.15} + 0.23x - 0.0585x = 0.489x.$$

Porosity is:

$$\frac{w_g}{0.489x + w_g} = 0.28$$

and total water is $0.23x + w_g = 30$. Hence, $x = 71.5$ g $= 22.7$ ml and $w_g = 13.5$ g. Thus, the volume of hydrated cement is

$$0.489 \times 71.5 + 13.5 = 48.5 \text{ ml}.$$

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The volume of unhydrated cement is 31.8 – 22.7 = 9.1 ml. Therefore, the volume of empty capillaries is

$$(31.8 + 30) - (48.5 + 9.1) = 4.2 \text{ ml.}$$

If water is available from outside, some further cement can hydrate, its quantity being such that the products of hydration occupy 4.2 ml more than the volume of dry cement. We found that 22.7 ml of cement hydrates to ocupy 48.5 ml, i.e. the products of hydration of 1 ml of cement occupy 48.5/22.7 = 2.13 ml. Thus 4.2 ml would be filled by the hydration of y ml of cement such that $(4.2 + y)/y = 2.13$; hence, $y = 3.7$ ml. Thus the volume of still unhydrated cement is 31.8 – (22.7 + 3.7) = 5.4 ml, which has a mass of 17 g. In other words, 19 per cent of the original mass of cement has remained unhydrated and can never hydrate because the gel already occupies all the space available, i.e. the gel/space ratio (see p. 274) of the hydrated cement paste is 1.0.

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It may be added that unhydrated cement is not detrimental to strength and, in fact, among cement pastes all with a gel/space ratio of 1.0 those with a higher proportion of unhydrated cement (i.e. a lower water/cement ratio) have a higher strength, possibly because in such pastes the layers of hydrated paste surrounding the unhydrated cement grains are thinner.[1.24]

Abrams obtained strengths of about 280 MPa (40 000 psi) using mixes with a water/cement ratio of 0.08 by mass, but clearly considerable pressure is necessary to obtain a properly consolidated mix of such proportions. Later on, Lawrence[1.52] made compacts of cement powder in a die assembly under a very high pressure (up to 672 MPa (or 97 500 psi)), using the techniques of powder metallurgy. Upon subsequent hydration for 28 days, compressive strengths up to 375 MPa (or 54 500 psi) and tensile strengths up to 25 MPa (or 3600 psi) were measured. The porosity of such mixes and therefore the 'equivalent' water/cement ratio are very low. Even high-

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er strengths, up to 655 MPa (or 95 000 psi), were obtained using very high pressure and a high temperature. The reaction products in these compacts were, however, different from those resulting from normal hydration of cement.[1.89]

In contrast to these compacts which had an extremely low water/cement ratio, if the water/cement ratio is higher than about 0.38 by mass, all the cement can hydrate but capillary pores will also be present. Some of the capillaries will contain excess water from the mix, others will fill by imbibing water from outside. Figure 1.10 shows the relative volumes of unhydrated cement, products of hydration, and capillaries for mixes with different water/cement ratios.

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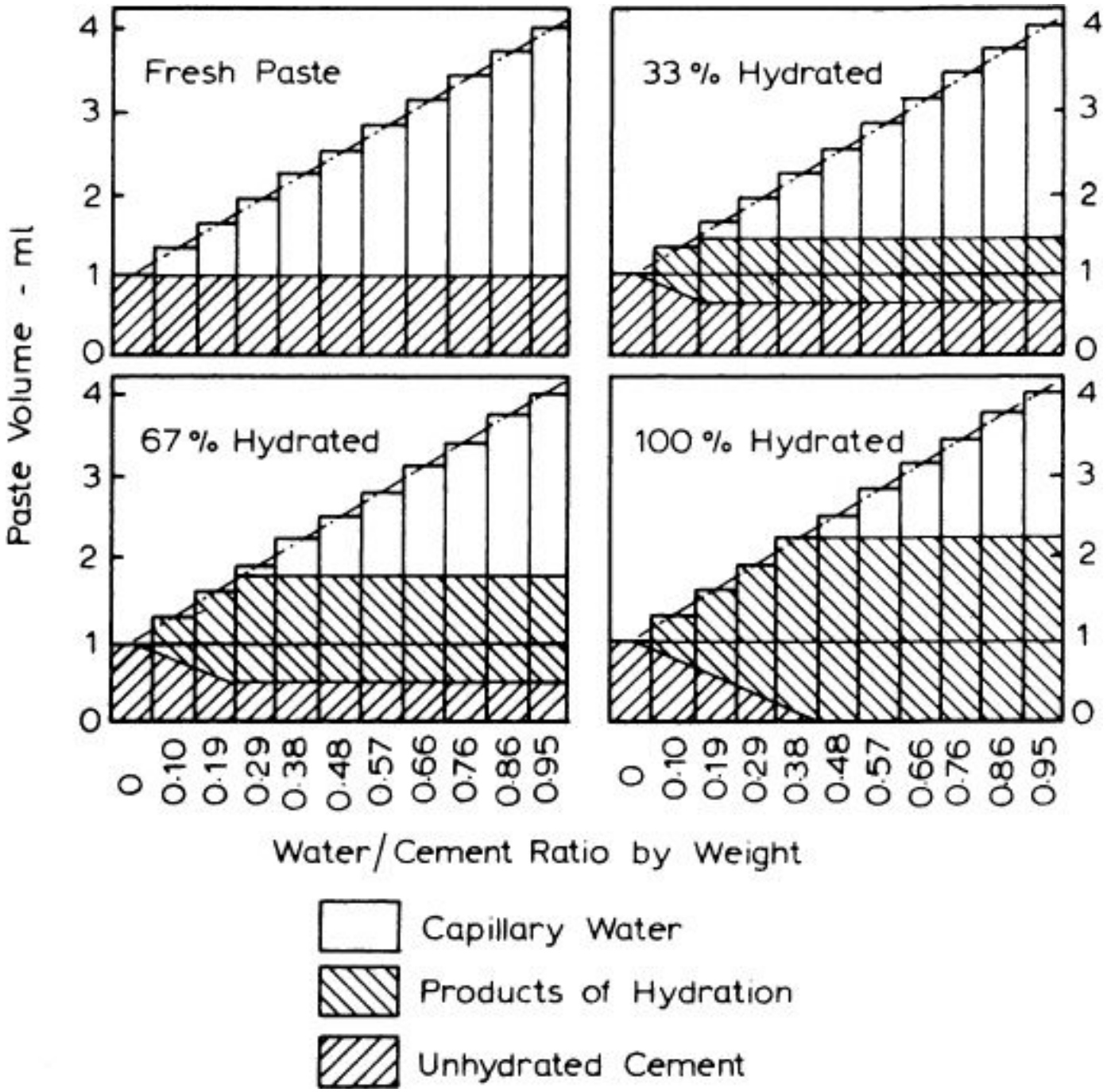


Fig. 1.10. Composition of cement paste at different stages of hydration.[1.10] The percentage indicated applies only to pastes with enough water-filled space to accommodate the products at the degree of hydration indicated

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As a more specific example, let us consider the hydration of a paste with a water/cement ratio of 0.475, sealed in a tube. Let the mass of dry cement be 126 g, which corresponds to 40 ml. The volume of water is then $0.475 \times 126 = 60$ ml. These mix proportions are shown in the left-hand diagram of Fig. 1.11, but in reality the cement and water are of course intermixed, the water forming a capillary system between the unhydrated cement particles.

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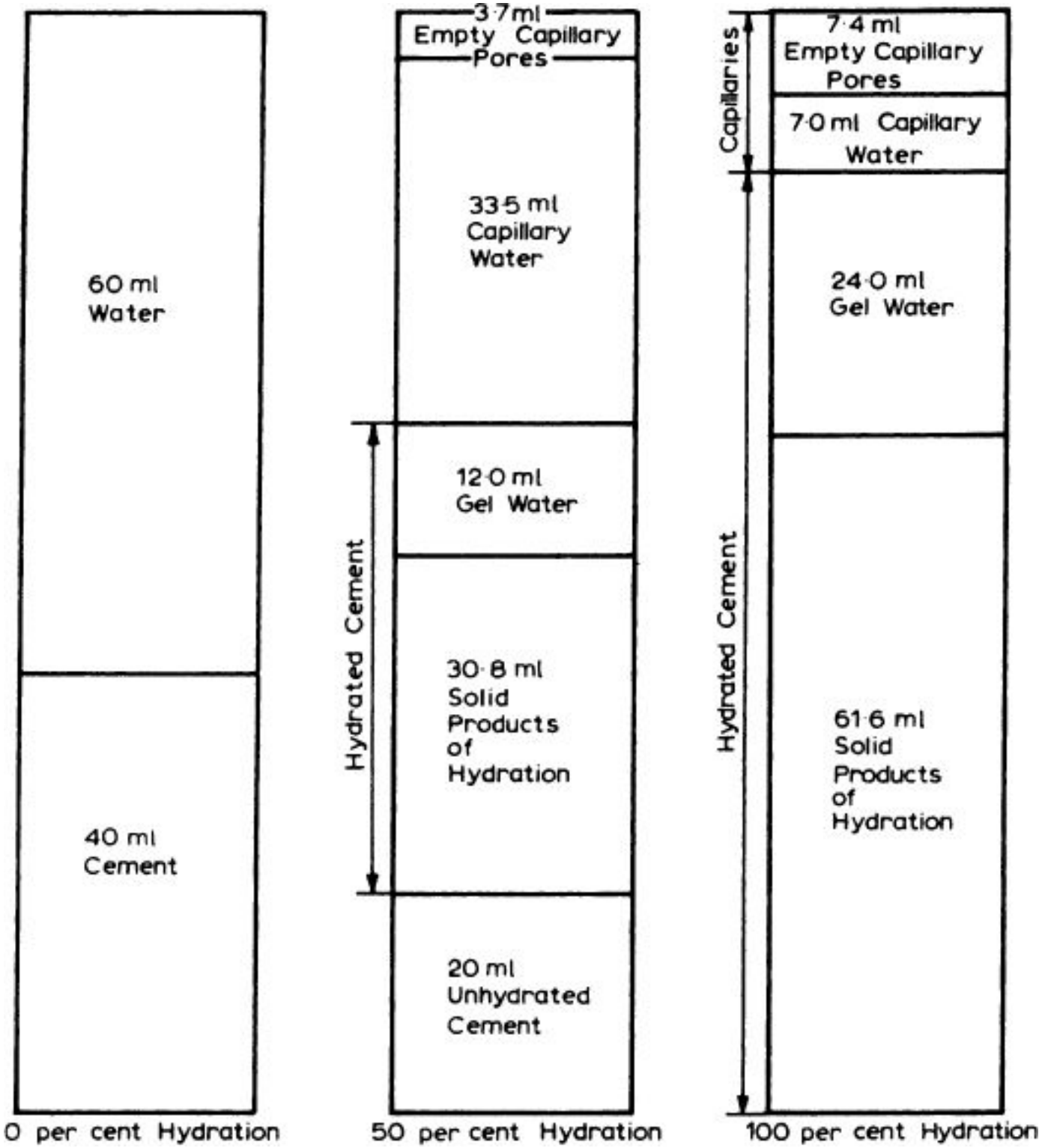


Fig. 1.11. Diagrammatic representation of the volumetric proportions of cement paste at different stages of hydration

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Let us now consider the situation when the cement has hydrated fully. The non-evaporable water is 0.23 × 126 = 29.0 ml and the gel water is w_g such that

$$\frac{w_g}{40 + 29.0(1 - 0.254) + w_g} = 0.28,$$

whence the volume of gel water is 24.0 ml, and the volume of hydrated cement is 85.6 ml. There are thus 60 – (29.0 + 24.0) = 7.0 ml of water left as capillary water in the paste. In addition, 100 – (85.6 + 7.0) = 7.4 ml form empty capillaries. If the cement paste had access to water during curing these capillaries would fill with imbibed water.

This then is the situation at 100 per cent hydration when the gel/space ratio is 0.856, as shown in the right-hand diagram of Fig. 1.11. As a further illustration, the centre diagram shows the volumes of different components when only half the cement has hydrated. The gel/space ratio is then

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$$\frac{\frac{1}{2}[40 + 29(1 - 0.254) + 24]}{100 - 20} = 0.535.$$

An approach similar to that of Powers, outlined above, has been applied to cements containing a limestone filler (see p. 88).[1.97]

Capillary pores

We can thus see that, at any stage of hydration, the capillary pores represent that part of the gross volume which has not been filled by the products of hydration. Because these products occupy more than twice the volume of the original solid phase (i.e. cement) alone, the volume of the capillary system is reduced with the progress of hydration.

Thus the capillary porosity of the paste depends both on the water/cement ratio of the mix and on the degree of hydration. The rate of hydration of the cement is of no importance *per se*, but the type of cement influences the degree of hydration achieved at a given age. As mentioned before, at water/cement ratios higher than about

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0.38, the volume of the gel is not sufficient to fill all the space available to it so that there will be some volume of capillary pores left even after the process of hydration has been completed.

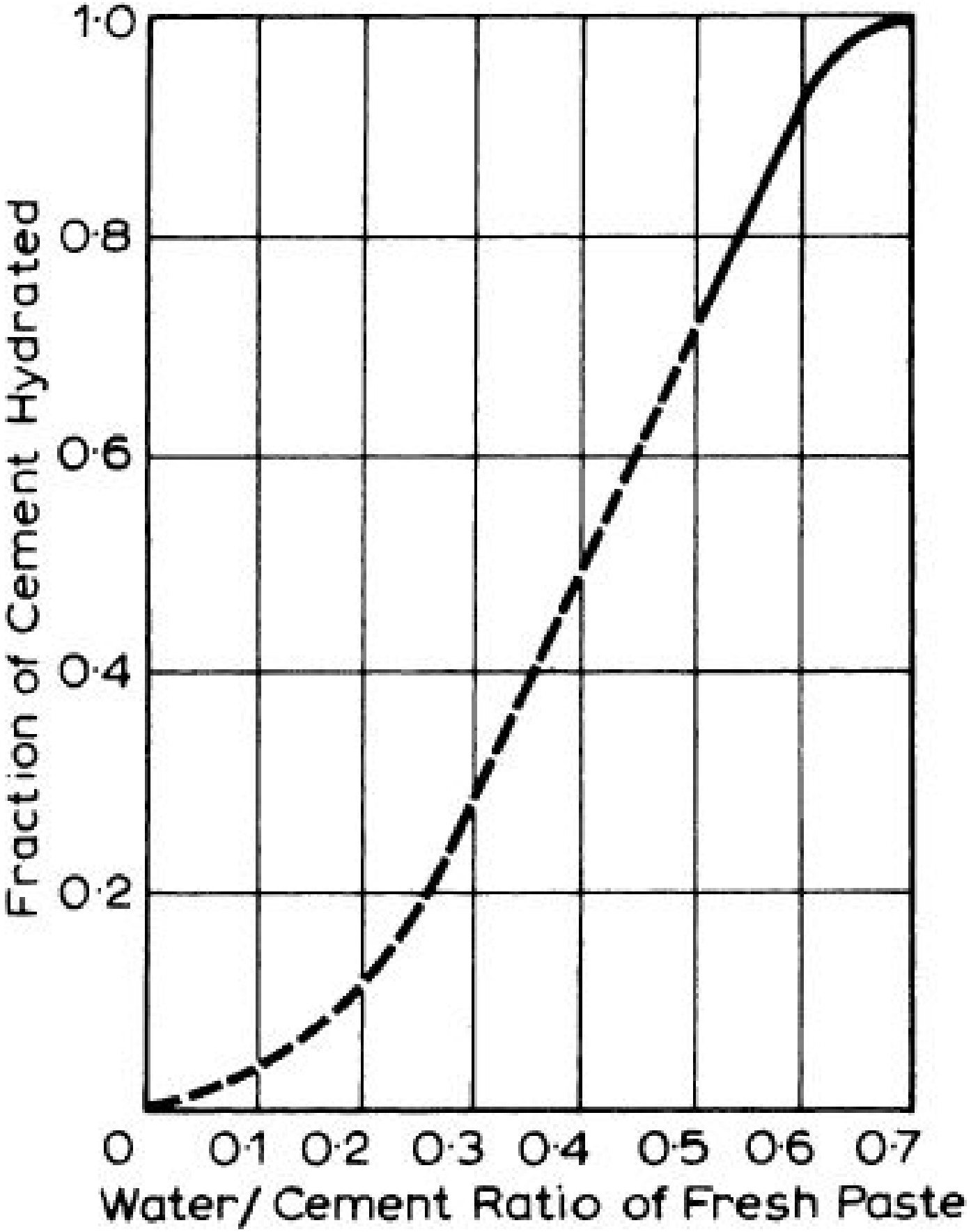
Capillary pores cannot be viewed directly but their median size was estimated from vapour pressure measurement to be about 1.3 μm. In fact, the size of pores in hydrated cement paste varies widely. Glasser's studies[1.85] indicate that mature cement pastes contain few pores larger than 1 μm, with most pores being smaller than 100 nm. They vary in shape but, as shown by measurement of permeability, form an interconnected system randomly distributed throughout the cement paste.[125] These interconnected capillary pores are mainly responsible for the permeability of the hardened cement paste and for its vulnerability to cycles of freezing and thawing.

However, hydration increases the solid content of the paste and, in mature and dense pastes, the capillaries can become blocked by gel and segmented so that they turn into capillary pores

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interconnected solely by the gel pores. The absence of continuous capillaries is due to a combination of a suitable water/cement ratio and a sufficiently long period of moist curing; the degree of maturity required for different water/cement ratios for ordinary Portland cements is indicated in Fig. 1.12. The actual time to achieve the required maturity depends on the characteristics of the cement used, but approximate values of the time required can be gauged from the data of Table 1.6. For water/cement ratios above about 0.7, even complete hydration would not produce enough gel to block all the capillaries. For extremely fine cement, the maximum water/cement ratio would be higher, possibly up to 1.0; conversely, for coarse cements, it would be below 0.7. The importance of eliminating continuous capillaries is such that this might be regarded a necessary condition for a concrete to be classified as ‘good’.

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Fig. 1.12. Relation between the water/cement ratio and the degree of hydration at which the capillaries cease to be continuous[1.26]

Table 1.6. Approximate Age Required to Produce Maturity at which Capillaries Become Segmented[1.26]

Water I cement ratio by weight	*Time required*
0.40	3 days
0.45	7 days
0.50	14 days
0.60	6 months
0.70	1 year
over 0.70	impossible

Gel pores

Let us now consider the gel itself. From the fact that it can hold large quantities of evaporable water it follows that the gel is porous, but the gel pores are really interconnected interstitial spaces

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between the gel particles, which are needle-, plate-, and foil-shaped. The gel pores are much smaller than the capillary pores: less than 2 or 3 nm in nominal diameter. This is only one order of magnitude greater than the size of molecules of water. For this reason, the vapour pressure and mobility of adsorbed water are different from the corresponding properties of free water. The amount of reversible water indicates directly the porosity of the gel.[1.24]

The gel pores occupy about 28 per cent of the total volume of gel, the material left after drying in a standard manner[1.48] being considered as solids. The actual value is characteristic for a given cement but is largely independent of the water/cement ratio of the mix and of the progress of hydration. This would indicate that gel of similar properties is formed at all stages and that continued hydration does not affect the products already in existence. Thus, as the total volume of gel increases with the progress of hydration, the total volume of gel pores also increases. On the other

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hand, as mentioned earlier, the volume of capillary pores decreases with the progress of hydration.

Porosity of 28 per cent means that the gel pores occupy a space equal to about one-third of the volume of the gel solids. The ratio of the surface of the solid part of the gel to the volume of the solids is equal to that of spheres about 9 nm in diameter. This must not be construed to mean that gel consists of spherical elements; the solid particles are of varied shapes, and bundles of such particles form a cross-linked network containing some more or less amorphous interstitial material.[1.27]

Another way of expressing the porosity of the gel is to say that the volume of the pores is about three times the volume of the water forming a layer one molecule thick over the entire solid surface in the gel.

From measurements of water adsorption, the specific surface of gel has been estimated to be of

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the order of 5.5×10^8 m^2 per m^3, or approximately 200 000 m^2/kg.[1.27] Small-angle X-ray scattering measurements have yielded values of the order of 600 000 m^2/kg, indicating a large internal surface within the particles.[1.63] By contrast, unhydrated cement has a specific surface of some 200 to 500 m^2/kg. At the other extreme, silica fume has a specific surface of 22 000 m^2/kg.

In connection with the pore structure, it may be relevant to note that high-pressure steam-cured cement paste has a specific surface of some 7000 m^2/kg only. This indicates an entirely different particle size of the products of hydration at a high pressure and a high temperature and, in fact, such treatment results in an almost entirely micro-crystalline material rather than gel.

The specific surface of normally cured cement paste depends on the curing temperature and on the chemical composition of cement. It has been suggested[1.27] that the ratio of the specific surface to the mass of non-evaporable water (which in

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turn is proportional to the porosity of the hydrated cement paste) is proportional to

$$0.230(C_3S) + 0.320(C_2S) + 0.317(C_3A) + 0.368(C_4AF),$$

where the symbols in brackets represent the percentages of the compounds present in the cement. There seems to be little variation between the numerical coefficients of the last three compounds, and this indicates that the specific surface of the hydrated cement paste varies little with a change in the composition of cement. The rather lower coefficient of C_3S is due to the fact that it produces a large quantity of micro-crystalline $Ca(OH)_2$, which has a very much lower specific surface than the gel.

The proportionality between the mass of water forming a monomolecular layer over the surface of the gel and the mass of non-evaporable water in the paste (for a given cement) means that gel of nearly the same specific surface is formed throughout the progress of hydration. In other

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words, particles of the same size are formed all the time and the already existing gel particles do not grow in size. This is not, however, the case in cements with a high C_2S content.[1.28]

Mechanical strength of cement gel

There are two classical theories of hardening or development of strength of cement. That put forward by H. Le Chatelier in 1882 states that the products of hydration of cement have a lower solubility than the original compounds, so that the hydrates precipitate from a supersaturated solution. The precipitate is in the form of interlaced elongated crystals with high cohesive and adhesive properties.

The colloidal theory propounded by W. Michaëlis in 1893 states that the crystalline aluminate, sulfoaluminate and hydroxide of calcium give the initial strength. The lime-saturated water then attacks the silicates and forms a hydrated calcium silicate which, being almost insoluble, forms a gelatinous mass. This mass hardens

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gradually due to the loss of water either by external drying or by hydration of the inner unhydrated core of the cement grains: in this manner cohesion is obtained.

In the light of modern knowledge it appears that both theories contain elements of truth and are in fact by no means irreconcilable. In particular, colloid chemists have found that many, if not most, colloids consist of crystalline particles but these, being extremely small, have a large surface area which gives them what appear to be different properties from the usual solids. Thus colloidal behaviour is essentially a function of the size of the surface area rather than of the non-regularity of internal structure of the particles involved.[1.42]

In the case of Portland cement, it has been found that, when mixed with a large quantity of water, cement produces within a few hours a solution supersaturated with $Ca(OH)_2$ and containing concentrations of calcium silicate hydrate in a metastable condition.[1.2] This hydrate rapidly precipitates in agreement with Le Chatelier's hy-

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pothesis; the subsequent hardening may be due to the withdrawal of water from the hydrated material as postulated by Michaëlis. Following the dormant period, precipitation of calcium silicate hydrate and $Ca(OH)_2$ continues.

Further experimental work has shown that the calcium silicate hydrates are in fact in the form of extremely small (measured in nanometres) interlocking crystals[1.20] which, because of their size, could equally well be described as gel. When cement is mixed with a small quantity of water, the degree of crystallization is probably even poorer, the crystals being ill-formed. Thus the Le Chatelier–Michaëlis controversy is largely reduced to a matter of terminology as we are dealing with a gel consisting of crystals. Moreover, the solubility of silica increases very substantially at a pH above 10, so that it is possible for the Michaëlis mechanism to operate initially and for that of Le Chatelier later on. A more detailed discussion of the two mechanisms is offered by Baron and Santeray.[1.94]

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The term ‘cement gel’ is considered, for convenience, albeit not correctly, to include the crystalline calcium hydroxide. Gel is thus taken to mean the cohesive mass of hydrated cement in its densest paste, i.e. inclusive of gel pores, the characteristic porosity being about 28 per cent.

The actual source of strength of the gel is not fully understood but it probably arises from two kinds of cohesive bonds.[1.27] The first type is the physical attraction between solid surfaces, separated only by the small (less than 3 nm) gel pores; this attraction is usually referred to as van der Waals’ forces.

The source of the second type of cohesion is the chemical bonds. Because cement gel is of the limited swelling type (i.e. the particles cannot be dispersed by addition of water) it seems that the gel particles are cross-linked by chemical forces. These are much stronger than van der Waals’ forces but the chemical bonds cover only a small fraction of the boundary of the gel particles. On the other hand, a surface area as high as that of

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cement gel is not a necessary condition for high strength development, as high-pressure steam-cured cement paste, which has a low surface area, exhibits extremely good hydraulic properties.[1.14]

We cannot thus estimate the relative importance of the physical and chemical bonds but there is no doubt that both contribute to the very considerable strength of the hardened cement paste. It has to be admitted that the understanding of the cohesive nature of the hydrated cement paste and its adhesion to aggregate is still imperfect. As Nonat and Mutin[1.92] put it, the microstructure has not been related in a general way to mechanical properties.

Water held in hydrated cement paste

The presence of water in hydrated cement has been repeatedly mentioned. The cement paste is indeed hygroscopic owing to the hydrophilic character of cement coupled with the presence of sub-microscopic pores. The actual water content of the paste depends on the ambient humidity. In

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particular, capillary pores, because of their comparatively large size, empty when the ambient relative humidity falls below about 45 per cent,[1.25] but water is adsorbed in the gel pores even at very low ambient humidities.

We can thus see that water in hydrated cement is held with varying degrees of firmness. At one extreme, there is free water; at the other, chemically combined water forming a definite part of the hydrated compounds. Between these two categories, there is gel water held in a variety of other ways.

The water held by the surface forces of the gel particles is called adsorbed water, and that part of it which is held between the surfaces of certain planes in a crystal is called interlayer or zeolitic water. Lattice water is that part of the water of crystallization which is not chemically associated with the principal constituents of the lattice. The diagrammatic representation of Fig. 1.13 may be of interest.

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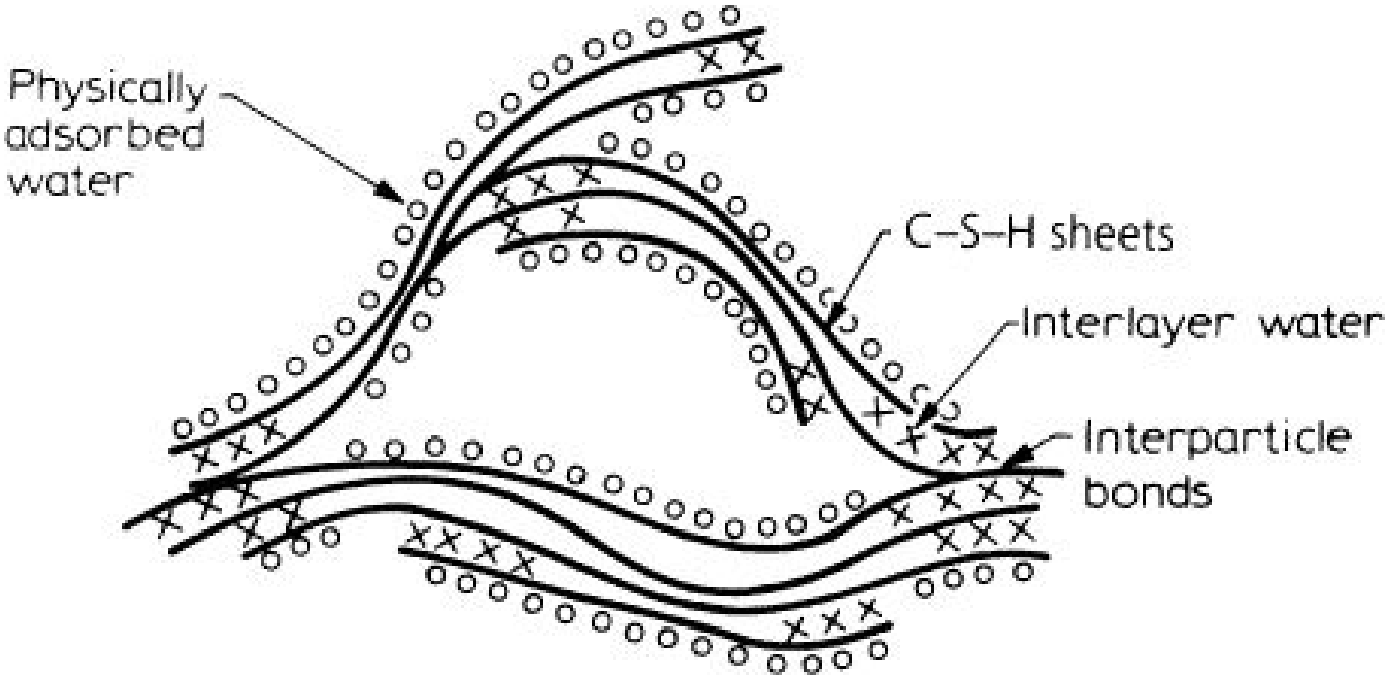


Fig. 1.13. Probable structure of hydrated silicates[1.53]

Free water is held in capillaries and is beyond the range of the surface forces of the solid phase.

There is no technique available for determining how water is distributed between these different states, nor is it easy to predict these divisions from theoretical considerations as the energy of binding of combined water in the hydrate is of the same order of magnitude as the energy of binding of the adsorbed water. However, investigations using nuclear magnetic resonance suggest that gel water has the same energy of binding as

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interlayer water in some swelling clays; thus the gel water may well be in interlayer form.[1.54]

A convenient division of water in the hydrated cement, necessary for investigation purposes, though rather arbitrary, is into two categories: evaporable and non-evaporable. This is achieved by drying the cement paste to equilibrium (i.e. to a constant mass) at a given vapour pressure. The usual value is 1 Pa at 23 °C, obtained over $Mg(ClO_4)_2.2H_2O$. Drying in an evacuated space which is connected to a moisture trap held at –79 °C has also been used; this corresponds to a vapour pressure of 0.07 Pa.[1.48] Alternatively, the evaporable water can be determined by the loss upon drying at a higher temperature, usually 105 °C, or by freezing out, or by removing with a solvent.

All these methods essentially divide water according to whether or not it can be removed at a certain reduced vapour pressure. Such a division is perforce arbitrary because the relation between vapour pressure and water content of cement is

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continuous; in contrast to crystalline hydrates, no breaks occur in this relationship. However, in general terms, the non-evaporable water contains nearly all chemically combined water and also some water not held by chemical bonds. This water has a vapour pressure lower than that of the ambient atmosphere and the quantity of such water is, in fact, a continuous function of the ambient vapour pressure.

The amount of non-evaporable water increases as hydration proceeds but, in a saturated paste, non-evaporable water can never become more than one-half of the total water present. In well-hydrated cement, the non-evaporable water is about 18 per cent by mass of the anhydrous material; this proportion rises to about 23 per cent in fully hydrated cement.[1.1] It follows from the proportionality between the amount of non-evaporable water and the solid volume of the cement paste that the former volume can be used as a measure of the quantity of the cement gel present, i.e. of the degree of hydration.

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The manner in which water is held in a cement paste determines the energy of binding. For instance, 1670 J (400 calories) are used in establishing the bond of 1 g of non-evaporable water, while the energy of the water of crystallization of $Ca(OH)_2$ is 3560 J/g (850 cal/g). Likewise, the density of the water varies; it is approximately 1.2 for non-evaporable water, 1.1 for gel water, and 1.0 for free water.[1.24] It has been suggested that the increase in the density of the adsorbed water at low surface concentrations is not the result of compression but is caused by the orientation, or ordering, of the molecules in the adsorbed phase due to the action of the surface forces,[1.12] resulting in a so-called disjoining pressure. The disjoining pressure is the pressure expected to maintain the film of adsorbed molecules against external action. A confirmation of the hypothesis that the properties of adsorbed water are different from those of free water is afforded by measurements of the absorption of microwaves by hardened cement paste.[1.64]

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Heat of hydration of cement

In common with many chemical reactions, the hydration of cement compounds is exothermic, energy of up to 500 J/g (120 cal/g) of cement being liberated. Because the thermal conductivity of concrete is comparatively low, it acts as an insulator, and in the interior of a large concrete mass, hydration can result in a large rise in temperature. At the same time, the exterior of the concrete mass loses some heat so that a steep temperature gradient may be established and, during subsequent cooling of the interior, serious cracking may result. This behaviour is, however, modified by the creep of concrete or by insulation of the surfaces of the concrete mass.

At the other extreme, the heat produced by the hydration of cement may prevent freezing of the water in the capillaries of freshly placed concrete in cold weather, and a high evolution of heat is therefore advantageous. It is clear, then, that it is advisable to know the heat-producing properties of different cements in order to choose the most

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suitable cement for a given purpose. It may be added that the temperature of young concrete can also be influenced by artificial heating or cooling.

The heat of hydration is the quantity of heat, in joules per gram of unhydrated cement, evolved upon complete hydration at a given temperature. The most common method of determining the heat of hydration is by measuring the heats of solution of unhydrated and hydrated cement in a mixture of nitric and hydrofluoric acids: the difference between the two values represents the heat of hydration. This method is described in BS 4550-3.8 : 1978, and is similar to the method of ASTM C 186-05. While there are no particular difficulties in this test, care should be taken to prevent carbonation of the unhydrated cement because the absorption of 1 per cent of CO_2 results in an apparent decrease in the heat of hydration of 24.3 J/g (5.8 cal/g) out of a total of between 250 and over 420 J/g (60 and 100 cal/g).[1.29]

The temperature at which hydration takes place greatly affects the rate of heat development,

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as shown by the data of Table 1.7, which gives the heat developed in 72 hours at different temperatures.[1.30] There is little effect of the temperature on the long-term value of the heat of hydration.[1.82]

Table 1.7. Heat of Hydration Developed After 72 Hours at Different Temperatures[1.30]

Cement Type	Heat of hydration developed at:							
	4 °C (40 °F)		24 °C (75 °F)		32 °C (90 °F)		41 °C (105 °F)	
	J/g	cal/g	J/g	cal/g	J/g	cal/g	J/g	cal/g
I	154	36.9	285	68.0	309	73.9	335	80.0
III	221	52.9	348	83.2	357	85.3	390	93.2
IV	108	25.7	195	46.6	192	45.8	214	51.2

Strictly speaking, the heat of hydration, as measured, consists of the chemical heat of the reactions of hydration and the heat of adsorption of water on the surface of the gel formed by the processes of hydration. The latter heat accounts for about a quarter of the total heat of hydration. Thus, the heat of hydration is really a composite quantity.[1.24]

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For practical purposes, it is not necessarily the total heat of hydration that matters but the rate of heat evolution. The same total heat produced over a longer period can be dissipated to a greater degree with a consequent smaller rise in temperature. The rate of heat development can be easily measured in an adiabatic calorimeter, and typical time–temperature curves obtained under adiabatic conditions are shown in Fig. 1.14. (The ratio 1:2:4 represents the proportion by mass of cement:fine aggregate:coarse aggregate.)

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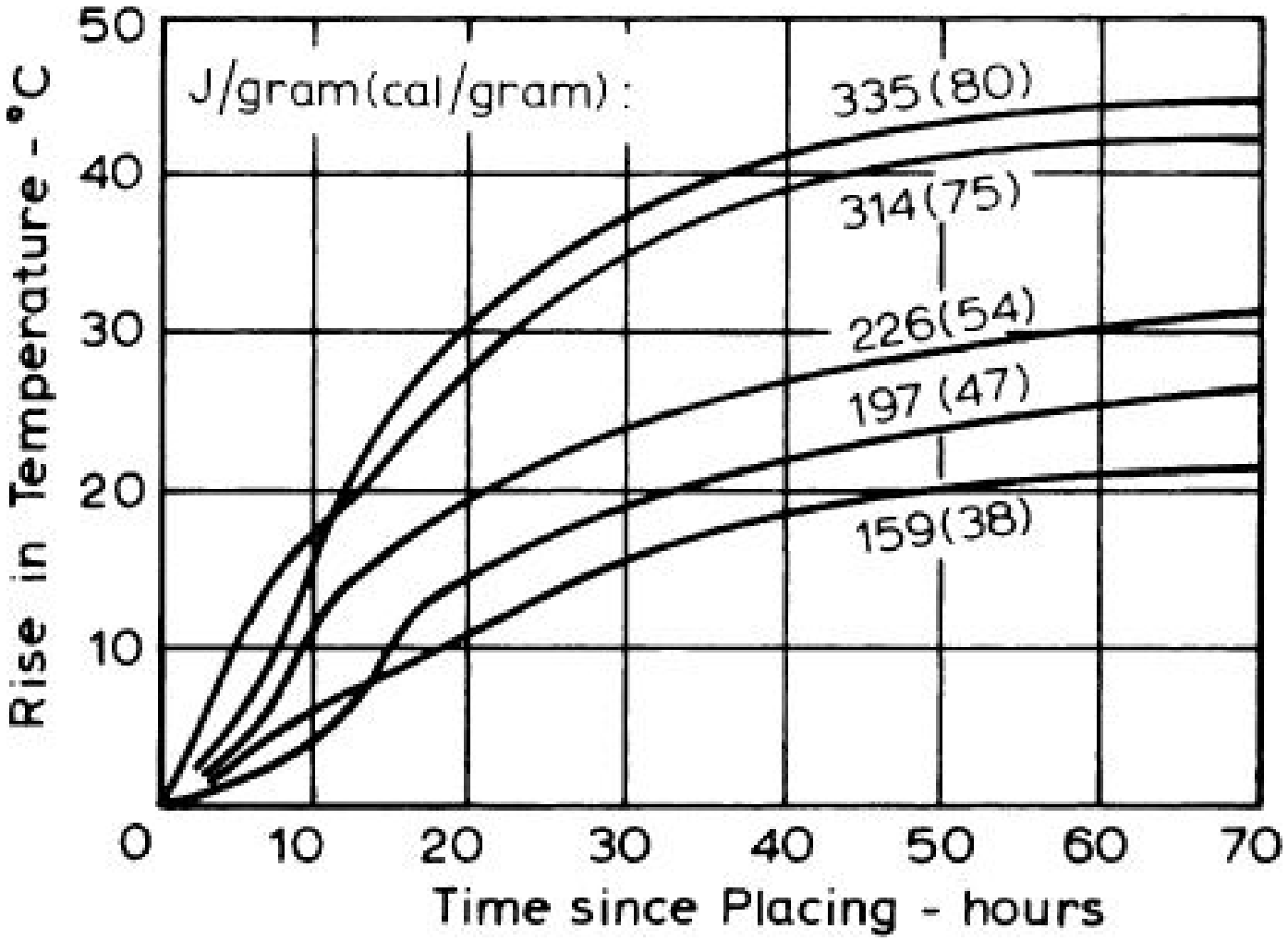


Fig. 1.14. Temperature rise in 1:2:4 concrete (water/cement ratio of 0.60) made with different cements and cured adiabatically.[1.31] The total heat of hydration of each cement at three days is shown (Crown copyright)

For the usual range of Portland cements, Bogue[1.2] observed that about one-half of the total heat is evolved between 1 and 3 days, about three-quarters in 7 days, and 83 to 91 per cent of

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the total heat in 6 months. The actual value of the heat of hydration depends on the chemical composition of the cement, and is very nearly a sum of the heats of hydration of the individual compounds when hydrated separately. It follows that, given the compound composition of a cement, its heat of hydration can be calculated with a fair degree of accuracy. Typical values of the heat of hydration of pure compounds are given in Table 1.8.

Table 1.8. Heat of Hydration of Pure Compounds[1.32]

Compound	*Heat of hydration*	
	J/g	*cal/g*
C_3S	502	120
C_2S	260	62
C_3A	867	207
C_4AF	419	100

It may be noted that there is no relation between the heat of hydration and the cementing

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properties of the individual compounds. Woods, Steinour and Starke[1.33] tested a number of commercial cements and, using the method of least squares, calculated the contribution of individual compounds to the total heat of hydration of cement. They obtained equations of the following type: heat of hydration of 1 g of cement is

$$136(C_3S) + 62(C_2S) + 200(C_3A) + 30(C_4AF)$$

where the terms in brackets denote the percentage by mass of the individual compounds in cement. Later work[1.83] broadly confirmed the contribution of the various compounds to the heat of hydration of cement except for C_2S whose contribution was found to be about one-half of that given above.

Because in the early stages of hydration the different compounds hydrate at different rates, the *rate* of heat evolution, as well as the total heat, depends on the compound composition of

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the cement. It follows that by reducing the proportions of the compounds that hydrate most rapidly (C_3A and C_3S) the high rate of heat evolution in the early life of concrete can be lowered. The fineness of the cement also influences the rate of heat development, an increase in fineness speeding up the reactions of hydration and therefore the heat evolved. It is reasonable to assume that the early rate of hydration of each compound in cement is proportional to the surface area of the cement. However, at later stages, the effect of the surface area is negligible and the total amount of heat evolved is not affected by the fineness of cement.

The influence of C_3A and C_3S can be gauged from Figs 1.15 and 1.16. As mentioned before, for many uses of concrete, a controlled heat evolution is advantageous and suitable cements have been developed. One such cement is low heat Portland cement discussed in more detail in Chapter 2. The rate of heat development of this and other cements is shown in Fig. 1.17.

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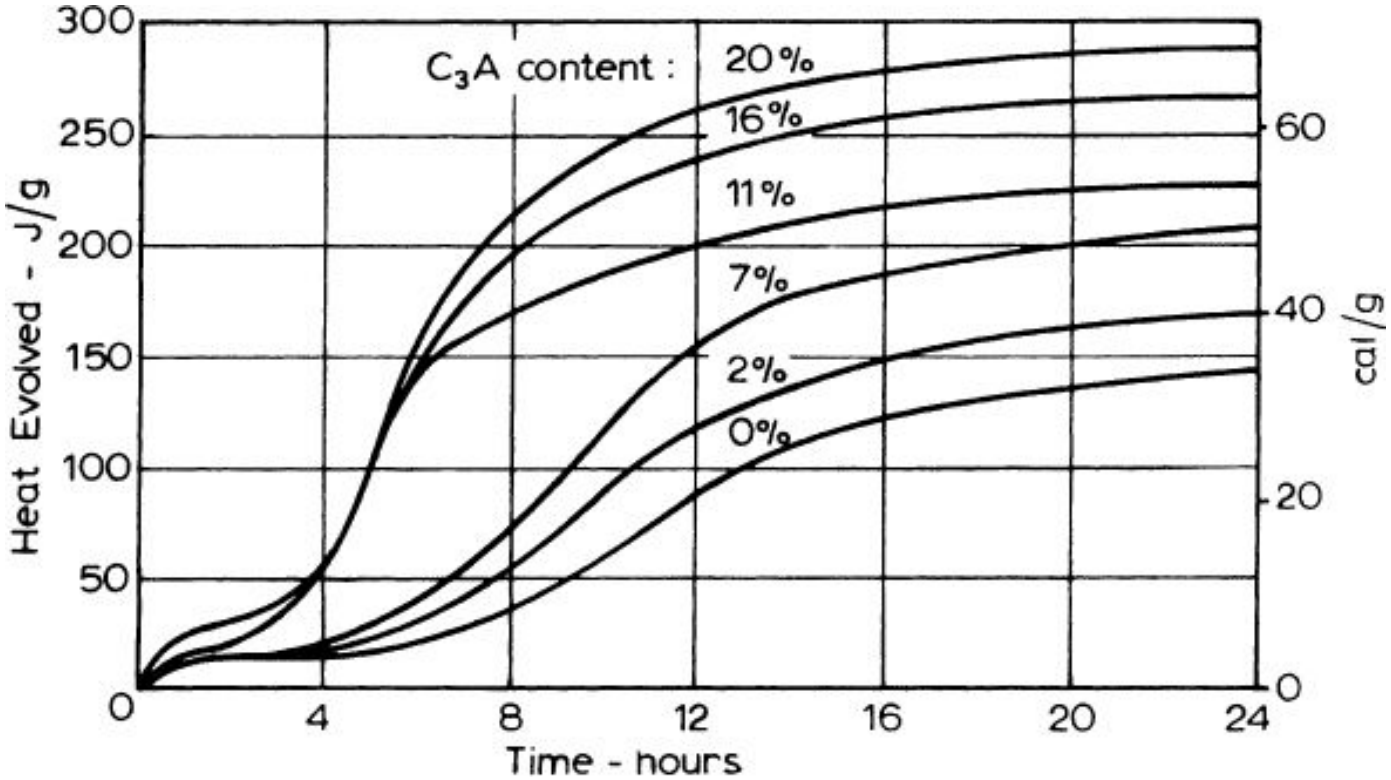


Fig. 1.15. Influence of C_3A content on heat evolution[1.32] (C_3S content approximately constant)

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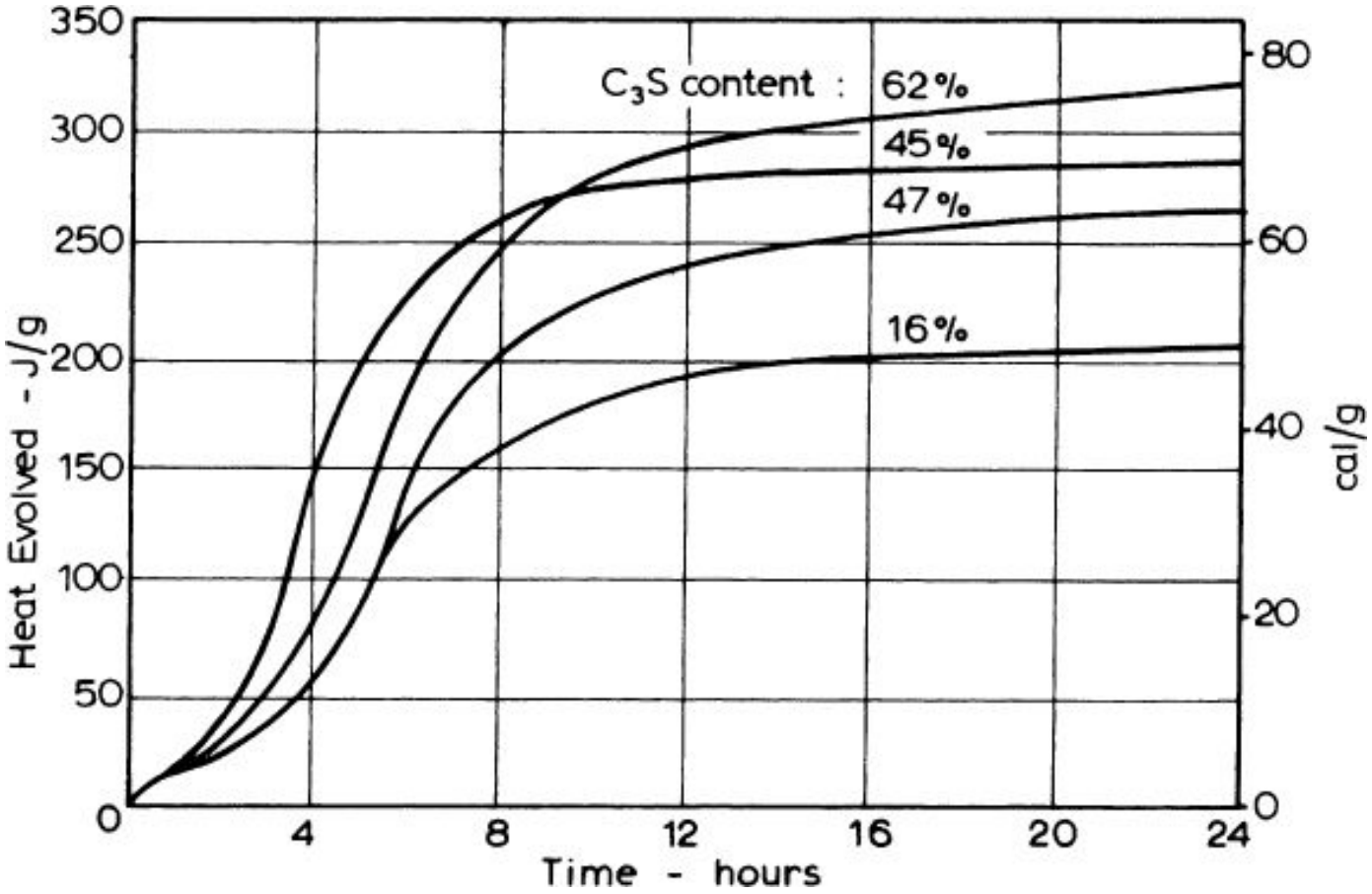


Fig. 1.16. Influence of C_3S content on heat evolution[1.32] (C_3A content approximately constant)

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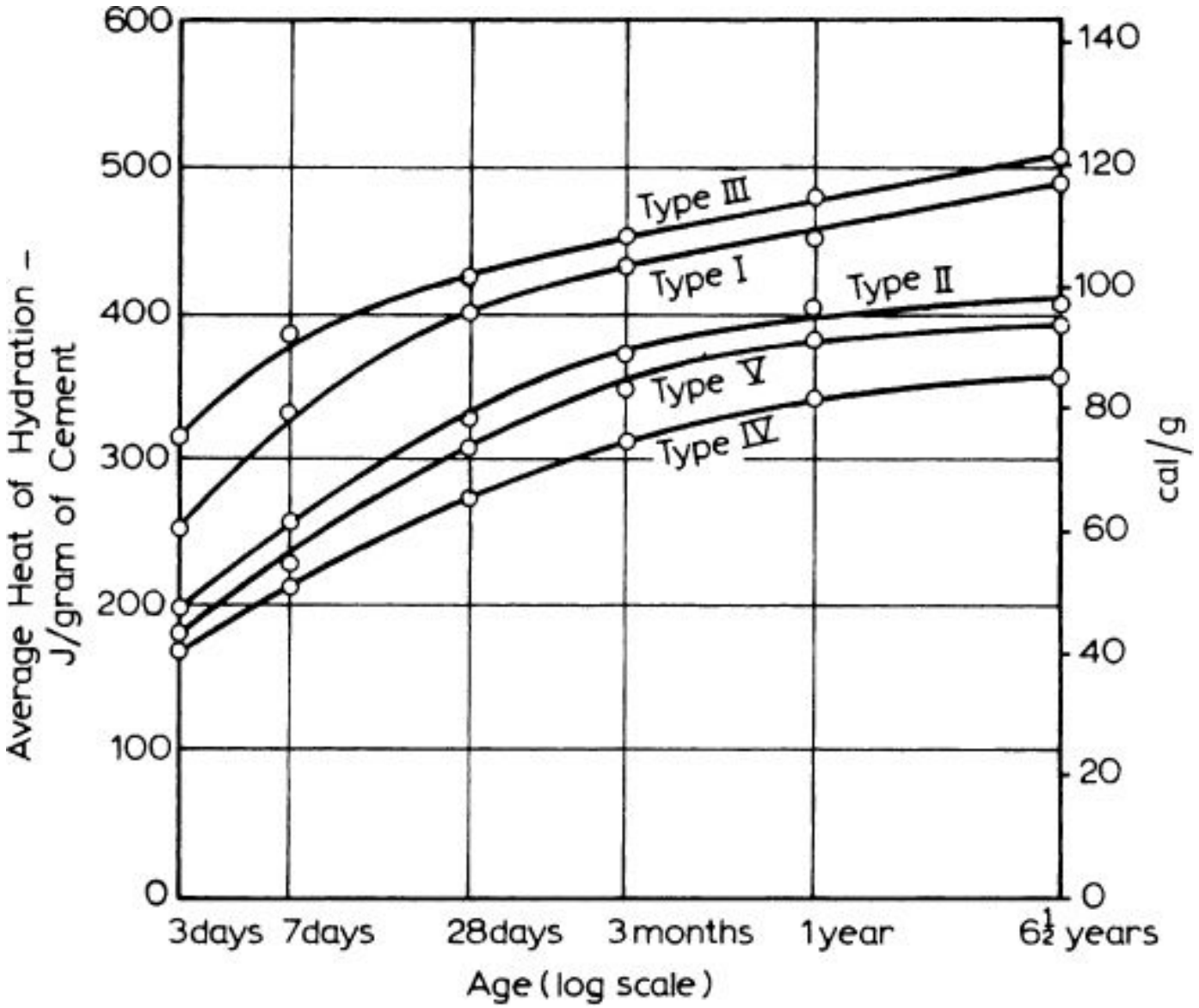


Fig. 1.17. Development of heat of hydration of different cements cured at 21 °C (70 °F) (water/cement ratio of 0.40)[1.34]

The quantity of cement in the mix also affects the total heat development: thus the richness of the mix, that is, the cement content, can be varied in order to help the control of heat development.

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Influence of the compound composition on properties of cement

In the preceding section, it was shown that the heat of hydration of cement is a simple additive function of the compound composition of cement. It would seem, therefore, that the various hydrates retain their identity in the cement gel, which can be considered thus to be a fine physical mixture or to consist of copolymers of the hydrates. A further corroboration of this supposition is obtained from the measurement of specific surface of hydrated cements containing different amounts of C_3S and C_2S: the results agree with the specific surface areas of hydrated neat C_3S and C_2S. Likewise, the water of hydration agrees with the additivity of the individual compounds.

This argument does not, however, extend to all properties of hardened cement paste, notably to shrinkage, creep, and strength; nevertheless, the compound composition gives some indication of the properties to be expected. In particular, the

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composition controls the rate of evolution of the heat of hydration and the resistance of cement to sulfate attack, so that limiting values of oxide or compound composition of different types of cement are prescribed by some specifications. The limitations of ASTM C 150-09 are less restrictive than they used to be (see Table 1.9).

Table 1.9. Compound Composition Limits for Cements of ASTM C 150-09

Compound	*Cement Type*				
	I	*II*	*III*	*IV*	*V*
C_3S maximum				35	
C_2S minimum				40	
C_3A maximum		8	15	7	5
$C_4AF + 2(C_3A)$ maximum					25*

*ASTM C 150-94.

The difference in the early rates of hydration of C_3S and C_2S – the two silicates primarily responsible for the strength of hydrated cement paste – has been mentioned earlier. A convenient approximate rule assumes that C_3S contributes most to the strength development during the first

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four weeks and C_2S influences the gain in strength from 4 weeks onwards.[1.35] At the age of about one year, the two compounds, mass for mass, contribute approximately equally to the ultimate strength.[1.36] Pure C_3S and C_2S have been found to have a strength of about 70 MPa (10 000 psi) at the age of 18 months, but at the age of 7 days C_2S had no strength while the strength of C_3S was about 40 MPa (6000 psi). The usually accepted development of strength of pure compounds is shown in Fig. 1.18.

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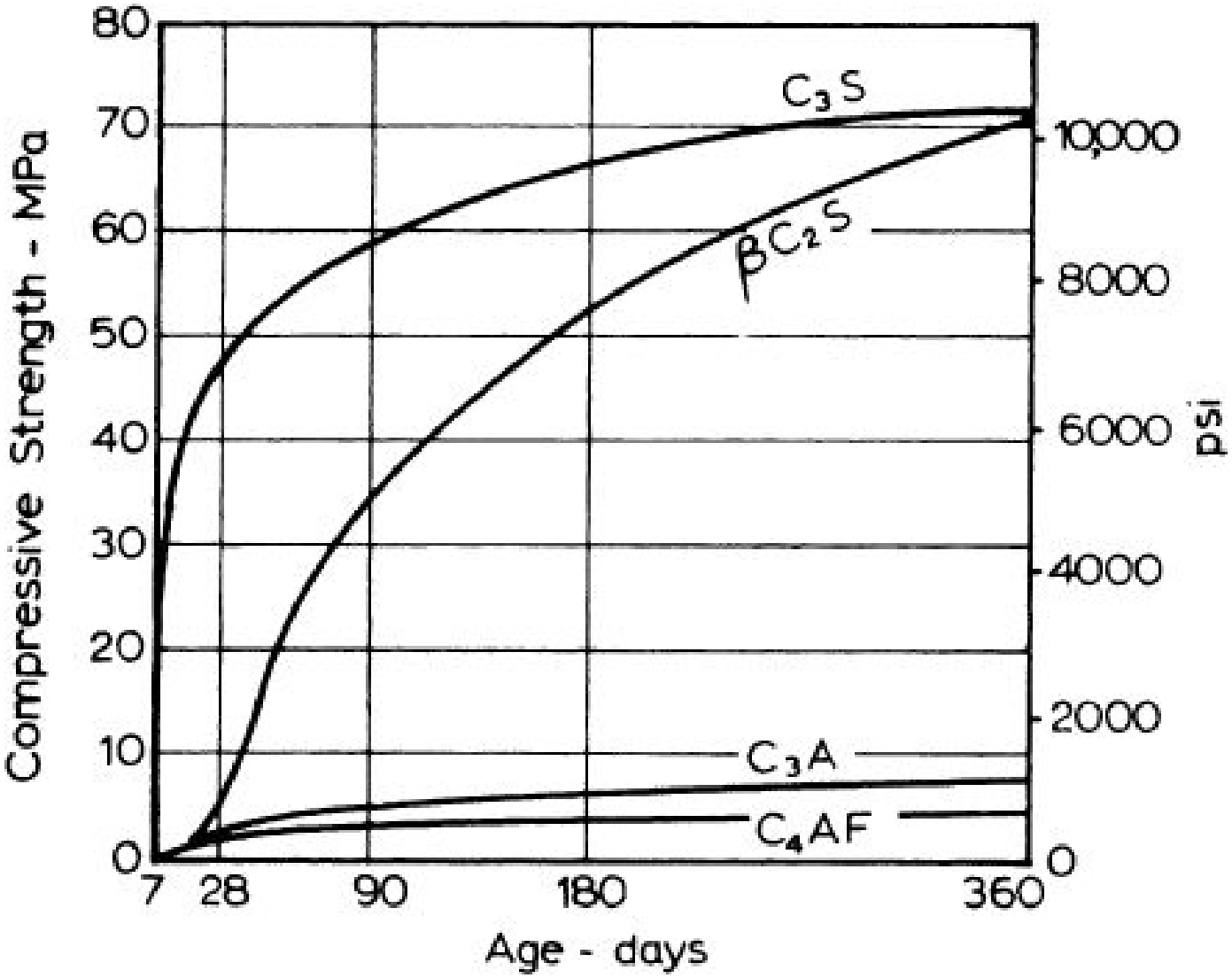


Fig. 1.18. Development of strength of pure compounds according to Bogue[1.2]

However, these relative values of the contribution to strength of the individual compounds in Portland cement have been challenged.[1.87] Tests, using particles with the same size distribution and at a fixed water/solid ratio of 0.45, have shown that, up to at least the age of 1 year, C_2S exhibits a

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lower strength than C_3S. Nevertheless, both silicates are much stronger than C_3A and C_4AF, although the latter compound exhibits a significant strength while C_3A has a negligible strength[1.87] (see Fig. 1.19).

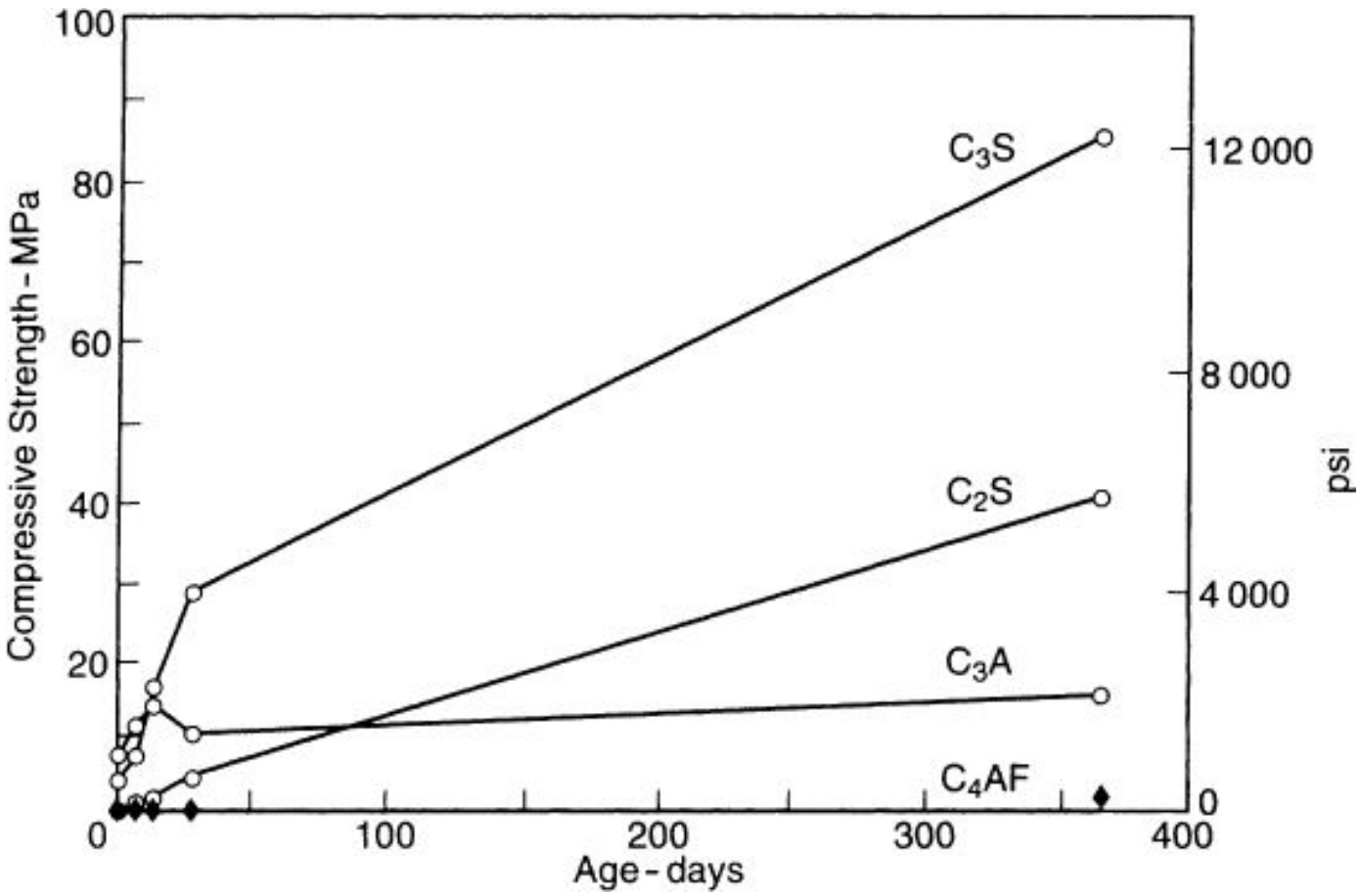


Fig. 1.19. Development of strength of pure compounds according to Beaudoin and Ramachandran (reprinted from ref. 1.87 by kind permission of Elsevier Science Ltd, Kidlington, U.K.)

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As mentioned on p. 14, the calcium silicates appear in commercial cements in 'impure' form. These impurities may strongly affect the rate of reaction and of strength development of the hydrates. For instance, the addition of 1 per cent of Al_2O_3 to pure C_3S increases the early strength of the hydrated paste, as shown in Fig. 1.20. According to Verbeck,[1.55] this increase in strength probably results from activation of the silicate crystal lattice due to introduction of the alumina (or magnesia) into the crystal lattice with resultant activating structural distortions.

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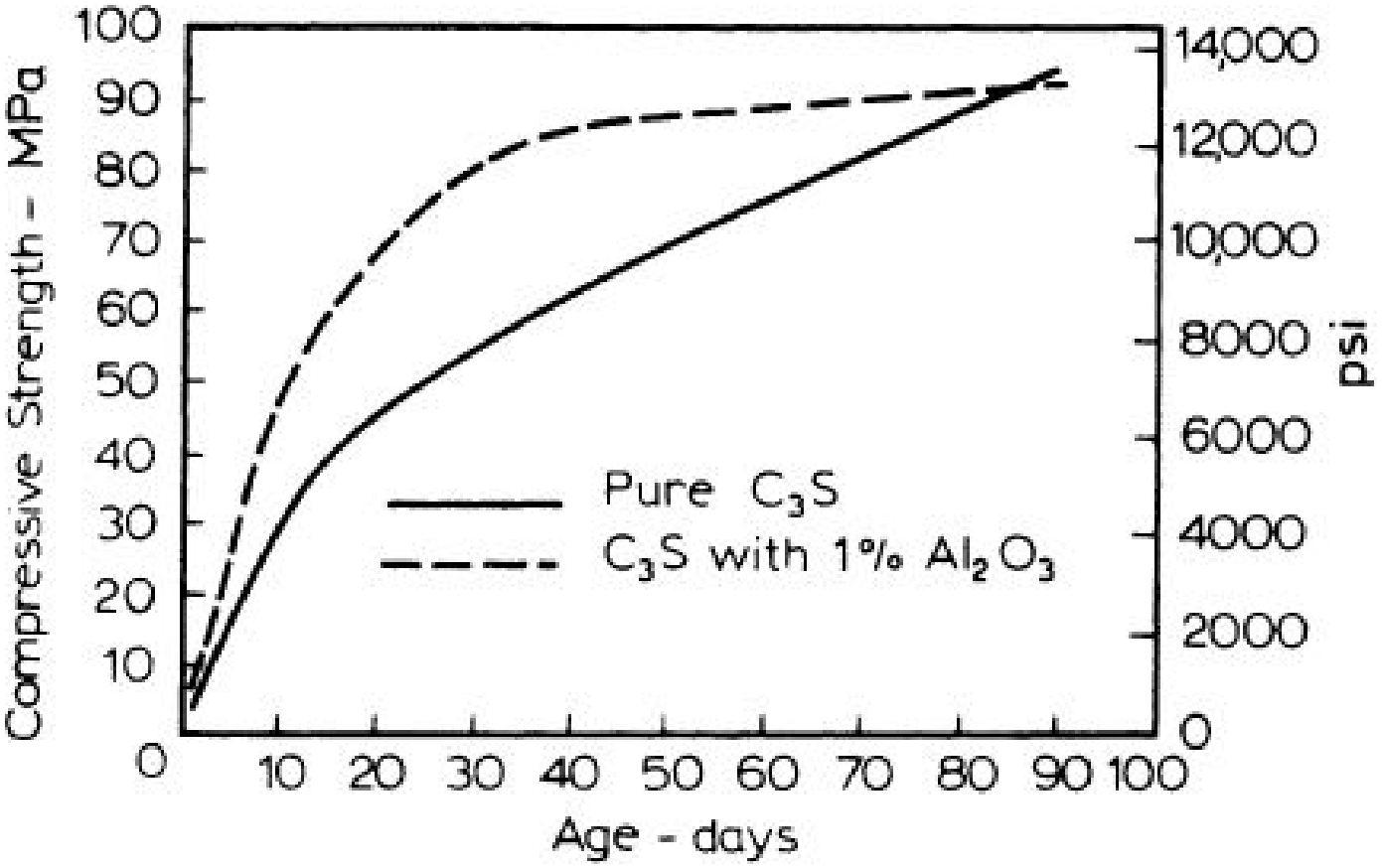


Fig. 1.20. Development of strength of pure C_3S and C_3S with 1 per cent of Al_2O_3[1.55]

The rate of hydration of C_2S is also accelerated by the presence of other compounds in cement but, within the usual range of the C_2S content in modern Portland cements (up to 30 per cent) the effect is not large.

The influence of the other major compounds on the strength development of cement has been established less clearly. C_3A contributes to the

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strength of the cement paste at one to three days, and possibly longer, but causes retrogression at an advanced age, particularly in cements with a high C_3A or (C_3A + C_4AF) content. The role of C_3A is still controversial, but is not important with respect to strength in practice.

The role of C_4AF in the development of strength of cement is also debatable, but there certainly is no appreciable positive contribution. It is likely that colloidal hydrated $CaO.Fe_2O_3$ is deposited on the cement grains, thus delaying the progress of hydration of other compounds.[1.7]

From the knowledge of the contribution to strength of the individual compounds present, it might be thought possible to predict the strength of cement on the basis of its compound composition. This would be in the form of an expression of the type:

$$\text{strength} = a(C_3S) + b(C_2S) + c(C_3A) + d(C_4AF),$$

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where the symbols in brackets represent the percentage by mass of the compound, and *a*, *b*, etc. are constants representing the contribution of 1 per cent of the corresponding compound to the strength of the hydrated cement paste.

The use of such an expression would make it easy to forecast, at the time of manufacture, the strength of cement and would reduce the need for conventional testing. Such a relation does indeed exist in laboratory tests using cements prepared from the pure four main compounds. In practice, however, the contribution of different compounds is not simply additive and has been found to depend on age and on the curing conditions.

All that can be said is that, in general terms, an increase in the C_3S content increases strength up to 28 days;[1.56] Figure 1.21 shows the 7-day strength of standard mortars made with cements of different composition and obtained from different plants.[1.37] The C_2S content has a positive influence on strength at 5 and 10 years only, and C_3A a positive influence up to 7 or 28 days but a

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negative influence later on.[1.56,1.57] The influence of the alkalis is considered on p. 46. Prediction of the effects of compounds other than silicates on strength is unreliable According to Lea,[1.38] these discrepancies may be due to the presence of glass in clinker, discussed more fully in the succeeding section.

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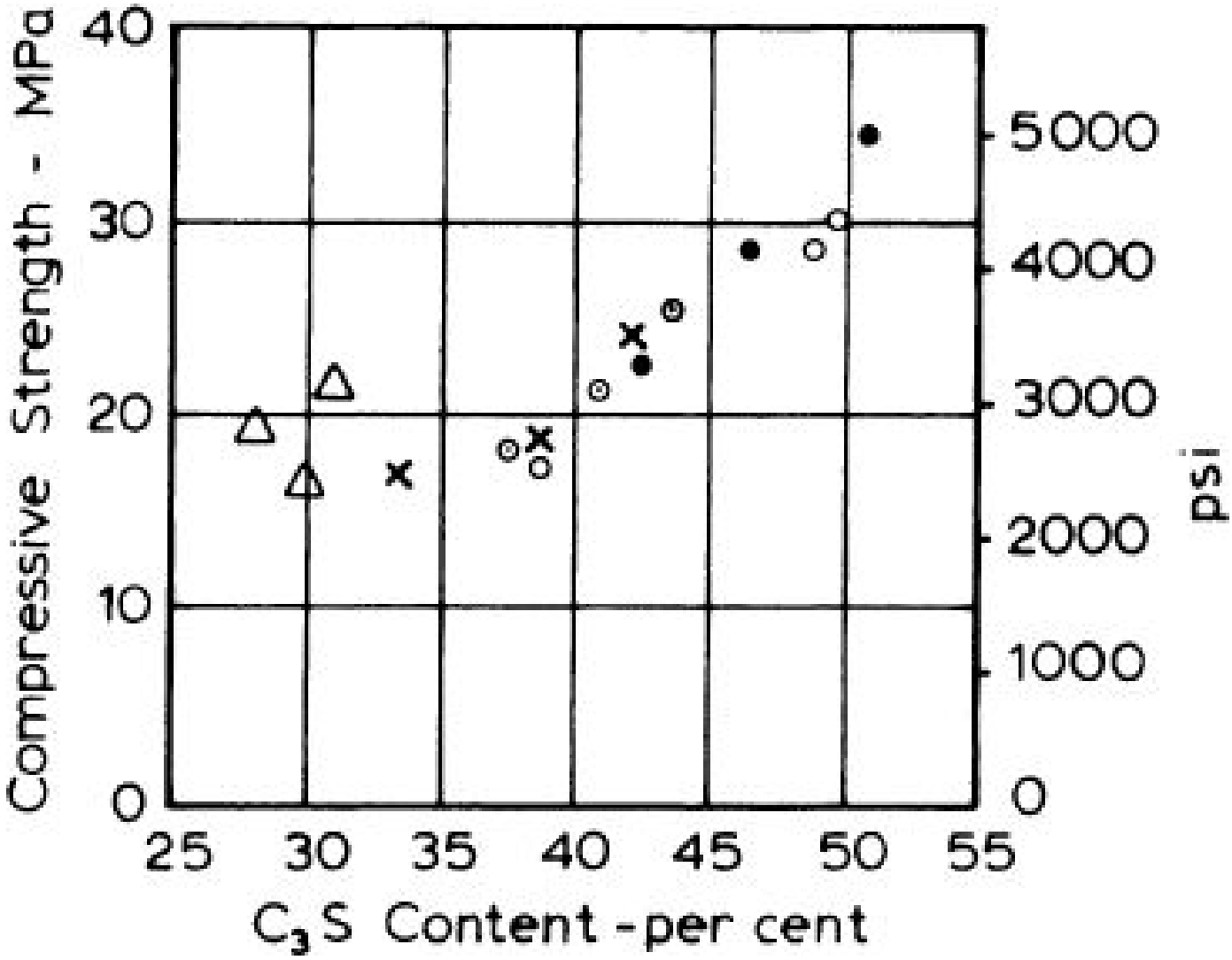


Fig. 1.21. Relation between 7-day strength of cement paste and the C_3S content in cement.[1.37] Each mark represents cement from one plant

An extensive review by Odler[1.79] has shown, moreover, that a generally applicable strength prediction equation for commercial cements is not possible for several reasons. These are: the in-

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teraction between the compounds; the influence of the alkalis and of gypsum; and the influence of the particle size distribution of the cement. The presence of glass, which does not contain all the compounds in the same proportions as the rest of the clinker, but which affects reactivity, as well as the amount of free lime, are also factors varying between cements with nominally the same composition of the four main compounds.

Attempts[1.93] have been made to generate strength prediction equations for mortar on the basis of parameters which include, in addition to the main compound composition, terms for SO_3, CaO, MgO and the water/cement ratio, but the reliability of prediction is marginal.

From the foregoing, we can conclude that the relations between strength and compound composition of Portland cements in general which have been observed are stochastic in nature. Deviations from these relations arise from the fact that they ignore some of the variables involved.[1.14] It can be argued, in any case, that all

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constituents of hydrated Portland cement contribute in some measure to strength in so far as all products of hydration fill space and thus reduce porosity.

Furthermore, there are some indications that the additive behaviour cannot be fully realized. In particular, Powers[1.22] suggested that the same products are formed at all stages of hydration of the cement paste; this follows from the fact that, for a given cement, the surface area of hydrated cement is proportional to the amount of water of hydration, whatever the water/cement ratio and age. Thus the fractional rates of hydration of all compounds in a given cement would be the same. This is probably the case only after the rate of diffusion through the gel coating has become the rate-determining factor, but not at early ages,[1.65] say up to 7 days.[1.49] Confirmation of equal fractional rate of hydration was obtained by Khalil and Ward,[1.70] but we now accept that early hydration of the different compounds proceeds at different rates; later on, the rates become equal.

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There is another factor influencing the rate of hydration: the fact that the composition is not the same at different points in space. This arises from the fact that, for diffusion to take place from the face of the still unhydrated part of the cement grain to the space outside (see p. 13), there must be a difference in ion concentration: the space outside is saturated but that inside is supersaturated. This diffusion varies the rate of hydration.

It is likely, therefore, that neither the suggestion of equal fractional rates of hydration, nor the assumption that each compound hydrates at a rate independent of other compounds, is valid. Indeed, we have to admit that our understanding of the hydration rates is still unsatisfactory.

For instance, the amount of heat of hydration per unit mass of hydrated material has been found to be constant at all ages[1.34] (see Fig. 1.22), thus suggesting that the nature of the products of hydration does not vary with time. It is therefore reasonable to use the assumption of equal fractional rates of hydration within the limited range

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of composition of ordinary and rapid-hardening Portland cements. However, other cements which have a higher C_2S content than ordinary cement or rapid-hardening cement do not conform to this behaviour. Measurements of heat of hydration indicate that C_3S hydrates earlier, and some C_2S is left to hydrate later.

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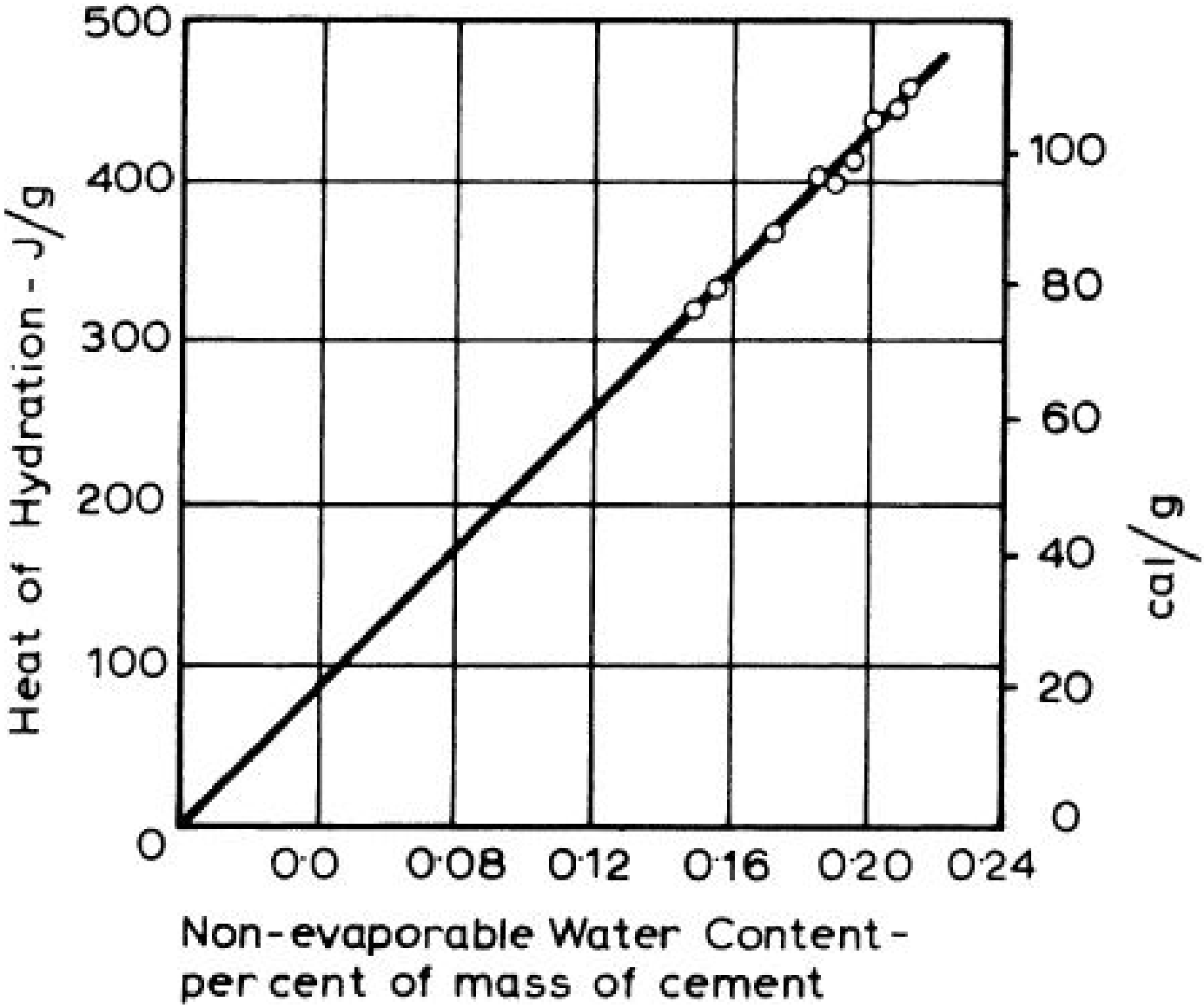


Fig. 1.22. Relation between the heat of hydration and the amount of non-evaporable water for ordinary Portland cement[1.22]

Furthermore, the initial framework of the paste established at the time of setting affects to a large degree the subsequent structure of the products of hydration. This framework influences

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especially the shrinkage and development of strength.[1.14] It is not surprising, therefore, that there is a definite relation between the degree of hydration and strength. Figure 1.23 shows, for instance, an experimental relation between the compressive strength of concrete and the combined water in a cement paste with a water/cement ratio of 0.25.[1.39] These data agree with Powers' observations on the gel/space ratio, according to which the increase in strength of a cement paste is a function of the increase in the relative volume of gel, regardless of age, water/cement ratio, or compound composition of cement. However, the total surface area of the solid phase is related to the compound composition, which does affect the actual value of the ultimate strength.[1.22]

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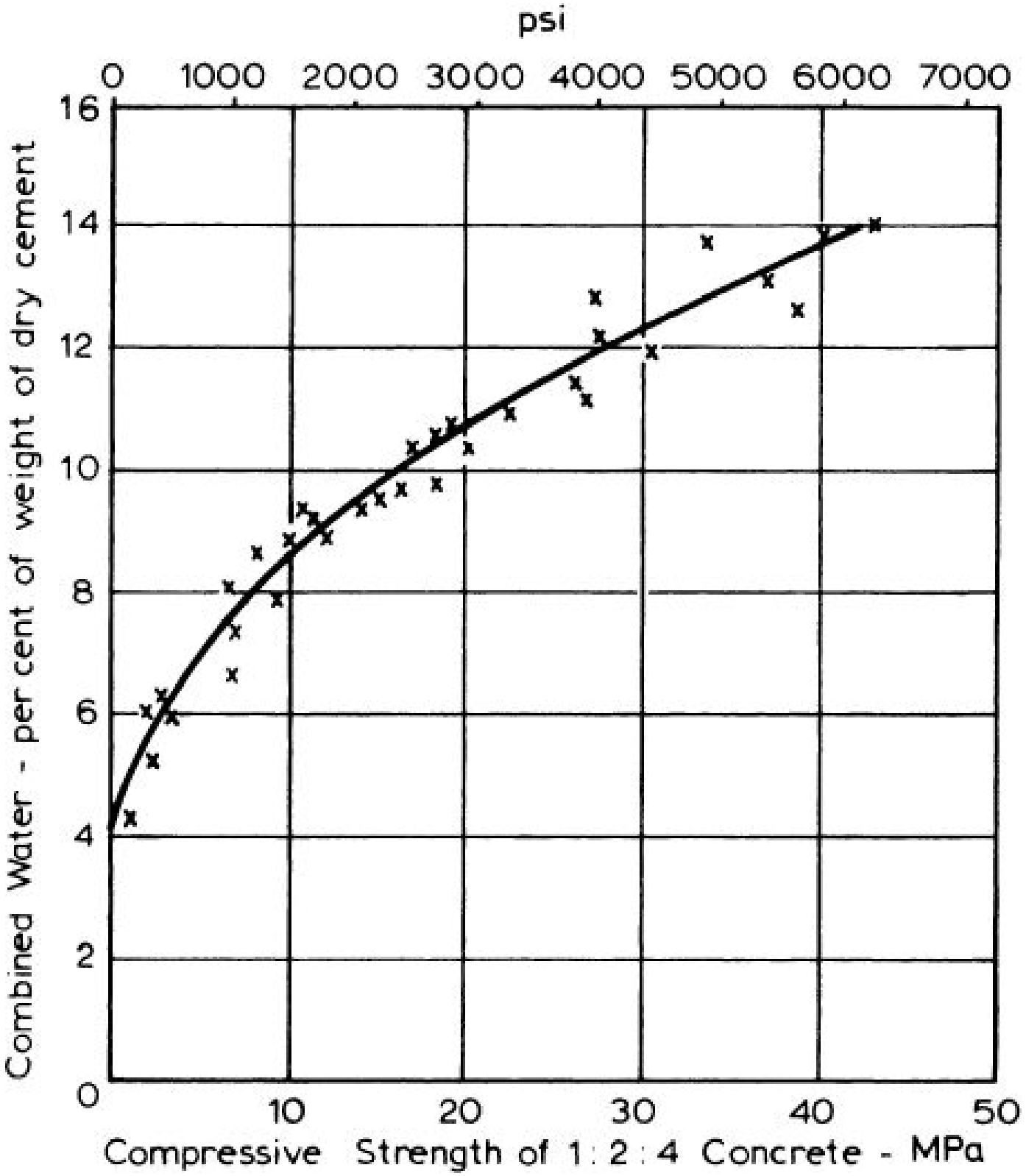


Fig. 1.23. Relation between compressive strength and combined water content[1.1]

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Effects of alkalis

The effects of the minor compounds on the strength of cement paste are complex and not yet fully established. Tests[1.3] on the influence of alkalis have shown that the increase in strength beyond the age of 28 days is strongly affected by the alkali content: the greater the amount of alkali present the lower the gain in strength. This has been confirmed by two statistical evaluations of strength of several hundred commercial cements.[1.56,1.57] The poor gain in strength between 3 and 28 days can be attributed more specifically to water-soluble K_2O present in the cement.[1.58] On the other hand, in the total absence of alkalis, the early strength of cement paste can be abnormally low.[1.58] Accelerated strength tests (see p. 623) have shown that, up to 0.4 per cent of Na_2O, strength increases with an increase in the alkali content[1.75] (Fig. 1.24).

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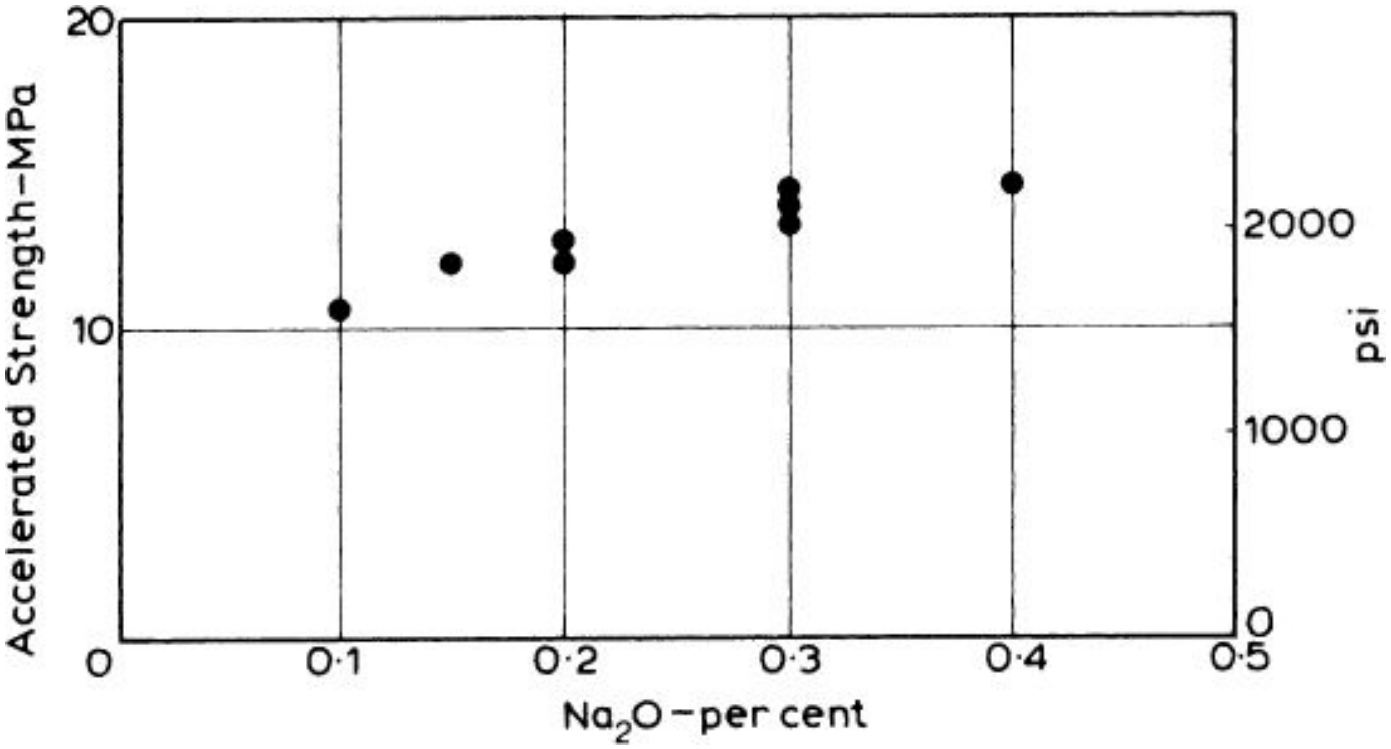


Fig. 1.24. Effect of alkali content on accelerated strength[1.75]

The influence of the alkalis on strength is complicated by the fact that they may be incorporated into the calcium silicate hydrates or may exist as soluble sulfates; their action in the two cases is not the same. K_2O is believed to replace one molecule of CaO in C_2S with a consequent rise in the C_3S content above that calculated.[1.6] However, we can say that, generally, the alkalis increase the early strength development and reduce the long-term strength.[1.79] Osbæck[1.95] con-

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firmed that a higher alkali content in Portland cement increases the early strength and decreases the long-term strength.

The alkalis are known to react with the so-called alkali-reactive aggregates (see p. 144), and cements used under such circumstances often have their alkali content limited to 0.6 per cent (measured as equivalent Na_2O). Such cements are referred to as *low-alkali cements*.

One other consequence of the presence of alkalis in cement should be mentioned. Fresh Portland cement paste has a very high alkalinity (pH above 12.5) but, in a cement with a high alkali content, the pH is even higher. In consequence, human skin is attacked and dermatitis or burns may result; eyes can also be injured. For this reason, the use of protective clothing is essential.

We can see then that the alkalis are an important constituent of cement, but full information on their role is yet to be obtained. It may be noted that the use of pre-heaters in modern dry-process cement plants has led to an increase in

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the alkali content of cement made from given raw materials. The alkali content, therefore, has to be controlled, but limiting the alkali content too severely results in an increased energy consumption.[1,76] A more efficient dust collection also increases the alkali content of the cement when the dust is re-incorporated into the cement because the dust contains a large amount of alkalis; this may be as high as 15 per cent, in which case the dust, or some of it, has to be discarded.

Effects of glass in clinker

It may be recalled that, during the formation of cement clinker in the kiln, some 20 to 30 per cent of the material becomes liquid; on subsequent cooling, crystallization takes place but there is always some material which undercools to glass. In fact, the rate of cooling of clinker greatly affects the properties of cement: if cooling were so slow that full crystallization could be achieved (e.g. in a laboratory), β-C_2S might become converted to γ-C_2S, this conversion being accompanied by ex-

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pansion and powdering, known as dusting. Furthermore, γ-C_2S hydrates too slowly to be a useful cementitious material. However, Al_2O_3, MgO and the alkalis may stabilize β-C_2S, even on very slow cooling in all practical cases.

Another reason why some glass is desirable is the effect of glass on the crystalline phases. Alumina and ferric oxide are completely liquefied at clinkering temperatures, and on cooling produce C_3A and C_4AF. The extent of glass formation would thus affect these compounds to a large degree while the silicates, which are formed mainly as solids, would be relatively unaffected. It may be noted, too, that glass can also hold a large proportion of minor compounds such as the alkalis and MgO; the latter is thus not available for expansive hydration.[1.40] It follows that rapid cooling of high-magnesia clinkers is advantageous. Because the aluminates are attacked by sulfates, their presence in glass would also be an advantage. C_3A and C_4AF in glass form can hydrate to a solid solution of C_3AH_6 and C_3FH_6 which is res-

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istant to sulfates. However, a high glass content adversely affects the grindability of clinker.

On the other hand, there are some advantages of a lower glass content. In some cements, a greater degree of crystallization leads to an increase in the amount of C_3S produced.

It can be seen, then, that a strict control of the rate of cooling of clinker so as to produce a desired degree of crystallization is very important. The range of glass content in commercial clinkers, determined by the heat of solution method, is between 2 and 21 per cent.[1.41] An optical microscope indicates much lower values.

It may be recalled that the Bogue compound composition assumes that the clinker has crystallized completely to yield its equilibrium products, and, as we have seen, the reactivity of glass is different from that of crystals of similar composition.

It can be seen then that the rate of cooling of clinker, as well as, possibly, other characteristics

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of the process of cement manufacture, affect the strength of cement, and this defies attempts to develop an expression for strength as a function of cement composition. Nevertheless, if one process of manufacture is used and the rate of cooling of clinker is kept constant, there is a definite relation between compound composition and strength.

Tests on properties of cement

The manufacture of cement requires stringent control, and a number of tests are performed in the cement plant laboratory to ensure that the cement is of the desired quality and that it conforms to the requirements of the relevant national standards. It is desirable, nonetheless, for the purchaser or for an independent laboratory to make acceptance tests or, more frequently, to examine the properties of a cement to be used for some special purpose. Tests on the chemical composition and fineness are prescribed in European Standards BS EN 196-1 : 2005 and BS EN 196-6 : 2010, respectively; further tests are prescribed by BS

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4550-3-1 : 1978 for ordinary and rapid-hardening Portland cements. Other relevant standards are mentioned when other types of cement are discussed in Chapter 2.

Consistency of standard paste

For the determination of the initial and final setting times and for the Le Chatelier soundness test, neat cement paste of a standard consistency has to be used. It is, therefore, necessary to determine for any given cement the water content of the paste which will produce the desired consistency.

The consistency is measured by the Vicat apparatus shown in Fig. 1.25, using a 10 mm diameter plunger fitted into the needle holder. A trial paste of cement and water is mixed in a prescribed manner and placed in the mould. The plunger is then brought into contact with the top surface of the paste and released. Under the action of its weight the plunger will penetrate the paste, the depth of penetration depending on the consistency. This is considered to be standard, in

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the meaning of BS EN 196-3 : 2005, when the plunger penetrates the paste to a point 6 ± 1 mm from the bottom of the mould. The water content of the standard paste is expressed as a percentage by mass of the dry cement, the usual range of values being between 26 and 33 per cent.

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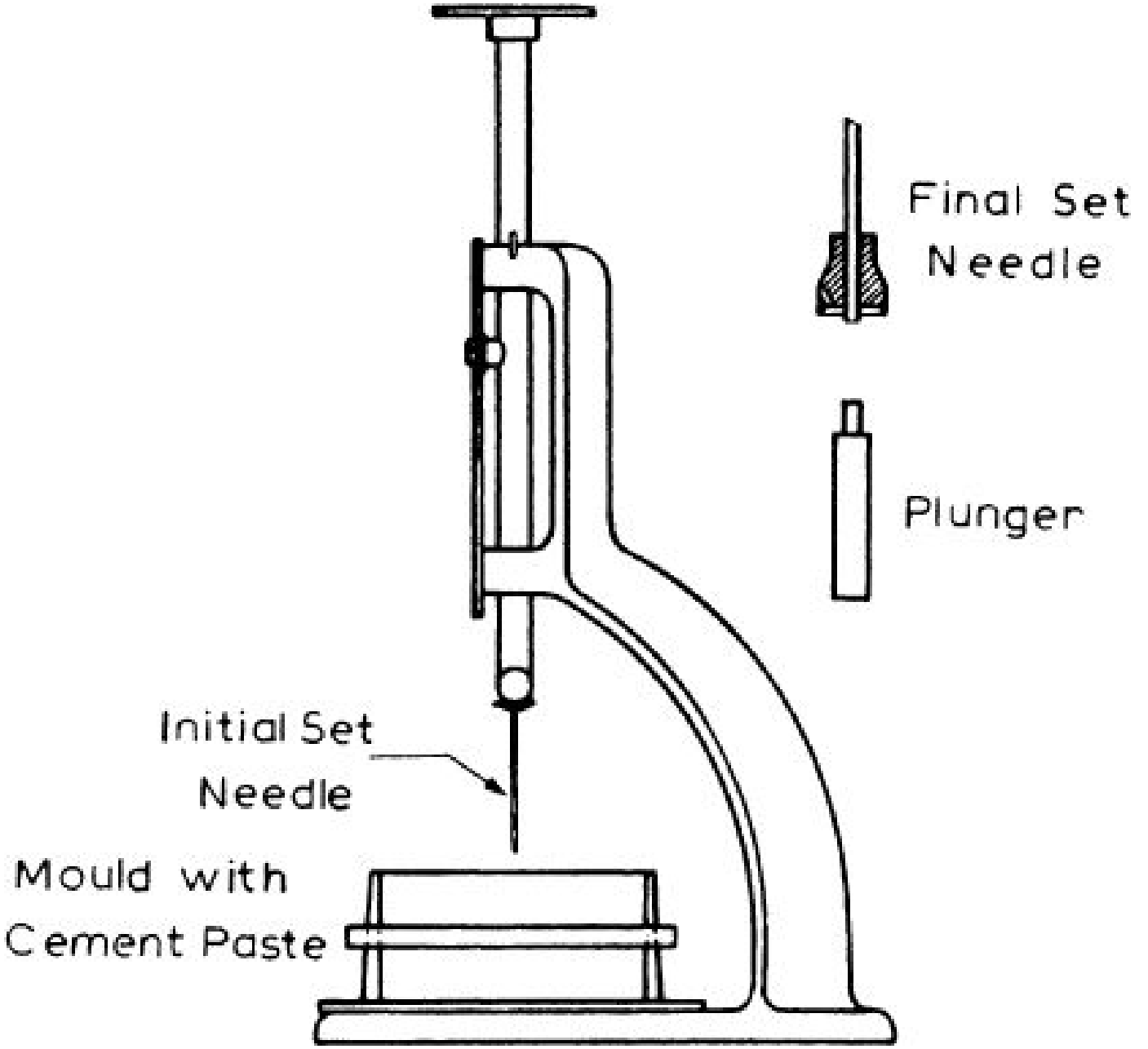


Fig. 1.25. Vicat apparatus

Setting time

The physical processes of setting were discussed on p. 19; here, the actual determination of setting times will be briefly dealt with. The setting times of cement are measured using the Vicat apparatus

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(Fig. 1.25) with different penetrating attachments. The test method is prescribed by BS EN 196-3 : 2005.

For the determination of the initial set, a round needle with a diameter 1.13 ± 0.05 mm is used. This needle, acting under a prescribed weight, is used to penetrate a paste of standard consistency placed in a special mould. When the paste stiffens sufficiently for the needle to penetrate no deeper than to a point 5 ± 1 mm from the bottom, initial set is said to have taken place. Initial set is expressed as the time elapsed since the mixing water was added to the cement. A minimum time of 60 minutes is prescribed by BS EN 197-1 : 2000 for cements with strengths of 42.5 MPa, 75 minutes for cements with a strength of 52.5 MPa and 45 minutes for cements with higher strengths. American Standard ASTM C 150-09 prescribes a minimum time for the initial set of 45 minutes, also using the Vicat apparatus prescribed in ASTM C 191-08. An alternative test

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using Gillmore needles (ASTM C 266-08) gives a higher value of setting time.

The initial setting time of high-alumina cement is prescribed by BS 915-2 : 1972 (1983) as between 2 and 6 hours.

Final set is determined by a similar needle fitted with a metal attachment hollowed out so as to leave a circular cutting edge 5 mm in diameter and set 0.5 mm behind the tip of the needle. Final set is said to have taken place when the needle, gently lowered to the surface of the paste, penetrates it to a depth of 0.5 mm but the circular cutting edge fails to make an impression on the surface of the paste. The final setting is reckoned from the moment when mixing water was added to the cement. Limits on the final setting time no longer appear in the European or ASTM standards.

If the knowledge of final setting time is required, but no test data are available, it may be useful to take advantage of the observation that, for the majority of American commercial ordin-

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ary and rapid-hardening Portland cements at room temperature, the initial and final setting times are approximately related as follows: final setting time (min) = 90 + 1.2 × initial setting time (min).

Because the setting of cement is affected by the temperature and the humidity of the surrounding air, these are specified by BS EN 196-3 : 2005: a temperature of 20 ± 2 °C (68 ± 4 °F) and minimum relative humidity of 65 per cent.

Tests[1.59] have shown that setting of cement paste is accompanied by a change in the ultrasonic pulse velocity through it (cf. p. 633) but it has not been possible to develop an alternative method of measurement of setting time of cement. Attempts at using electrical measurements have also been unsuccessful, mainly because of the influence of admixtures on electrical properties.[1.73]

It should be remembered that the speed of setting and the rapidity of hardening, i.e. of gain of strength, are independent of one another. For

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instance, the prescribed setting times of rapid-hardening cement are no different from those for ordinary Portland cement, although the two cements harden at different rates.

It may be relevant to mention here that the setting time of concrete can also be determined, but this is a different property from the setting time of cement. ASTM Standard C 403-08 lays down the procedure for the former, which uses a Proctor penetration probe applied to mortar sieved from the given concrete. The definition of this setting time is arbitrary as there is no abrupt advent of setting in practice.[1.73] The Russians have attempted to define the setting time of concrete by the minimum resistance between two embedded metal electrodes between which is passed a high-frequency electric current.[1.77]

Soundness

It is essential that cement paste, once it has set, does not undergo a large change in volume. In particular, there must be no appreciable expan-

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sion which, under conditions of restraint, could result in a disruption of the hardened cement paste. Such expansion may take place due to the delayed or slow hydration, or other reaction, of some compounds present in the hardened cement, namely free lime, magnesia, and calcium sulfate.

If the raw materials fed into the kiln contain more lime than can combine with the acidic oxides, or if burning or cooling are unsatisfactory, the excess lime will remain in a free condition. This hard-burnt lime hydrates only very slowly and, because slaked lime occupies a larger volume than the original free calcium oxide, expansion takes place. Cements which exhibit this expansion are described as unsound.

Lime added to cement does not produce unsoundness because it hydrates rapidly before the paste has set. On the other hand, free lime present in clinker is intercrystallized with other compounds and is only partially exposed to water during the time before the paste has set.

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Free lime cannot be determined by chemical analysis of cement because it is not possible to distinguish between unreacted CaO and $Ca(OH)_2$ produced by a partial hydration of the calcium silicates when cement is exposed to the atmosphere. On the other hand, a test on clinker, immediately after it has left the kiln, would show the free lime content as no hydrated cement is then present.

A cement can also be unsound due to the presence of MgO, which reacts with water in a manner similar to CaO. However, only periclase, that is, 'dead-burnt' crystalline MgO, is deleteriously reactive, and MgO present in glass is harmless. Up to about 2 per cent of periclase (by mass of cement) combines with the main cement compounds, but excess periclase generally causes expansion and can lead to slow disruption.

Calcium sulfate is the third compound liable to cause expansion: in this case, calcium sulfoaluminate is formed. It may be recalled that a hydrate of calcium sulfate – gypsum – is added

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to cement clinker in order to prevent flash set, but if gypsum is present in excess of the amount that can react with C_3A during setting, unsoundness in the form of a slow expansion will result. For this reason, standards limit very strictly the amount of gypsum that can be added to clinker; the limits are well on the safe side as far as the danger of unsoundness is concerned.[1.46]

Because unsoundness of cement is not apparent until after a period of months or years, it is essential to test the soundness of cement in an accelerated manner: a test devised by Le Chatelier is prescribed by BS EN 196-3 : 2005. The Le Chatelier apparatus, shown in Fig. 1.26, consists of a small brass cylinder split along its generatrix. Two indicators with pointed ends are attached to the cylinder on either side of the split; in this manner, the widening of the split, caused by the expansion of cement, is greatly magnified and can easily be measured. The cylinder is placed on a glass plate, filled with cement paste of standard consistency, and covered with another glass plate.

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The whole assembly is then placed in a cabinet at 20 ± 1 °C (68 ± 2 °F) and a relative humidity of not less than 98 per cent. At the end of that period, the distance between the indicators is measured, and the mould is immersed in water and gradually brought to the boil in 30 minutes. After boiling for 3 hours, the assembly is taken out and, after cooling, the distance between the indicators is again measured. The increase in this distance represents the expansion of the cement, and for Portland cements is limited to 10 mm by BS EN 197-1 : 2000. If the expansion exceeds this value, a further test is made after the cement has been spread and aerated for 7 days. During this time, some of the lime may hydrate or even carbonate, and a physical breakdown in size may also take place. At the end of the 7-day period, the Le Chatelier test is repeated, and the expansion of aerated cement must not exceed a specified value, which in the past was 5 mm. A cement which fails to satisfy at least one of these tests should not be used.

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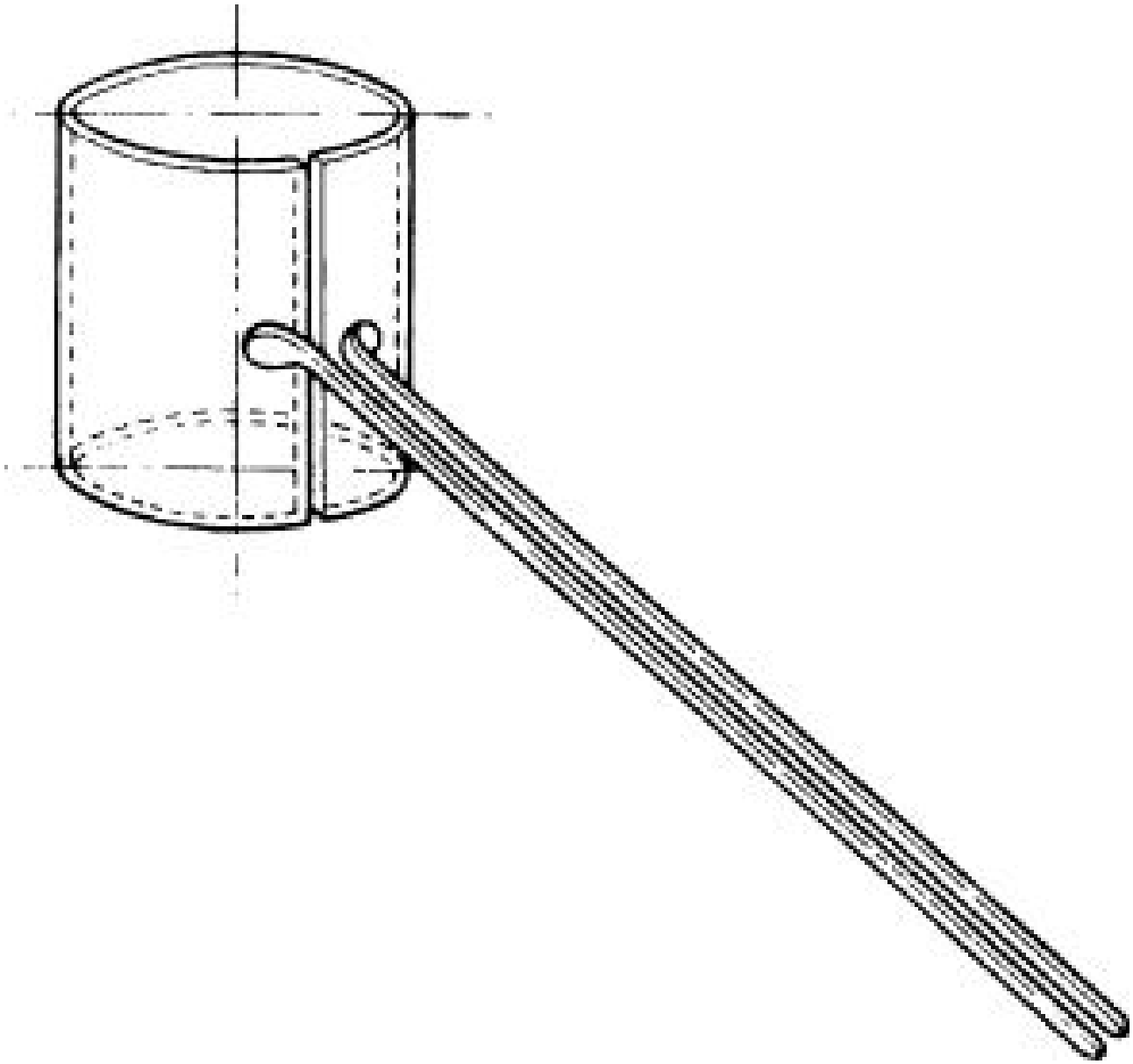


Fig. 1.26. Le Chatelier apparatus

The Le Chatelier test detects unsoundness due to free lime only. Magnesia is rarely present in large quantities in the raw materials from which cement is manufactured in Great Britain, but it is encountered in other countries. An example is India, where low-magnesia limestone occurs only

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to a limited extent. The bulk of cement there has, therefore, a high MgO content but expansion can be significantly reduced by the addition of active siliceous material such as fly ash or finely ground burnt clay.

Because of the importance of avoiding delayed expansion, in the United States, for instance, soundness of cement is checked by the autoclave test, which is sensitive to both free magnesia and free lime. In this test, prescribed by ASTM C 151-09, a neat cement bar, 25 mm (or 1 in.) square in cross-section and with a 250 mm (or 10 in.) gauge length, is cured in humid air for 24 hours. The bar is then placed in an autoclave (a high-pressure steam boiler), which is raised to a temperature of 216 °C (420 °F) (steam pressure of 2 ± 0.07 MPa (295 psi)) in 60 ± 15 min, and maintained at this temperature for 3 hours. The high steam-pressure accelerates the hydration of both magnesia and lime. The expansion of the bar due to autoclaving must not exceed 0.8 per cent.

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The results of the autoclave test are affected not only by the compounds causing expansion, but also by the C_3A content, and by materials blended with the cement,[1.71] and are also subject to other anomalies. The test gives, therefore, no more than a broad indication of the risk of long-term expansion in practice,[1.1] but it is generally overly severe as some MgO may remain inert; the test thus errs on the safe side.[1.86]

No test is available for the detection of unsoundness due to an excess of calcium sulfate, but its content can easily be determined by chemical analysis.

Strength of cement

The mechanical strength of hardened cement is the property of the material that is perhaps most obviously required for structural use. It is not surprising, therefore, that strength tests are prescribed by all specifications for cement.

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The strength of mortar or concrete depends on the cohesion of the cement paste, on its adhesion to the aggregate particles, and to a certain extent on the strength of the aggregate itself. The last factor is not considered at this stage, and is eliminated in tests on the quality of cement by the use of standard aggregates.

Strength tests are not made on a neat cement paste because of difficulties of moulding and testing with a consequent large variability of test results. Cement–sand mortar and, in some cases, concrete of prescribed proportions and made with specified materials under strictly controlled conditions, are used for the purpose of determining the strength of cement.

There are several forms of strength tests: direct tension, direct compression, and flexure. The latter determines in reality the tensile strength in bending because, as is well known, hydrated cement paste is considerably stronger in compression than in tension.

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In the past, the direct tension test on briquettes used to be commonly employed but pure tension is rather difficult to apply so that the results of such a test show a fairly large scatter. Moreover, since structural techniques are designed mainly to exploit the good strength of concrete in compression, the direct tensile strength of cement is of lesser interest than its compressive strength.

Similarly, flexural strength of concrete is generally of lesser interest than its compressive strength, although in pavements the knowledge of the strength of concrete in tension is of importance. In consequence, nowadays, it is the compressive strength of cement that is considered to be crucial, and it is believed that the appropriate test on cement is that on sand–cement mortar.

European Standard BS EN 196-1 : 2005 prescribes a compressive strength test on mortar specimens. The specimens are tested as 40 mm equivalent cubes; they are derived from 40 by 40 by 160 mm prisms, which are first tested in flexure so as to break into halves, or are otherwise broken

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into halves. Thus an optional flexural centre-point test over a span of 100 mm is possible.

The test is performed on mortar of fixed composition, made with a 'CEN standard sand'. (CEN is the acronym of the French name of the European Committee for Standardization.) The sand is natural, siliceous, rounded sand which can be obtained from various sources. Unlike Leighton Buzzard sand (see below), it is not of uniform size but is graded between 80 μm and 1.6 mm. The sand/cement ratio is 3 and the water/cement ratio is 0.50. The mortar is mixed in a cake mixer and compacted on a jolting table with a drop of 15 mm; a vibrating table can also be used, provided it results in similar compaction. The specimens are demoulded after 24 hours in a moist atmosphere and thereafter cured in water at 20 °C.

Because the earlier British or similar tests are used in some countries, it may be appropriate to give a brief description of those tests. Fundamentally, there are two British standard methods

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of testing the compressive strength of cement: one uses mortar, the other concrete.

In the mortar test, a 1 : 3 cement–sand mortar is used. The sand is standard Leighton Buzzard sand, obtained from pits near a town of that name in Bedfordshire, England; the sand is of single size. The mass of water in the mix is 10 per cent of the mass of the dry materials. Expressed as a water/cement ratio, this corresponds to 0.40 by mass. A standard procedure, prescribed by BS 4550-3.4 : 1978, is followed in mixing, and 70.7 mm (2.78 in.) cubes are made using a vibrating table with a frequency of 200 Hz applied for two minutes. The cubes are demoulded after 24 hours and further cured in water until tested in a wet-surface condition.

The vibrated mortar test gives fairly reliable results but it has been suggested that mortar made with single-size aggregate leads to a greater scatter of strength values than would be obtained with concrete made under similar conditions. It can also be argued that we are interested in the per-

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formance of cement in concrete and not in mortar, especially one made with a single-size aggregate and never used in practice. For these reasons, a test on concrete was introduced in British Standards.

In the concrete test, three water/cement ratios can be used, viz. 0.60, 0.55 and 0.45, depending on the type of cement. The amounts of coarse and fine aggregate, which have to come from particular quarries, are specified in BS 4550 : 4 and 5 : 1978. Batches of 100 mm (or 4 in.) cubes are made by hand in a prescribed manner; the temperature and humidity conditions of the mixing room, curing chamber, compression testing room, and the temperature of the water curing tank are specified. Apart from satisfying the minimum strength at specific ages, the strength at later ages has to be higher than at an earlier age, because strength retrogression might be a sign of unsoundness or other faults in the cement. The requirement of strength increase with age applies also to the vibrated mortar cubes but it is not in-

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cluded in BS EN 197-1 : 2000. The characteristic strength is specified. There are three classes: 32.5, 42.5, and 52.5. These are values at 28 days, the latter two with a normal or early stength at 2 days. BS 1881 : 131 : 1998 specifies the test method on concrete cubes with made-up aggregate whose grading is prescribed in BS EN 196-1 : 2005.

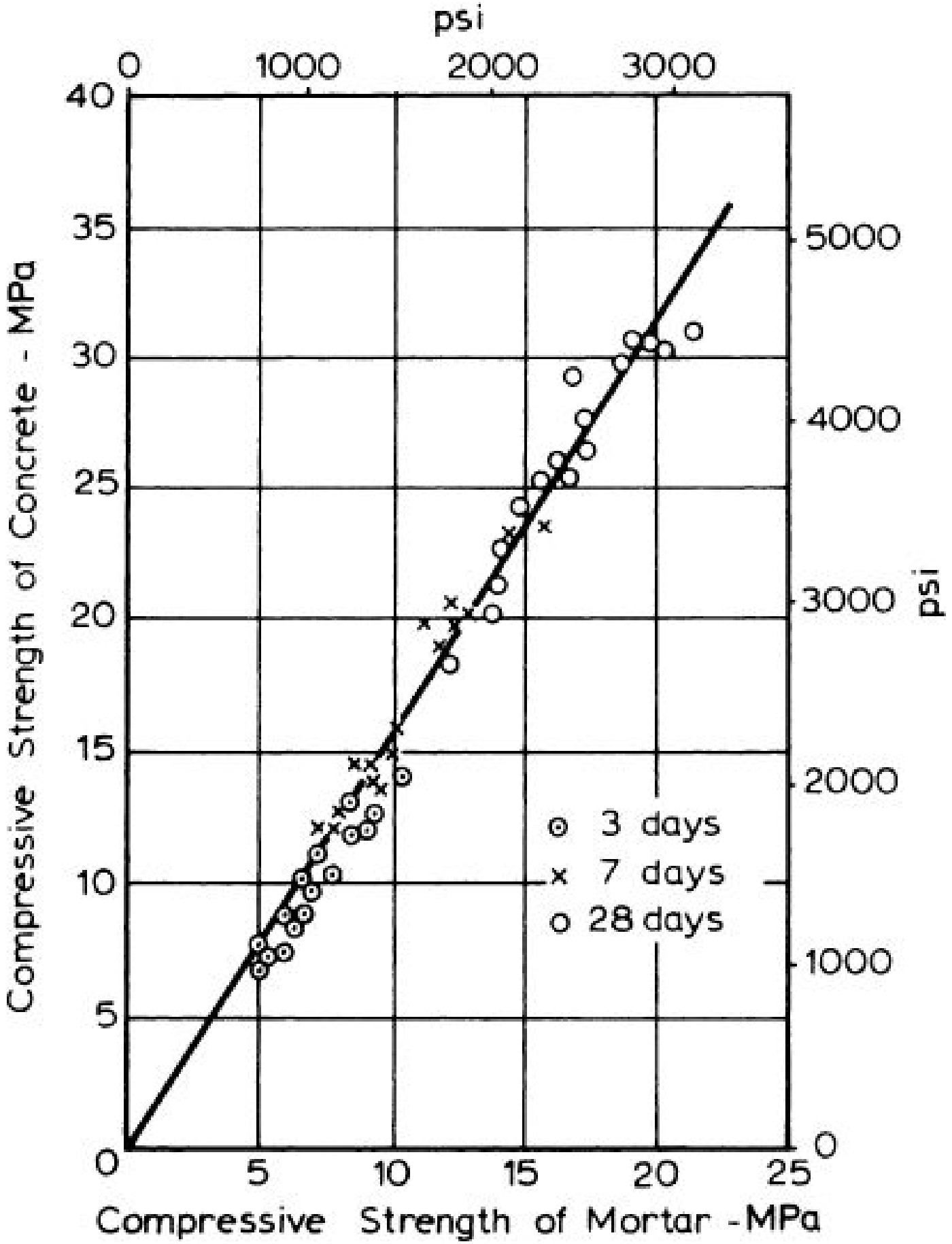
The ASTM method for testing the mean strength of cement is prescribed by ASTM C 109-08 and uses a 1:2.75 mortar made with standard graded sand at a water/cement ratio of 0.485; 50 mm (or 2 in.) cubes are tested.

It may be appropriate to consider the question: should tests on the strength of cement be made on samples of cement paste, mortar or concrete? We have already stated that specimens of neat cement paste are difficult to make. As far as concrete is concerned, it is an appropriate medium for tests but the strength of concrete specimens is influenced by the properties of the aggregate used. It would be difficult, or even impractical, to

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use a standard aggregate for tests on concrete in various parts of the country, let alone in different countries. The use of mortar with a reasonably standard aggregate is a sensible compromise. In any case, all tests are comparative in nature, rather than a direct measure of the compressive strength of hydrated cement paste. Moreover, the influence of cement on the properties of mortar and concrete is qualitatively the same, and the relation between the strengths of corresponding specimens of the two materials is linear. This is shown, for instance, in Fig. 1.27: mortar and concrete of fixed proportions, each with a water/cement ratio of 0.65, were used. The strengths are not the same for the specimens of each pair, at least in part because specimens of different shape and size were used, but there may also be an inherent quantitative difference between the strengths of mortar and of concrete due to the greater amount of entrapped air in mortar.

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Fig. 1.27. Relation between the strengths of concrete and of mortar of the same water/cement ratio[1.37]

Another comparison of interest is that between the strength of concrete made to BS 4550-3.4 : 1978 with a water/cement ratio of 0.60 and the strength of mortar made to BS EN 196-1 : 2005, with a water/cement ratio of 0.50. Not only the water/cement ratio but also other conditions differ between the two tests, so that the resulting strength values differ, too. Harrison[1.88] found the following relation:

$$\log_e(M/C) = 0.28/d + 0.25$$

where C = compressive strength of BS 4550 concrete cubes in MPa

M = compressive strength of BS EN 196 mortar prisms in MPa

and d = age at test in days.

More conveniently, the ratio M/C can be tabulated as follows:

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Age (days)	2	3	7	28
Ratio *M/C*	1.48	1.41	1.34	1.30

In addition to the characteristics of the test specimens, there is an important difference between the significance of the strength values obtained in the European Standard BS EN 196-1 : 2005 and in the old British and most other standards: this concerns the meaning of the term 'minimum strength'. In the traditional standards, the minimum value prescribed had to be exceeded by all test results. On the other hand, in BS EN 196-1 : 2005, the minimum strength represents a characteristic value (see p. 734) such that it is exceeded by 95 per cent of test results; in addition, there is laid down an absolute value below which the specified strength must not fall.

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Chapter 2. Cementitious materials of different types

The previous chapter dealt with the properties of Portland cement in general, and we have seen that cements differing in chemical composition and physical characteristics may exhibit different properties when hydrated. It should thus be possible to select mixtures of raw materials for the production of cements with various desired properties. In fact, several types of Portland cement are available commercially and additional special cements can be produced for specific uses. Several non-Portland cements are also available.

Before describing the various types of Portland cement, a more general discussion of the cementitious materials used in concrete may be useful.

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Categorization of cementitious materials[*]

[*]This section was substantially published in ref. 2.5.

Originally, concrete was made using a mixture of only three materials: cement, aggregate, and water; almost invariably, the cement was Portland cement, as discussed in Chapter 1. Later on, in order to improve some of the properties of concrete, either in the fresh or in the hardened state, very small quantities of chemical products were added into the mix. These *chemical admixtures*, often called simply admixtures, are discussed in Chapter 4.

Later still, other materials, inorganic in nature, were introduced into the concrete mix. The original reasons for using these materials were usually economic: they were cheaper than Portland cement, sometimes because they existed as natural deposits requiring no, or little, processing, sometimes because they were a byproduct or

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waste from industrial processes. A further spur to the incorporation of these ‘supplementary’ materials in the concrete mix was given by the sharp increase in the cost of energy in the 1970s, and we recall that the cost of energy represents a major proportion of the cost of the production of cement (see p. 7).

Yet further encouragement for the use of some of the ‘supplementary’ materials was provided by the ecological concerns about opening of pits and quarries for the raw materials required for the production of Portland cement on the one hand and, on the other, about the means of disposal of the industrial waste materials such as blast-furnace slag, fly ash, or silica fume. Moreover, the manufacture of Portland cement itself is ecologically harmful in that the production of one tonne of cement results in about one tonne of carbon dioxide being discharged into the atmosphere.

It would be incorrect to infer from the previous, historical account that the supplementary

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materials were introduced into concrete solely by the 'push' of their availability. These materials also bestow various desirable properties on concrete, sometimes in the fresh state, but more often in the hardened state. This 'pull', combined with the 'push', has resulted in a situation such that, in very many countries, a high proportion of concrete contains one or more of these supplementary materials. It is therefore inappropriate to consider them, as was sometimes done in the past, as cement replacement materials or as 'extenders'.

If, as just stated, the materials which we have hitherto described as supplementary are, in their own right, proper components of the cementitious materials used in making concrete, then a new terminology has to be sought. No single terminology has been agreed or accepted on a world-wide basis, and it may be useful briefly to discuss the nomenclature used in various publications.

In so far as concrete is concerned, the cementitious material always contains Portland cement of the traditional variety, that is 'pure' Portland

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cement. Therefore, when other materials are also included, it is possible to refer to the ensemble of the cementitious materials used as *Portland composite cements*. This is a logical term, and so is the term *blended Portland cements*.

The European approach of BS EN 197-1 : 2000 is to use the term *CEM cement*, which requires the presence of the Portland cement component; thus CEM cement by implication excludes high-alumina cement. The name CEM cement is not thought to be explicit or of general appeal. There are 27 common cements in five categories, CEM I to V.

The current American approach is given in ASTM C 1157-10, which covers blended hydraulic cements for both general and special applications. A blended hydraulic cement is defined as follows: "A hydraulic cement consisting of two or more inorganic constituents which contribute to the strength-gaining properties of the cement, with or without other constituents, processing additions and functional additions."

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This terminology is sound except that the term ‘inorganic constituent’ is difficult to relate to the actual materials incorporated in concrete, typically natural or industrially produced pozzolana, fly ash, silica fume, or ground granulated blast-furnace slag. Moreover, emphasis on the term ‘hydraulic’ may conjure up a wrong image in the eyes of the general users of cement. Furthermore, the ASTM terminology is not used by the American Concrete Institute.

The preceding, rather lengthy, discussion explains the difficulty of classifying and categorizing the different materials involved. The situation is not helped by a lack of international nomenclature. Indeed, more than one approach is possible but the difficulty is exacerbated by the fact that some of the divisions are not mutually exclusive.

In view of the international use of this book, it has been decided to use the following terminology.

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A cement consisting of Portland cement with no more than 5 per cent of another inorganic material will be referred to as Portland cement. We should recall that prior to 1991, Portland cements were generally expected to be ‘pure’, that is, not to contain minor additions other than gypsum or grinding aids.

A cement consisting of Portland cement and one or more appropriate inorganic materials will be called *blended cement*. This term is close to that used in ASTM C 1157-10. Like ASTM, we use the term ‘blended’ to include both the results of blending the separate powders and of intergrinding the parent materials, e.g. Portland cement clinker and ground granulated blastfurnace slag (see p. 79).

There is some difficulty in choosing the term for the components which make up a blended cement. The terms ‘constituent’ and ‘component’ run the risk of confusion with the chemical compounds in Portland cement. What all the materials with which we are concerned have in common

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is that, in the words of ASTM C 1157-10, they "contribute to the strength-gaining properties of the cement". In actual fact, some of these materials are cementitious in themselves, some have latent cementitious properties, yet others contribute to the strength of concrete primarily through their physical behaviour. It is proposed, therefore, to refer to all these materials as *cementitious materials*. Purists might criticize this choice, but it has the important merits of simplicity and clarity.

The individual cementitious materials will be discussed later in this chapter but, for convenience, Table 2.1 describes their relevant properties; it can be seen that there are no clear-cut divisions with respect to hydraulic, that is, truly cementitious, properties.

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Table 2.1. Cementitious Nature of Materials in Blended Cements

Material	*Cementitious nature*
Portland cement clinker	Fully cementitious (hydraulic)
Ground granulated blastfurnace slag (ggbs)	Latent hydraulic, sometimes hydraulic
Natural pozzolana (Class N)	Latent hydraulic with Portland cement
Siliceous fly ash (pfa) (Class F)	Latent hydraulic with Portland cement
High-lime fly ash (Class C)	Latent hydraulic with Portland cement but also slightly hydraulic
Silica fume	Latent hydraulic with Portland cement but largely physical in action
Calcareous filler	Physical in action but with slight latent hydraulic action with Portland cement
Other fillers	Chemically inert; only physical in action

As already mentioned, all the cementitious materials, as just defined, have one property in common: they are at least as fine as the particles of Portland cement, and sometimes much finer. Their other features, however, are diverse. This applies to their origin, their chemical composition, and their physical characteristics such as surface texture or specific gravity.

There are several ways of preparing a blended cement. One way is to integrind the other cementitious materials with the cement clinker so that an integral blended cement is produced. The

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second way is for two or, more rarely, three materials in their final form to be truly blended. Alternatively, Portland cement and one or more cementitious materials can be separately, but simultaneously or nearly so, fed into the concrete mixer.

Furthermore, the relative amounts of Portland cement and of the other cementitious materials in the concrete mix vary widely: sometimes the proportion of the other cementitious materials is low, in other mixes they constitute a significant proportion, even a major part, of the blended cement.

Thus, in this book, the term ‘cementitious material’ will be used for all the powder material, other than that which forms the finest particles of aggregate, provided that one of the powder materials is cement. With very few exceptions considered on p. 82 and 91, the cement is Portland cement. Thus, the cementitious material may be Portland cement alone or it may comprise Portland cement and one or more other cementitious materials.

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A given cementitious material may be hydraulic in nature, that is, it may undergo hydration on its own and contribute to the strength of the concrete. Alternatively, it may have latent hydraulic properties: that is, it may exhibit hydraulic activity only in consequence of chemical reaction with some other compounds such as the products of hydration of Portland cement which co-exists in the mixture. Yet a third possibility is for the cementitious material to be largely chemically inert but to have a catalytic effect on the hydration of other materials, e.g. by fostering nucleation and densifying the cement paste, or to have a physical effect on the properties of the fresh concrete. Materials in this category are called *fillers*. Fillers will be discussed on p. 88.

For the benefit of American readers, it should be mentioned that the term “mineral admixtures”, used by the American Concrete Institute to describe the non-hydraulic supplementary materials, will not be used in this book. The word “admixture” conjures up a minor component,

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something added to the 'main mix', and yet, as already mentioned, some of the 'supplementary' materials are present in large proportions.

The different categories of cementitious materials will be discussed later in this chapter. Their more specific uses and their detailed influence on the properties of concrete will be considered, as appropriate, throughout the book.

Different cements

In the preceding section, we discussed cementitious materials on the basis of their broad composition and rational classification. For practical purposes of selection of an appropriate Portland cement or a blended cement, it is useful to consider a classification based on the relevant physical or chemical property, such as a rapid gain of strength, low rate of evolution of the heat of hydration, or resistance to sulfate attack.

In order to facilitate the discussion, a list of different Portland cements, with or without other cementitious materials, together with the Amer-

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ican description according to ASTM Standards C 150-09 or C 595-10, where available, is given in Table 2.2. The former ASTM composition limits for some of these cements have already been listed (Table 1.9), and typical, historical values of compound composition are given in Table 2.3.[2.34]

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Table 2.2. Main Types of Portland Cement

Traditional British description	*ASTM description*
Ordinary Portland	Type I
Rapid-hardening Portland	Type III
Extra rapid-hardening Portland	
Ultra high early strength Portland	Regulated set*
Low heat Portland	Type IV
Modified cement	Type II
Sulfate-resisting Portland	Type V
Portland blastfurnace	Type IS Type I (SM)
White Portland	—
Portland–pozzolana	Type IP Type I (PM)
Slag cement	Type S

Note: All American cements except Types IV and V are also available with an interground air-entraining agent, and are then denoted by letter A, e.g. Type IA.
*Not an ASTM description.

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Table 2.3. Typical Values of Compound Composition of Portland Cements of Different Types[2.34]

Cement	Value	Compound composition, per cent								Number of samples
		C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	Free CaO	MgO	Ignition Loss	
Type I	Max.	67	31	14	12	3.4	1.5	3.8	2.3	
	Min.	42	8	5	6	2.6	0.0	0.7	0.6	
	Mean	49	25	12	8	2.9	0.8	2.4	1.2	21
Type II	Max.	55	39	8	16	3.4	1.8	4.4	2.0	
	Min.	37	19	4	6	2.1	0.1	1.5	0.5	
	Mean	46	29	6	12	2.8	0.6	3.0	1.0	28
Type III	Max.	70	38	17	10	4.6	4.2	4.8	2.7	
	Min.	34	0	7	6	2.2	0.1	1.0	1.1	
	Mean	56	15	12	8	3.9	1.3	2.6	1.9	5
Type IV	Max.	44	57	7	18	3.5	0.9	4.1	1.9	
	Min.	21	34	3	6	2.6	0.0	1.0	0.6	
	Mean	30	46	5	13	2.9	0.3	2.7	1.0	16
Type V	Max.	54	49	5	15	3.9	0.6	2.3	1.2	
	Min.	35	24	1	6	2.4	0.1	0.7	0.8	22
	Mean	43	36	4	12	2.7	0.4	1.6	1.0	

The unification of standards within the European Union, including also some other European countries, has led to the first common standard for cement published by the European Committee for Standardization, namely, BS EN 197-1 : 2000 “Cement–composition, specifications and conformity criteria for common ce-

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ments". A simplified version of the classification used in that standard is given in Table 2.4.

Table 2.4. Classification of main cements according to European Standard BS EN 197-1 : 2000

*Type**	*Designation*	*Mass as percentage of mass of cementitious material†*			
		Portland cement clinker	*Pozzolana‡ or fly ash*	*Silica fume*	*ggbs§*
I	Portland	95–100	—	—	—
II/A	Portland slag	80–94	—	—	6–20
II/B		65–79	—	—	21–35
II/A	Portland pozzolana or Portland fly ash	80–94	6–20	—	—
II/B		65–79	21–35	—	—
II/A	Portland silica fume	90–94	—	6–10	—
II/A	Portland composite	80–94	← 6–20 →		
II/B		65–79	← 21–35 →		
III/A	Blastfurnace	35–64	—	—	36–65
III/B		20–34	—	—	66–80
III/C		5–19	—	—	81–95
IV/A	Pozzolanic	65–89	← 11–35 →		—
IV/B		45–64	← 36–55 →		—

*An additional letter describes the nature of the second cementitious material.
†Exclusive of filler permitted up to 5 per cent.
‡Other than fly ash or silica fume.
§Ground granulated blastfurnace slag.

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Many of the cements have been developed to ensure good durability of concrete under a variety of conditions. It has not been possible, however, to find in the composition of cement a complete answer to the problem of durability of concrete: the principal mechanical properties of hardened concrete, such as strength, shrinkage, permeability, resistance to weathering, and creep, are affected also by factors other than cement composition, although this determines to a large degree the rate of gain of strength.[2.2] Figure 2.1 shows the rate of development of strength of concretes made with cements of different types: while the rates vary considerably, there is little difference in the 90-day strength of cements of all types;[2.1] in some cases, e.g. Fig. 2.2, the differences are greater.[2.4] The general tendency is for the cements with a low rate of hardening to have a slightly higher ultimate strength. For instance, Fig. 2.1 shows that Type IV cement has the lowest strength at 28 days but develops the second highest strength at the age of 5 years. A compar-

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ison of Figs 2.1 and 2.2 illustrates the fact that differences between cement types are not readily quantified.

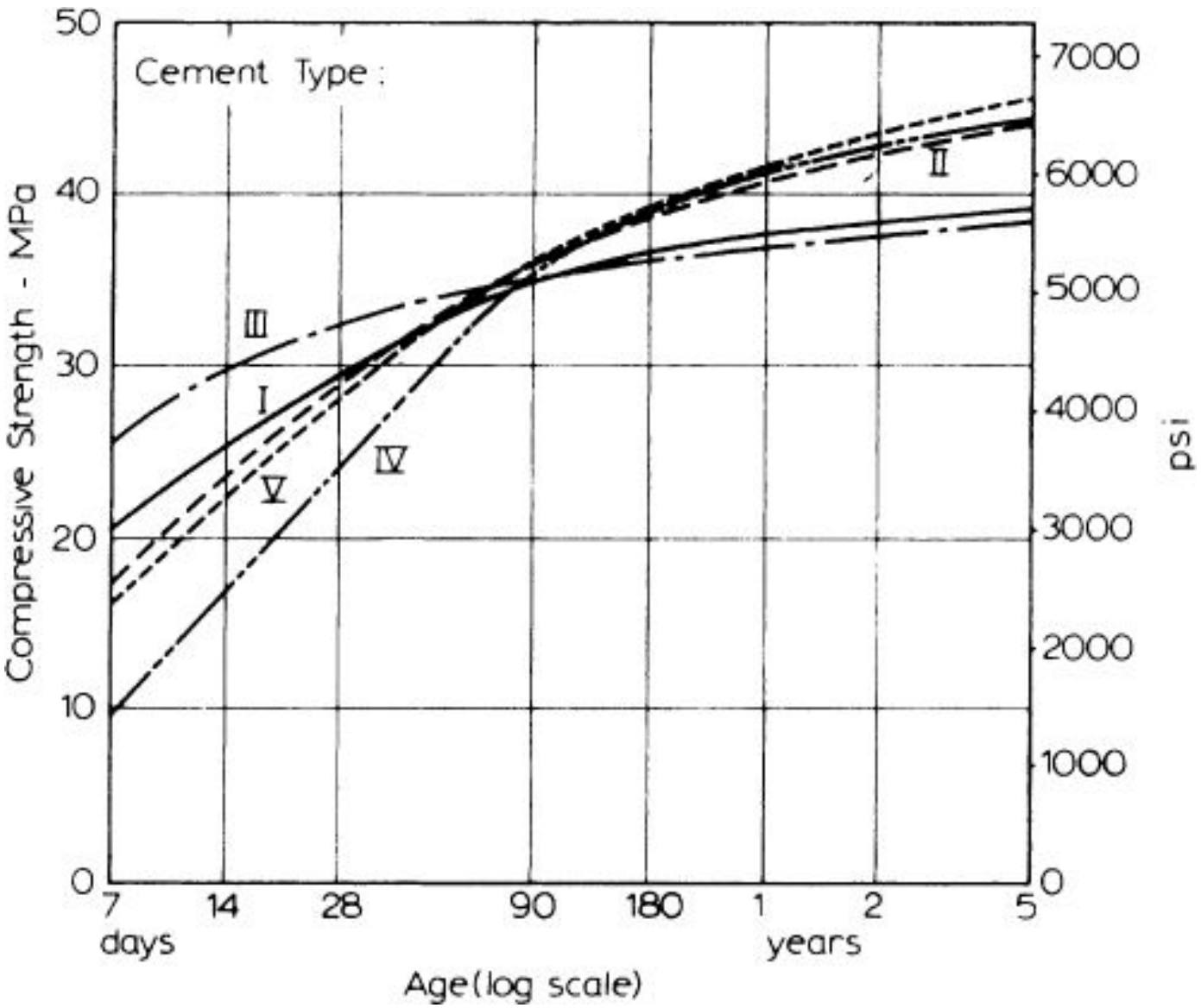


Fig. 2.1. Strength development of concretes containing 335 kg of cement per cubic metre (565 lb/yd^3) and made with cements of different types[2.1]

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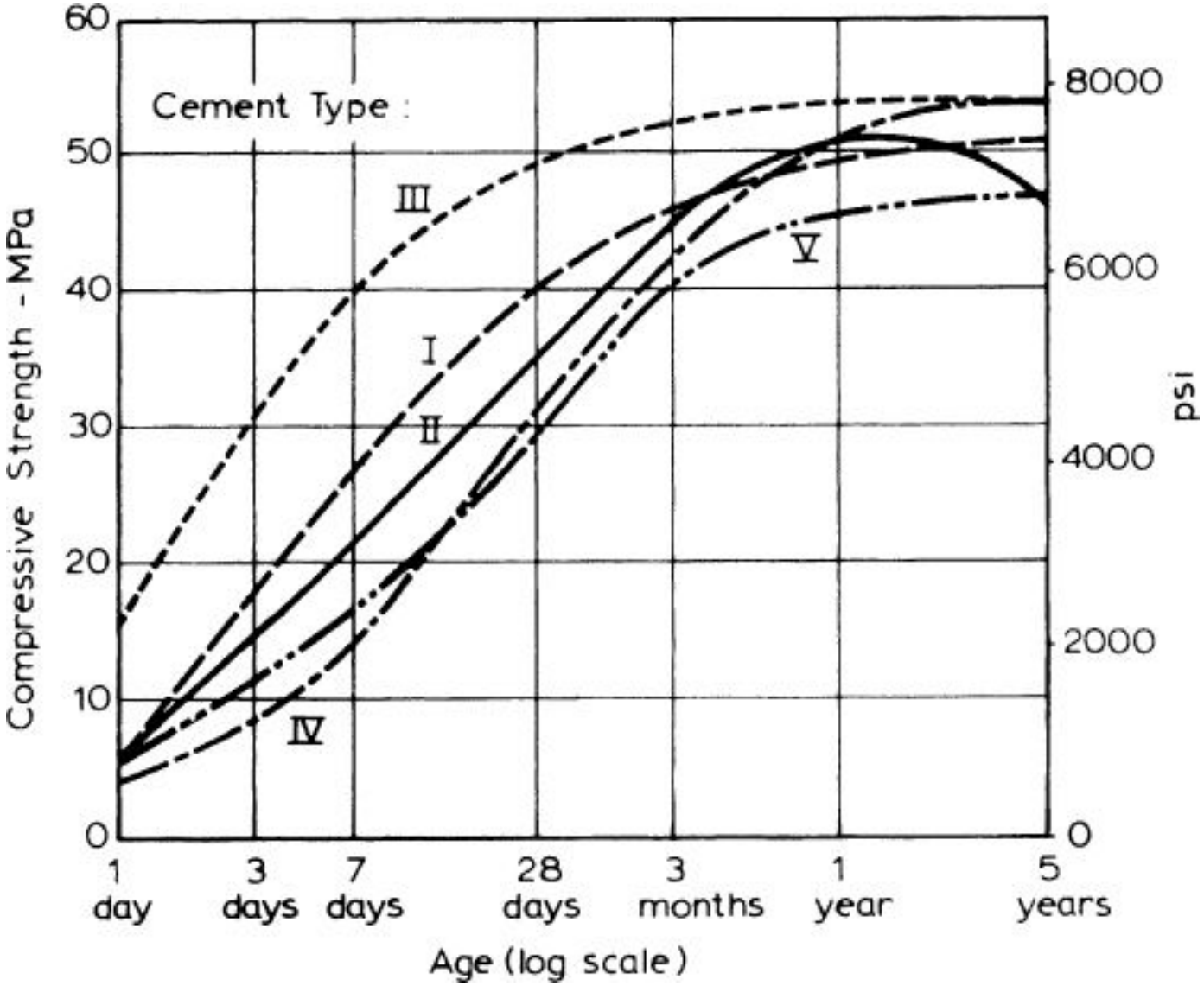


Fig. 2.2. Strength development of concretes with a water/cement ratio of 0.49 made with cements of different types[2.4]

Still referring to Fig. 2.2, we should note that the retrogression of strength of concrete made with Type II cement is not characteristic of this type of cement. The pattern of low early and high late strength agrees with the influence of the ulti-

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tial framework of hardened cement on the ultimate development of strength: the more slowly the framework is established the denser the gel and the higher the ultimate strength. Nevertheless, significant differences in the important physical properties of cements of different types are found only in the earlier stages of hydration:[2,3] in well-hydrated pastes the differences are only minor.

The division of cements into different types is necessarily no more than a broad functional classification, and there may sometimes be wide differences between cements of nominally the same type. On the other hand, there are often no sharp discontinuities in the properties of different types of cement, and many cements can be classified as more than one type.

Obtaining some special property of cement may lead to undesirable features in another respect. For this reason, a balance of requirements may be necessary, and the economic aspect of manufacture must also be considered. Type II ce-

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ment is an example of a ‘compromise’ all-round cement.

The methods of manufacture have improved steadily over the years, and there has been a continual development of cements to serve different purposes with a corresponding change in specifications. On the other hand, some of the changes proved to be disadvantageous when they were not accompanied by a change in concrete practice; this is discussed on p. 335.

Ordinary Portland cement

This is by far the most common cement in use: about 90 per cent of all cement used in the United States (total production in 2008 of about 73 million tonnes per annum) and a like percentage of the ordinary type in the United Kingdom (total production of 12 million tonnes per annum in 2005). It may be interesting to note that in 2007 the annual consumption of cement in the United Kingdom was equivalent to nearly 250 kg per head of population: the corresponding figure for the Un-

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ited States was 360 kg. For every man, woman and child in the world, the consumption in 2007 was 420 kg per annum, which is second only to the consumption of water. The biggest change occurred in China, where the increase between 1995 and 2004 was 90%, and the current consumption represents over 50% of the world production. The global production is forecast to reach 3.5 billion tonnes in 2013. With the considerable increase in the use of fly ash as a cementitious material, the quantity of concrete used is no longer proportional to consumption of Portland cement.

Ordinary Portland (Type I) cement is admirably suitable for use in general concrete construction when there is no exposure to sulfates in the soil or groundwater. The specification for this cement is given in European Standard BS EN 197-1 : 2000. In keeping with the modern trend towards performance-oriented specifications, little is laid down about the chemical composition of the cement, either in terms of compounds or of oxides. Indeed, the standard requires only that it is

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made from 95 to 100 per cent of Portland cement clinker and 0 to 5 per cent of minor additional constituents, all by mass, the percentages being those of the total mass except calcium sulfate and manufacturing additives such as grinding aids.

The limitation on the clinker composition is that not less than two-thirds of its mass consists of C_3S and C_2S taken together, and that the ratio of CaO to SiO_2, also by mass, be not less than 2.0. The content of MgO is limited to a maximum of 5.0 per cent.

The minor additional constituents, referred to above, are one or more of the other cementitious materials (see p. 64) or a *filler*. A filler is defined as any natural or inorganic material other than a cementitious material. An example of a filler is a calcareous material which, due to its particle distribution, improves the physical properties of the cement, for example, workability or water retention. Fillers are discussed more fully on p. 88.

Thus, BS EN 197-1 : 2000 contains no detailed requirements about the proportion of the

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various oxides in the clinker which were included in the previous versions of British Standards. As some of those requirements are still used in many countries, it is useful to mention the lime saturation factor which is to be not greater than 1.02 and not less than 0.66. For cement, the factor is defined as:

$$\frac{1.0(CaO) - 0.7(SO_3)}{2.8(SiO_2) + 1.2(Al_2O_3) + 0.65(Fe_2O_3)}$$

where each term in brackets denotes the percentage by mass of the given compound present in the cement.

The upper limit of the lime saturation factor ensures that the amount of lime is not so high as to result in free lime appearing at the clinkering temperature in equilibrium with the liquid present. The unsoundness of cement caused by free lime was discussed in the previous chapter, and is indeed controlled by the Le Chatelier test. Too low a lime saturation factor would make the burning in the kiln difficult and the proportion of

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C_3S in the clinker would be too low for the development of early strength.

Methods of chemical analysis of cement are prescribed in European Standard BS EN 196-2 : 2005.

As British Standard BS 12 : 1996 (withdrawn 2000) is still in use in some countries, it should be mentioned that it limits the expansion in the Le Chatelier test, determined in accordance with BS EN 196-3 : 2005 to not more than 10 mm. Further requirements of BS 12 : 1996 and BS EN 197-1 : 2000, which replaced BS 12 : 1996, are: the SO_3 content of not more than 3.5 or 4.0 per cent; and chloride content of not more than 0.10 per cent. Limits on the insoluble residue and the loss on ignition are also given. ASTM C 150-09 does not specify any limits on the SO_3 content.

British Standard BS 12 : 1996 classifies Portland cements according to their compressive strength, as shown in Table 2.5. The 28-day minimum strength in MPa gives the name of the class: 32.5, 42.5, 52.5, and 62.5. The 28-day

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strengths of the two lower classes are prescribed by a range, that is, each class of cement has a maximum value of strength as well as a minimum. Moreover, cements of class 32.5 and 42.5 are each subdivided into two subclasses, one with an ordinary early strength, the other with a high early strength. The two subclasses with a high early strength, denoted by the letter R, are rapid-hardening cements, and they will be considered in the next section.

Table 2.5. Compressive Strength Requirements of Cement According to BS 12 : 1996

Class	*Minimum strength, MPa at the age of:*			*Maximum strength, MPa at the age of 28 days*
	2 days	*7 days*	*28 days*	
32.5 N	—	16	32.5	52.5
32.5 R	10	—		
42.5 N	10	—	42.5	62.5
42.5 R	20	—		
52.5 N	20	—	52.5	—
62.5 N	20	—	62.5	—

The advantage of prescribing the class 32.5 and 42.5 cements by a range of strength of 20

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MPa is that, during construction, wide variations in strength, especially downwards, are avoided. Furthermore, and perhaps more importantly, an excessively high strength at the age of 28 days would allow, as was the case in the 1970s and 1980s, a specified strength of concrete to be achieved at an unduly low cement content. This topic is considered more fully on p. 335.

Rapid-hardening Portland cement

This cement comprises Portland cement sub-classes of 32.5 and 42.5 MPa as prescribed by BS EN 197-1 : 2000. Rapid-hardening Portland cement (Type III), as its name implies, develops strength more rapidly, and should, therefore, be correctly described as high early strength cement. The rate of hardening must not be confused with the rate of setting: in fact, ordinary and rapid-hardening cements have similar setting times, prescribed by BS 12 : 1996 as an initial setting time of not less than 45 minutes. The final setting

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time is no longer prescribed. BS EN 197-1 : 2000 does not prescribe fineness.

The increased rate of gain of strength of the rapid-hardening Portland cement is achieved by a higher C_3S content (higher than 55 per cent, but sometimes as high as 70 per cent) and by a finer grinding of the cement clinker. British Standard BS 12 : 1996, unlike previous versions of BS 12, does not prescribe the fineness of cement, either ordinary or rapid-hardening. However, the standard provides for an optional *controlled fineness Portland cement* and so does BS EN 197-1 : 2000. The range of fineness is agreed between the manufacturer and the user. Such cement is valuable in applications where it makes it easier to remove excess water from the concrete during compaction because the fineness is more critical than the compressive strength.

In practice, rapid-hardening Portland cement has a higher fineness than ordinary Portland cement. Typically, ASTM Type III cements have a specific surface, measured by the Blaine method,

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of 450 to 600 m^2/kg, compared with 300 to 400 m^2/kg for Type I cement. The higher fineness significantly increases the strength at 10 to 20 hours, the increase persisting up to about 28 days. Under wet curing conditions, the strengths equalize at the age of 2 to 3 months, but later on the strength of the cements with a lower fineness surpasses that of the high fineness cements.[2.9]

This behaviour should not be extrapolated to cements with a very high fineness, which increase the water demand of the mix. In consequence, at a given cement content and for a given workability, the water/cement ratio is increased and this offsets the benefits of the higher fineness with respect to early strength.

The requirements of soundness and chemical properties are the same for rapid-hardening as for ordinary Portland cement and need not, therefore, be repeated.

The use of rapid-hardening cement is indicated where a rapid strength development is de-

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sired, e.g. when formwork is to be removed early for re-use, or where sufficient strength for further construction is wanted as quickly as practicable. Rapid-hardening cement is not much dearer than ordinary cement but it accounts for only a few per cent of all cement manufactured in the United Kingdom and in the United States. Because, however, the rapid gain of strength means a high rate of heat development, rapid-hardening Portland cement should not be used in mass construction or in large structural sections. On the other hand, for construction at low temperatures the use of cement with a high rate of heat evolution may prove a satisfactory safeguard against early frost damage.

Special very rapid-hardening Portland cements

There exist several specially manufactured cements which are particularly rapid hardening. One of these, a so-called *ultra high early strength cement*. This type of cement is not standardized

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but rather supplied by individual cement manufacturers. Generally, the rapid strength development is achieved by grinding the cement to a very high fineness: 700 to 900 m^2/kg. Because of this, the gypsum content has to be higher (4 per cent expressed as SO_3) than in cements complying with BS EN 197-1 : 2000, but in all other respects the ultra high early strength cement satisfies the requirements of that standard. It can be noted that the high gypsum content has no adverse effect on long-term soundness as the gypsum is used up in the early reactions of hydration.

The effect of the fineness of cement on the development of strength is illustrated in Fig. 2.3. All the cements used in this study[2.19] had the C_3S content between 45 and 48 per cent, and the C_3A content between 14.3 and 14.9 per cent.

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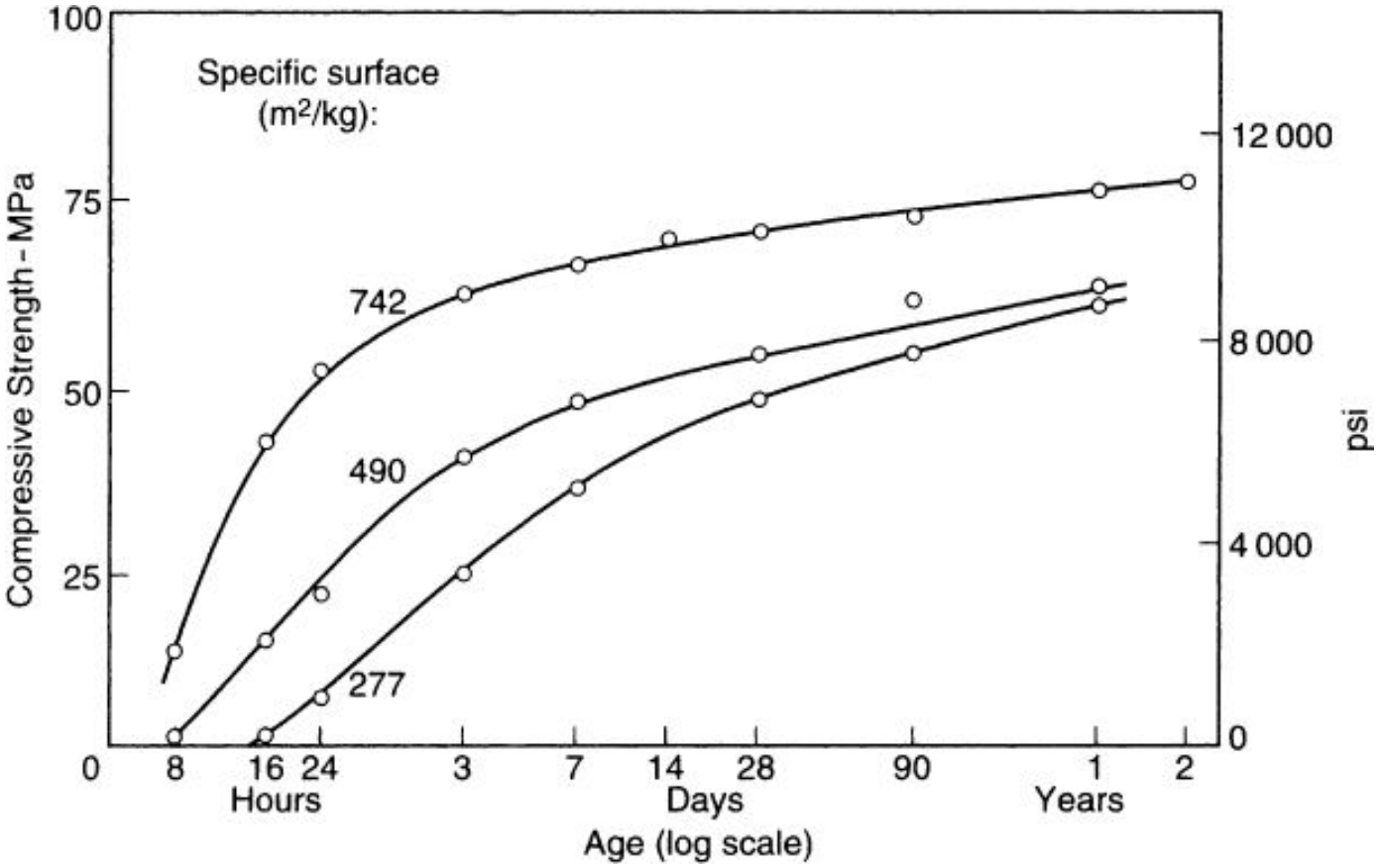


Fig. 2.3. Increase in strength of concrete with a water/cement ratio of 0.40 using Portland cements of varying specific surface (determined by the air permeability method)[2.19]

Ultra high early strength cement is manufactured by separating fines from rapid-hardening Portland cement by a cyclone air elutriator. Because of its high fineness, the ultra high early strength cement has a low bulk density and deteriorates rapidly on exposure. High fineness leads to rapid hydration, and therefore to a high

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rate of heat generation at early ages and to a rapid strength development; for instance, the 3-day strength of rapid-hardening Portland cement is reached at 16 hours, and the 7-day strength at 24 hours.[2.35] There is, however, little gain in strength beyond 28 days. Typical strengths of 1:3 concretes made with the ultra high early strength cement are given in Table 2.6. (The ratio 1:3 represents the proportion of cement to aggregate by mass.)

Table 2.6. Typical Values of Strength of a 1:3 Concrete made with Ultra High Early Strength Portland Cement[2.35]

Age	*Compressive strength at water/cement ratio of:*					
	0.40		*0.45*		*0.50*	
	MPa	*psi*	*MPa*	*psi*	*MPa*	*psi*
8 hours	12	1800	10	1400	7	1000
16 hours	33	4800	26	3800	22	3200
24 hours	39	5700	34	5000	30	4300
28 days	59	8600	57	8200	52	7600
1 year	62	9000	59	8600	57	8200

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More recent ultra high early strength cements have been reported[2.12] to have a very high C_3S content, 60 per cent, and a very low C_2S content, 10 per cent. The initial set occurred at 70 minutes but the final set followed soon after, at 95 minutes.[2.21] We should note, however, that for the same mix proportions, the use of ultra high early strength cement results in a lower workability.

Ultra high early strength cement has been used successfully in a number of structures where early prestressing or putting into service is of importance. Shrinkage and creep are not significantly different from those obtained with other Portland cements when the mix proportions are the same;[2.36] in the case of creep, the comparison has to be made on the basis of the same stress/strength ratio (see p. 456).

The ultra high early strength cements discussed so far contain no integral admixtures and are fundamentally of the Portland-cement-only variety. There exist also cements with a propri-

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etary composition. One of these is the so-called *regulated-set cement*, or *jet cement*, developed in the United States. The cement consists essentially of a mixture of Portland cement and calcium fluoroaluminate ($C_{11}A_7.CaF_2$) with an appropriate retarder (usually citric acid or lithium salts). The setting time of the cement can vary between 1 and 30 minutes (the strength development being slower the slower the setting) and is controlled in the manufacture of the cement as the raw materials are interground and burnt together. Grinding is difficult because of hardness differences.[2.65]

The early strength development is controlled by the content of calcium fluoroaluminate: when this is 5 per cent, about 6 MPa (900 psi) can be achieved at 1 hour; a 50 per cent mixture will produce 20 MPa (3000 psi) at the same time or even earlier. These values are based on a mix with a cement content of 330 kg/m^3 (560 lb/yd^3). The later strength development is similar to that of the parent Portland cement but at room temperature

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there is virtually no gain in strength between 1 and 3 days.

A typical Japanese jet cement[2.23] has a Blaine specific surface of 590 m^2/kg and an oxide composition (in per cent) as follows:

CaO	SiO_2	Al_2O_3	Fe_2O_3	SO_3
59	14	11	2	11

At a water/cement ratio of 0.30, compressive strengths of 8 MPa (1200 psi) at 2 hours, and 15 MPa (2260 psi) at 6 hours, were reached.[2.30] Drying shrinkage of concrete made with jet cement was found[2.23] to be lower than when Portland cement at the same content per cubic metre of concrete was used. Also, permeability at ages up to 7 days is very much lower.[2.23] These features are important when regulated-set cement is used for urgent repairs, for which this cement is particularly appropriate in view of its rapid setting and very rapid early strength development. Clearly, the mixing procedure must be appropriate. When required, a retarding admixture can be used.[2.23]

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Regulated-set cement is vulnerable to sulfate attack because of the high content of calcium aluminate.[2.37]

There exist other special, very rapid-hardening cements. These are sold under proprietary or trade names and have undisclosed composition. For these reasons, it would not be appropriate or reliable to discuss them in this book. However, to give an indication of what is available, at least in some countries, and to indicate the performance of such cements, one of these will be discussed below. Let us call it Cement X.

Cement X is a blended cement consisting of about 65 per cent of Portland cement with a Blaine fineness of 500 m^2/kg, about 25 per cent of Class C fly ash, and undisclosed functional chemical additions. These are likely to include citric acid, potassium carbonate and a superplasticizer, but no chlorides. The cement is used, typically, at a content of 450 kg per cubic metre of concrete (or 750 lb/yd^3) with a water/cement ratio of approximately 0.25. The setting time is

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30 minutes or more. It is claimed that the concrete can be placed at temperatures slightly below freezing point, but insulation of concrete to retain heat is necessary.

The strength development of concrete made with Cement X is very rapid: about 20 MPa (or 3000 psi) at 4 hours. The 28-day compressive strength is about 80 MPa (or 12 000 psi). The concrete is said to have a good resistance to sulfate attack and to freezing and thawing, without air entrainment. The latter is due to the very low water/cement ratio. Shrinkage is also said to be low.

These features make Cement X appropriate for rapid repair work, and possibly also for precast concrete. It should be noted, however, that Cement X has an alkali content of about 2.4 per cent (expressed as soda equivalent) and this should be borne in mind when alkali-reactive aggregates may be used. Because of its high reactivity and fineness, storage of this cement under very dry conditions is essential.

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Low heat Portland cement

The rise in temperature in the interior of a large concrete mass due to the heat development by the hydration of cement, coupled with a low thermal conductivity of concrete, can lead to serious cracking (see p. 396). For this reason, it is necessary to limit the rate of heat evolution of the cement used in this type of structure: a greater proportion of the heat can then be dissipated and a lower rise in temperature results.

Cement having such a low rate of heat development was first produced for use in large gravity dams in the United States, and is known as low heat Portland cement (Type IV). However, for some time now, Type IV cement has not been produced in the United States.

In the United Kingdom, low heat Portland cement is covered by BS 1370 : 1979, which limits the heat of hydration of this cement to 250 J/g (60 cal/g) at the age of 7 days, and 290 J/g (70 cal/g) at 28 days.

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The limits of lime content of low heat Portland cement, after correction for the lime combined with SO_3, are:

$$\frac{CaO}{2.4(SiO_2) + 1.2(Al_2O_3) + 0.65(Fe_2O_3)} \leqslant 1$$

and

$$\frac{CaO}{1.9(SiO_2) + 1.2(Al_2O_3) + 0.65(Fe_2O_3)} \geqslant 1.$$

The rather lower content of the more rapidly hydrating compounds, C_3S and C_3A, results in a slower development of strength of low heat cement as compared with ordinary Portland cement, but the ultimate strength is unaffected. In any case, to ensure a sufficient rate of gain of strength the specific surface of the cement must be not less than 320 m^2/kg. There is no separate recognition of low heat Portland cement in the European Standard BS EN 197-1 : 2000.

In the United States, Portland–pozzolana cement Type P can be specified to be of the low heat variety; the Type IP Portland–pozzolana cement

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can be required to have moderate heat of hydration, which is denoted by the suffix MH. ASTM Standard C 595-10 deals with these cements.

In some applications, a very low early strength may be a disadvantage, and for this reason a so-called modified (Type II) cement was developed in the United States. This modified cement successfully combines a somewhat higher rate of heat development than that of low heat cement with a rate of gain of strength similar to that of ordinary Portland cement. Modified cement is recommended for use in structures where a moderately low heat generation is desirable or where moderate sulfate attack may occur. This cement is extensively used in the United States.

Modified cement, referred to as Type II cement, and low heat cement (Type IV) are covered by ASTM C 150-09.

As mentioned earlier, Type IV cement has not been used in the United States for some time, and the problem of avoiding excessive generation of heat due to the hydration of cement is usually

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solved by other means. These include the use of fly ash or pozzolana and a very low cement content. The cement used can be Type II cement with a heat of hydration of 290 J/g (70 cal/g) at 7 days (offered as an option in ASTM C 150-09), as compared with 250 J/g (60 cal/g) for Type IV cement.

Sulfate-resisting cement

In discussing the reactions of hydration of cement, and in particular the setting process, mention was made of the reaction between C_3A and gypsum ($CaSO_4.2H_2O$) and of the consequent formation of calcium sulfoaluminate. In hardened cement, calcium aluminate hydrate can react with a sulfate salt from outside the concrete in a similar manner: the product of addition is calcium sulfoaluminate, forming within the framework of the hydrated cement paste. Because the increase in the volume of the solid phase is 227 per cent, gradual disintegration of concrete results. A second type of reaction is that of base exchange

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between calcium hydroxide and the sulfates, resulting in the formation of gypsum with an increase in the volume of the solid phase of 124 per cent.

These reactions are known as *sulfate attack*. The salts particularly active are magnesium sulfate and sodium sulfate. Sulfate attack is greatly accelerated if accompanied by alternating wetting and drying.

The remedy lies in the use of cement with a low C_3A content, and such cement is known as sulfate-resisting Portland cement. The British Standard for this cement, BS 4027 : 1996, stipulates a maximum C_3A content of 3.5 per cent. The SO_3 content is limited to 2.5 per cent. In other respects, sulfate-resisting cement is similar to ordinary Portland cement but it is not separately recognized in BS EN 197-1 : 2000. In the United States, sulfate-resisting cement is known as Type V cement and is covered by ASTM C 150-09. This specification limits the C_3A content to 5 per cent, and also restricts the sum of the content of

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C_4AF plus twice the C_3A content to 25 per cent. The magnesia content is limited to 6 per cent. There exists also a cement with moderate sulfate resistance (ASTM C 595-10).

The role of C_4AF is not quite clear. From the chemical standpoint, C_4AF would be expected to form calcium sulfoaluminate, as well as calcium sulfoferrite, and thus cause expansion. It seems, however, that the action of calcium sulfate on *hydrated* cement is smaller the lower the Al_2O_3:Fe_2O_3 ratio. Some solid solutions are formed and they are liable to comparatively little attack. The tetracalcium ferrite is even more resistant, and it may form a protective film over any free calcium aluminate.[2.6]

As it is often not feasible to reduce the $A1_2O_3$ content of the raw material, Fe_2O_3 may be added to the mix so that the C_4AF content increases at the expense of C_3A.[2.7]

An example of a cement with a very low Al_2O_3:Fe_2O_3 ratio is the *Ferrari cement*, in

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whose manufacture iron oxide is substituted for some of the clay. A similar cement is produced in Germany under the name of *Erz cement*. The name of *iron-ore cement* is also used for this type of cement.

The low C_3A content and comparatively low C_4AF content of sulfate-resisting cement mean that it has a high silicate content and this gives the cement a high strength but, because C_2S represents a high proportion of the silicates, the early strength is low. The heat developed by sulfate-resisting cement is not much higher than that of low heat cement. It could therefore be argued that sulfate-resisting cement is theoretically an ideal cement but, because of the special requirements for the composition of the raw materials used in its manufacture, sulfate-resisting cement cannot be generally and cheaply made.

It should be noted that the use of sulfate-resisting cement may be disadvantageous when there is a risk of the presence of chloride ions in the concrete containing steel reinforcement or other

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embedded steel. The reason for this is that C_3A binds chloride ions, forming calcium chloroaluminate. In consequence, these ions are not available for initiation of corrosion of the steel. This topic is discussed on p. 571.

Provision for a low-alkali sulfate-resisting cement is made in BS 4027 : 1996. In this connection, it is worth noting that a low alkali content in cement is beneficial with respect to sulfate attack, regardless of the C_3A content in the cement. The reason for this is that a low alkali content reduces the early availability of sulfate ions for reaction with the C_3A;[2.12] it is not known whether this effect persists for a long time.

White cement and pigments

For architectural purposes, white or a pastel colour concrete is sometimes required. To achieve best results it is advisable to use white cement with, of course, a suitable fine aggregate and, if the surface is to be treated, also an appropriate coarse aggregate. White cement has also the ad-

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vantage that it is not liable to cause staining because it has a low content of soluble alkalis.

White Portland cement is made from raw materials containing very little iron oxide (less than 0.3 per cent by mass of clinker) and manganese oxide. China clay is generally used, together with chalk or limestone, free from specified impurities. Oil or gas is used as fuel for the kiln in order to avoid contamination by coal ash. Since iron acts as a flux in clinkering, its absence necessitates higher kiln temperatures (up to 1650 °C) but sometimes cryolite (sodium aluminium fluoride) is added as a flux.

Contamination of the cement with iron during grinding of clinker has also to be avoided. For this reason, instead of the usual ball mill, the rather inefficient flint pebble grinding or expensive nickel and molybdenum alloy balls are used in a stone- or ceramic-lined mill. The cost of grinding is thus higher, and this, coupled with the more expensive raw materials, makes white

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cement rather expensive (about three times the price of ordinary Portland cement).

Because of this, white cement concrete is often used in the form of a facing placed against ordinary concrete backing, but great care is necessary to ensure full bond between the two concretes. To obtain good colour, white concrete of rich-mix proportions is generally used, the water/cement ratio being not higher than about 0.4. A possible saving in some cases can be achieved by a partial replacement of white cement by blastfurnace slag, which has a very light colour.

Strictly speaking, white cement has a faint green or yellow hue, depending on impurities; traces of chromium, manganese, and iron are mainly responsible for the slight coloration of green, bluish-green, and yellow, respectively.[2.20]

A typical compound composition of white Portland cement is given in Table 2.7 but the C_3S and C_2S contents may vary widely. White cement has a slightly lower specific gravity than ordinary Portland cement, generally between 3.05 and

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3.10. Because the brightness of the white colour is increased by a higher fineness of cement, it is usually ground to a fineness of 400 to 450 kg/m^2. The strength of white Portland cement is usually somewhat lower than that of ordinary Portland cement but white cement nevertheless satisfies the requirements of BS 12 : 1996.

Table 2.7. Typical Compound Composition of White Portland Cement

Compound	*Content, per cent*
C_3S	51
C_2S	26
C_3A	11
C_4AF	1
SO_3	2.6
Alkalis	0.25

White high-alumina cement is also made; this is considered on p. 103.

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When a pastel colour is required, white concrete can be used as a base for painting. Alternatively, pigments can be added to the mixer; those are powders of fineness similar to, or higher than, that of cement. A wide range of colours is available; for example, iron oxides can produce yellow, red, brown and black colours; chromic oxide produces green colour, and titanium dioxide produces white colour.[2.38] It is essential that the pigments do not affect adversely the development of strength of the cement or affect air entrainment. For instance, carbon black, which is extremely fine, increases the water demand and reduces the air content of the mix. For this reason, some pigments are marketed in the United States with an interground air-entraining agent; it is, of course, essential to be aware of this at the mix proportioning stage.

Mixing of concrete with pigments is not common because it is rather difficult to maintain a uniform colour of the resulting concrete. An improvement in the dispersion of the pigment can

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be obtained by the use of superplasticizers.[2.42] However, it is essential to verify the compatibility of any pigment with the admixtures which it is proposed to use. When the mix contains silica fume, light-coloured pigment may not perform well because of the extreme fineness of silica fume which exerts a masking effect.

Requirements for pigments are given in BS EN 12878 : 2005; American specification ASTM C 979-05 covers coloured and white pigments. It is desirable that the 28-day compressive strength be not less than 90 per cent of the strength of a pigment-free control mix, and the water demand is required to be not more than 110 per cent of the control mix. Setting time must not be unduly affected by the pigment. It is essential that pigments are insoluble and not affected by light.

A better way to obtain a uniform and durable coloured concrete is to use coloured cement. This consists of white cement interground with 2 to 10 per cent of pigment, usually an inorganic oxide. Specifications for the use of this type of cement

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are given by the individual manufacturers of this rather specialized product. Because the pigment is not cementitious, slightly richer mixes than usual should be used. The use of coloured concrete is reviewed by Lynsdale and Cabrera.[2.38]

For paving blocks, a ‘dry-shake’ of a mixture of pigment, cement, and hard fine aggregate is sometimes applied prior to finishing.

Portland blastfurnace cement

Cements of this name consist of an intimate mixture of Portland cement and ground granulated blastfurnace slag (in ASTM parlance, simply slag). This slag is a waste product in the manufacture of pig iron, about 300 kg of slag being produced for each tonne of pig iron. Chemically, slag is a mixture of lime, silica, and alumina, that is, the same oxides that make up Portland cement but not in the same proportions. There exist also non-ferrous slags; their use in concrete may become developed in the future.[2.39]

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Blastfurnace slag varies greatly in composition and physical structure depending on the processes used and on the method of cooling of the slag. For use in the manufacture of blastfurnace cement, the slag has to be quenched so that it solidifies as glass, crystallization being largely prevented. This rapid cooling by water results also in fragmentation of the material into a granulated form. Pelletizing, which requires less water, can also be used.

Slag can make a cementitious material in different ways. Firstly, it can be used together with limestone as a raw material for the conventional manufacture of Portland cement in the dry process. Clinker made from these materials is often used (together with slag) in the manufacture of Portland blastfurnace cement. This use of slag, which need not be in glass form, is economically advantageous because lime is present as CaO so that the energy to achieve decarbonation (see p. 3) is not required.

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Secondly, granulated blastfurnace slag, ground to an appropriate fineness, can be used on its own, but in the presence of an alkali activator or starter, as a cementitious material; in other words, ground granulated blastfurnace slag, abbreviated as *ggbs*, is a hydraulic material.[2.41] It is used as such in masonry mortar and in other construction, but the use of ggbs alone is outside the scope of this book.

The third, and in most countries by far the major, use of ggbs is in Portland blastfurnace cement, as defined in the opening paragraph of this section. This type of cement can be produced either by intergrinding Portland cement clinker and dry granulated blastfurnace slag (together with gypsum) or by dry blending of Portland cement powder and ggbs. Both methods are used successfully, but it should be noted that slag is harder than clinker, and this should be taken into account in the grinding process. Separate grinding of granulated slag results in a smoother sur-

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face texture, which is beneficial for workability.[2.45]

Another approach is to feed dry-ground granulated blastfurnace slag into the mixer at the same time as Portland cement: Portland blastfurnace cement concrete is thus manufactured in situ. This procedure is covered by BS 5328 : 1991 (withdrawn and replaced by BS EN 206-1 : 2000).

A Belgian development is the *Trief process* in which wet-ground granulated slag is fed in the form of a slurry direct into the concrete mixer, together with Portland cement and aggregate. The cost of drying the slag is thus avoided, and grinding in the wet state results in a greater fineness than would be obtained with dry grinding for the same power input.

There are no detailed requirements for the content of the individual oxides in ggbs to be used in concrete, but slags with the following

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percentages are known to be satisfactory in cement:[2.54]

lime	40 to 50
silica	30 to 40
alumina	8 to 18
magnesia	0 to 8.

Lower amounts of lime and higher amounts of magnesia are also used.[2.56] The magnesia is not in crystalline form and does not therefore lead to harmful expansion.[2.58] Small amounts of iron oxide, manganese oxide, alkalis and sulfur can also be present.

The specific gravity of ggbs is about 2.9, which is somewhat lower than the specific gravity of Portland cement (that is, 3.15). The specific gravity of blended cement is correspondingly affected.

When Portland blastfurnace cement is mixed with water, the Portland cement component begins to hydrate first, although there is also a small

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amount of immediate reaction of ggbs which releases calcium and aluminium ions into solution.[2.56] The ggbs then reacts with alkali hydroxide; this is followed by reaction with calcium hydroxide released by Portland cement, C-S-H being formed.[2.56]

European Standard BS EN 197-1 : 2000 requires that, for use in the production of any of the blended cements containing ggbs, the slag has to satisfy certain requirements. According to BS 146 : 1996 and BS 4246 : 1996, at least two-thirds of the slag must consist of glass. At least two-thirds of the total mass of slag must consist of the sum of CaO, MgO, and SiO_2. Also, the ratio of the mass of CaO plus MgO to the mass of SiO_2 must exceed 1.0. This ratio assures a high alkalinity, without which the slag would be hydraulically inactive. The shape of ggbs is angular, in contrast to fly ash. The replacement BS EN 197-4 : 2004 is not prescriptive in this respect.

The ASTM Specification C 989-09a prescribes a maximum proportion of 20 per cent of

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ggbs coarser than a 45 μm sieve. The British Standards do not use such a requirement. The specific surface of ggbs is not normally determined, but an increase in fineness of Portland blastfurnace cement, accompanied by optimizing the SO_3 content, leads to an increased strength; when the specific surface is increased from 250 to 500 m^2/kg (by the Blaine method) the strength is more than doubled.[2.59]

The American approach, given in ASTM C 989-09a, is to grade blastfurnace slag according to its hydraulic activity. This is determined by the strength of mortars of standard mass proportions containing slag as compared with mortars containing Portland cement only. Three grades are recognized.

European Standard BS EN 197-1 : 2000 recognizes three classes of Portland blastfurnace cement, called Blastfurnace cement III/A, III/B, and III/C. All of them are allowed to contain up to 5 per cent of filler, but they differ in the mass of ggbs as a percentage of the mass of the total ce-

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mentitious material, that is Portland cement plus ggbs exclusive of calcium sulfate and the manufacturing additive. The percentages of slag are as follows:

Class III/A	36 to 65
Class III/B	66 to 80
Class III/C	81 to 95.

Class III/C Blastfurnace cement, at its upper limit of ggbs, is virtually a pure slag cement, which, as already stated, will not be further considered in this book.

Cements with a high content of ggbs can be used as low heat cements in structures in which a large mass of concrete is to be placed so that the temperature increase arising from the early development of the heat of hydration of cement needs to be controlled; this topic is considered on p. 395. British Standard BS 4246 : 2002 (replaced by BS EN 197-4 : 2004) provides an option for a purchaser's specification of the heat of hydration. It must not be forgotten that a concom-

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itant of a low rate of heat development is a low rate gain of strength. Therefore, in cold weather the low heat of hydration of Portland blastfurnace cement, coupled with a moderately low rate of strength development, can lead to frost damage.

Cements containing ggbs are often also beneficial from the standpoint of resistance to chemical attack. This is discussed on p. 667.

Hydraulic activity of ggbs is conditional on its high fineness but, as in the case of other cements, the fineness of Portland blastfurnace cements is not specified in the British Standards. The only exception is when ggbs and Portland cement are dry blended: in that case, ggbs has to conform to BS 6699 : 1992 (1998). In practice, the fineness of ggbs tends to be higher than that of Portland cement.

In addition to the Portland blastfurnace cements discussed above, BS EN 197-1 : 2000 recognizes two cements containing lesser amounts of slag. These are cements Class II A-S with 6

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to 20 per cent of ggbs and Class II B-S with 21 to 35 per cent of ggbs, by mass. These are called *Portland slag cements*; they form part of the large variety of Class II cements, all of which consist predominantly of Portland cement, but are blended with another cementitious material (see Table 2.4).

British Standards BS 146 : 1996 and BS 4246 : 1996 contain some additional requirements and also classify the cements on the basis of compressive strength. The classification is the same as for other cements but it is important to note that two of the classes of Portland blastfurnace cement are subdivided into categories: a low early strength; an ordinary early strength; and a high early strength. These are a reflection of the progress of hydration of blastfurnace cements: at very early ages, the rate of hydration is lower than in the case of Portland cement alone. British Standard BS 4246 : 1996 allows cements with a slag content of 50 to 85 per cent by mass to have a 7-day compressive strength as low as 12 MPa.

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Supersulfated cement

Supersulfated cement is made by intergrinding a mixture of 80 to 85 per cent of granulated blast-furnace slag with 10 to 15 per cent of calcium sulfate (in the form of dead-burnt gypsum or anhydrite) and up to 5 per cent of Portland cement clinker. A fineness of 400 to 500 m^2/kg is usual. Supersulfated cement is thus fundamentally different from Portland cement, in which calcium silicate is the main component. The cement has to be stored under very dry conditions as otherwise it deteriorates rapidly.

Supersulfated cement is used extensively in Belgium, where it is known as *ciment métallurgique sursulfaté*, also in France, and was previously manufactured in Germany (under the name of *Sulfathüttenzement*). In the United Kingdom, the cement was covered by BS 4248- : 2004 (withdrawn) but, because of production difficulties, the manufacture of the cement has been discontinued. The European standard for super-

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sulfated cement is BS EN 15743 : 2010, which gives physical and chemical requirements.

Supersulfated cement is highly resistant to sea water and can withstand the highest concentrations of sulfates normally found in soil or ground water, and is also resistant to peaty acids and to oils. Concrete with a water/cement ratio not greater than 0.45 has been found not to deteriorate in contact with weak solutions of mineral acids of pH down to 3.5. For these reasons, supersulfated cement is used in the construction of sewers and in contaminated ground, although it has been suggested that this cement is less resistant than sulfate-resisting Portland cement when the sulfate concentration exceeds 1 per cent.[2.31]

The heat of hydration of supersulfated cement is very low: about 170 to 190 J/g (40 to 45 cal/g) at 7 days, and 190 to 210 J/g (45 to 50 cal/g) at 28 days.[2.6] The cement is, therefore, suitable for mass concrete construction but care must be taken if used in cold weather because the rate of strength development is considerably reduced at

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low temperatures. The rate of hardening of supersulfated cement increases with temperature up to about 50 °C (122 °F), but at higher temperatures anomalous behaviour has been encountered. For this reason, steam curing above 50 °C (122 °F) should not be used without prior tests. It may also be noted that supersulfated cement should not be mixed with Portland cements because the lime released by the hydration of an excessive amount of the latter interferes with the reaction between the slag and the calcium sulfate.

Wet curing for not less than four days after casting is essential as premature drying out results in a friable or powdery surface layer, especially in hot weather, but the depth of this layer does not increase with time.

Supersulfated cement combines chemically with more water than is required for the hydration of Portland cement, so that concrete with a water/cement ratio of less than 0.4 should not be made. Mixes leaner than about 1:6 are not recommended. The decrease in strength with an increase in

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the water/cement ratio has been reported to be smaller than in other cements but, because the early strength development depends on the type of slag used in the manufacture of the cement, it is advisable to determine the actual strength characteristics prior to use. Typical strengths attainable are given in Table 2.8. BS EN 15743 : 2010 specifies three classes of strength: 32.5, 42.5, and 52.5.

Table 2.8. Typical Values of Strength of Supersulfated Cement[2.6]

Age, days	*Compressive strength*			
	Standard vibrated mortar test		*Standard concrete test*	
	MPa	*psi*	*MPa*	*psi*
1	7	1000	5–10	700–1500
3	28	4000	17–28	2500–4000
7	35–48	5000–7000	28–35	4000–5000
28	38–66	5500–9500	38–45	5500–6500
6 months	—	—	52	7500

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Pozzolanas

One of the common materials classified as cementitious in this book (although in reality only in latent form) is pozzolana, which is a natural or artificial material containing silica in a reactive form. A more formal definition of ASTM 618-08a describes pozzolana as a siliceous or siliceous and aluminous material which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties. It is essential that pozzolana be in a finely divided state as it is only then that silica can combine with calcium hydroxide (produced by the hydrating Portland cement) in the presence of water to form stable calcium silicates which have cementitious properties. We should note that the silica has to be amorphous, that is, glassy, because crystalline silica has very low reactivity. The glass content can be determined by X-ray diffraction spectro-

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scopy or by solution in hydrochloric acid and potassium hydroxide.[2.24]

Broadly speaking, pozzolanic materials can be natural in origin or artificial. The main artificial pozzolanic material, fly ash, will be considered in the next section.

The natural pozzolanic materials most commonly met with are: volcanic ash – the original pozzolana – pumicite, opaline shales and cherts, calcined diatomaceous earth, and burnt clay. ASTM C 618-08a describes these materials as Class N.

Some natural pozzolanas may create problems because of their physical properties; e.g. diatomaceous earth, because of its angular and porous form, requires a high water content. Certain natural pozzolanas improve their activity by calcination in the range of 550 to 1100 °C, depending on the material.[2.63]

Rice husks are a natural waste product and there is interest in using this material in concrete.

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Rice husks have a very high silica content, and slow firing at a temperature of 500 to 700 °C results in an amorphous material with a porous structure. Thus the specific surface (measured by nitrogen adsorption) can be as high as 50 000 m^2/kg, even though the particle size is large: 10 to 75 μm.[2.26] The rice husk ash particles have complex shapes, reflecting their plant origins[2.27] and they therefore have a high water demand unless interground with clinker so as to break down the porous structure.

Rice husk ash is reported to contribute to the strength of concrete already at 1 to 3 days.[2.26]

However, to achieve adequate workability, as well as high strength, the use of superplasticizers may be necessary;[2.28] this negates the economic benefits of the use of rice husk ash in less affluent areas of the world where collection of the husks for processing may also present problems. The use of rice husks can lead to increased shrinkage[2.80] but this has not been confirmed.

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There exist also other processed amorphous silica materials. One of these is *metakaolin*, obtained by calcination of pure or refined kaolinitic clay at a temperature of between 650 and 850 °C, followed by grinding to achieve a fineness of 700 to 900 m^2/kg. The resulting material exhibits high pozzolanicity.[2.53,2.60]

The use of siliceous clay, ground to a very high fineness (specific surface of 4000 to 12 000 m^2/kg, determined by nitrogen adsorption), as a highly reactive pozzolana has been suggested by Kohno *et al*.[2.61]

For an assessement of pozzolanic activity with cement, ASTM C 311-07 prescribes the measurement of a *strength activity index*. This is established by the determination of strength of mortar with a specified replacement of cement by pozzolana. The outcome of the test is influenced by the cement used, especially its fineness and alkali content.[2.25] There is also a *pozzolanic activity in-*

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dex with lime, which determines the total activity of pozzolana.

The pozzolanicity of pozzolanic cements, that is, cements containing between 11 and 55 per cent of pozzolana and silica fume according to BS EN 197-1 : 2000, is tested according to BS EN 196-5 : 2005. The test compares the quantity of calcium hydroxide in an aqueous solution in contact with the hydrated pozzolanic cement, with the quantity of calcium hydroxide which saturates a solution of the same alkalinity. If the former concentration is lower than the latter, then the pozzolanicity of the cement is considered to be satisfactory. The underlying principle is that the pozzolanic activity consists of fixing of calcium hydroxide by the pozzolana so that the lower the resulting quantity of calcium hydroxide the higher the pozzolanicity.

Pozzolanicity is still imperfectly understood; specific surface and chemical composition are known to play an important role but, because they are inter-related, the problem is complex. It has

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been suggested that, in addition to reacting with $Ca(OH)_2$, pozzolanas react also with C_3A or its products of hydration.[2.76] A good review of the subject of pozzolanicity has been written by Massazza and Costa.[2.77]

There exists one other material, silica fume, which is formally an artificial pozzolana but whose properties put it into a class of its own. For this reason, silica fume will be considered in a separate section (see p. 86).

Fly ash

Fly ash, known also as *pulverized-fuel ash*, is the ash precipitated electrostatically or mechnically from the exhaust gases of coal-fired power stations; it is the most common artificial pozzolana. The fly ash particles are spherical (which is advantageous from the water requirement point of view) and have a very high fineness: the vast majority of particles have a diameter between less than 1 μm and 100 μm, and the specific surface of fly ash is usually between 250 and 600 m^2/kg, us-

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ing the Blaine method. The high specific surface of the fly ash means that the material is readily available for reaction with calcium hydroxide.

The specific surface of fly ash is not easy to determine because, in the air permeability test, the spherical particles pack more closely than the irregularly shaped particles of cement so that the resistance of fly ash to air flow is greater. On the other hand, the porous carbon particles in the ash allow air to flow through them, leading to a misleadingly high air flow.[2.62] Moreover, the determination of the specific gravity of fly ash (which enters the calculation of the specific surface, see p. 22) is affected by the presence of hollow spheres (whose specific gravity can be less than 1).[2.62] At the other extreme, some small particles which contain magnetite or haematite have a high specific gravity. The typical overall value of specific gravity is 2.35. An important use of the determination of the specific surface of fly ash is in detecting its variability.[2.64]

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The American classification of fly ash, given in ASTM C 618-08a, is based on the type of coal from which the ash originates. The most common fly ash derives from bituminous coal, is mainly siliceous, and is known as Class F fly ash.

Sub-bituminous coal and lignite result in high-lime ash, known as Class C fly ash. This will be considered later in the present section.

The pozzolanic activity of Class F fly ash is in no doubt, but it is essential that it has a constant fineness and a constant carbon content. The two are often interdependent because the carbon particles tend to be coarser. Modern boiler plants produce fly ash with a carbon content of about 3 per cent, but much higher values are encountered in fly ash from older plants. The carbon content is assumed to be equal to the loss on ignition, although the latter includes also any combined water or fixed CO_2 present.[2.64] British Standard BS EN 450-1 : 2005 1 : 1997 specifies a maximum 12 per cent residue on the 45 μm sieve, which is a convenient basis of classification of size.

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The main requirements of ASTM C 618-08a are: a minimum content of 70 per cent of silica, alumina, and ferric oxide taken all together, a maximum SO_3 content of 5 per cent, a maximum loss on ignition of 12 per cent. Also, to control any alkali-aggregate reaction, the expansion of a mix with fly ash should not exceed that of a low alkali cement control mix at 14 days. British Standard BS 3892-1 : 1997 specifies a maximum content of SO_3 of 2.5 per cent and some other requirements. A limitation on the MgO content is no longer specified because it exists in a non-reactive form.

It should be noted that fly ash may affect the colour of the resulting concrete, the carbon in the ash making it darker. This may be of importance from the standpoint of appearance, especially when concretes with and without fly ash are placed side by side.

Let us now turn to Class C fly ash, that is, high-lime ash originating from lignite coal. Such ash may occasionally have a lime content as high

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as 24 per cent.[2.63] High-lime ash has some cementitious (hydraulic) properties of its own but, because its lime will combine with the silica and alumina portions of the ash, there will be less of these compounds to react with the lime liberated by the hydration of cement. The carbon content is low, the fineness is high, and the colour is light. However, the MgO content can be high, and some of the MgO as well as some of the lime can lead to deleterious expansion.[2.63]

The behaviour of high-lime ash is sensitive to temperature: specifically, in mass concrete when a rise in temperature occurs, the products of reaction may not be of high strength. However, the development of strength is not simply related to temperature, being satisfactory in the region of 120 to 150 °C (250 to 300 °F) but not at about 200 °C (about 400 °F) when the products of reaction are substantially different.[2.55]

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Pozzolanic cements

Pozzolanas, being a latent hydraulic material, are always used in conjunction with Portland cement. The two materials may be interground or blended. Sometimes, they can be combined in the concrete mixer. The possibilities are thus similar to those of granulated blastfurnace slag (see p. 79). By far the largest proportion of pozzolanas used consists of siliceous fly ash (Class F), and we shall concentrate on that material.

European Standard BS EN 197-1 : 2000 recognizes two subclasses of *Portland fly ash cement*: Class II/A-V with a fly ash content of 6 to 20 per cent, and Class II/B-V with a fly ash content of 21 to 35 per cent. The British Standard for Portland pulverized-fuel ash cements, BS 6588 : 1996, has somewhat different limits for the fly ash content, the maximum value being 40 per cent. There is no great significance in the precise upper limit on the fly ash content. However, BS 6610 : 1991 allows an even higher content of fly ash, namely 53 per cent, in so-called *pozzolan-*

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ic cement. Like the high slag blastfurnace cement (see p. 81), pozzolanic cement has a low 7-day strength (minimum of 12 MPa) but also a low 28-day strength: minimum of 22.5 MPa. The concomitant advantage is a low rate of heat development so that pozzolanic cement is a low heat cement. Additionally, pozzolanic cement has some resistance to sulfate attack and to attack by weak acids.

Silica fume

Silica fume is a relatively recent arrival among cementitious materials. It was originally introduced as a pozzolana. However, its action in concrete is not only that of a very reactive pozzolana but is also beneficial in other respects (see p. 669). It can be added that silica fume is expensive.

Silica fume is also referred to as *microsilica* or *condensed silica fume*, but the term 'silica fume' has become generally accepted. It is a by-product of the manufacture of silicon and ferrosilicon al-

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loys from high-purity quartz and coal in a submerged-arc electric furnace. The escaping gaseous SiO oxidizes and condenses in the form of extremely fine spherical particles of amorphous silica (SiO_2); hence, the name silica fume. Silica in the form of glass (amorphous) is highly reactive, and the smallness of the particles speeds up the reaction with calcium hydroxide produced by the hydration of Portland cement. The very small particles of silica fume can enter the space between the particles of cement, and thus improve packing. When the furnace has an efficient heat recovery system, most of the carbon is burnt so that silica fume is virtually free from carbon and is light in colour. Furnaces without a full heat recovery system leave some carbon in the fume, which is therefore dark in colour.

The production of silicon alloys, which include non-ferrous metals, such as ferrochromium, ferromanganese, and ferromagnesium, also results in the formation of silica fume but its suit-

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ability for use in concrete has not yet been established.[2.67]

The usual ferrosilicon alloys have nominal silicon contents of 50, 75, and 90 per cent; at 48 per cent, the product is called silicon metal. The higher the silicon content in the alloy the higher the silica content in the resulting silica fume. Because the same furnace can produce different alloys, it is important to know the provenance of any silica fume to be used in concrete. In particular, ferrosilicon with a 50-per cent content of silicon results in a silica fume with a content of silica of only about 80 per cent. However, steady production of a given alloy results in a silica fume with constant properties.[2.66] Typical silica contents are as follows (per cent): silicon metal, 94 to 98; 90 per cent ferrosilicon, 90 to 96; and 75 per cent ferrosilicon, 86 to 90.[2.66]

The specific gravity of silica fume is generally 2.20, but it is very slightly higher when the silica content is lower.[2.66] This value can be compared

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with the specific gravity of Portland cement, which is 3.15. The particles of silica fume are extremely fine, most of them having a diameter ranging between 0.03 and 0.3 μm; the median diameter is typically below 0.1 μm. The specific surface of such fine particles cannot be determined using the Blaine method; nitrogen adsorption indicates a specific surface of about 20 000 m^2/kg, which is 13 to 20 times higher than the specific surface of other pozzolanic materials, determined by the *same* method.

Such a fine material as silica fume has a very low bulk density: 200 to 300 kg/m^3 (12 to 19 lb/ft^3). Handling this light powder is difficult and expensive. For this reason, silica fume is available in the densified form of micropellets, that is, agglomerates of the individual particles (produced by aeration), with a bulk density of 500 to 700 kg/m^3. Another form of silica fume is a slurry of equal parts by mass of water and silica fume. The density of the slurry is about 1300 to

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1400 kg/m^3. The slurry is stabilized and has been reported to have a pH of about 5.5, but this is of no consequence with respect to the use in concrete.[2.68] Periodic agitation is necessary to maintain a uniform distribution of the silica fume in the slurry. Admixtures, such as water reducers, superplasticizers or retarders, can be included in the slurry.[2.69]

Each of the different forms in which silica fume is available has operational advantages, but all forms can be successfully used; claims of significant beneficial effects of one or other of these forms upon the resulting concrete have not been substantiated.[2.70]

Although silica fume is usually incorporated in the mix at the batcher, in some countries, blended cement containing silica fume, usually 6.5 to 8 per cent by mass, is produced.[2.71] Such a blended cement simplifies the batching operations but, obviously, the content of silica fume in the

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total cementitious material cannot be varied to suit specific needs.

Few standards for silica fume or its use in concrete are in existence. ASTM C 1240-05 specifies the requirements for silica fume, but ASTM C 618-08a, by its title, excludes it. Indeed, the clause about the water requirement in that standard may well not be satisfied by silica fume.

Fillers

In the classification of blended Portland cements (see p. 65) it was mentioned that fillers may be included up to a certain maximum content. Indeed, fillers have been used in many countries for some time but it is only recently that their use became permitted in the United Kingdom.

A filler is a very finely-ground material, of about the same fineness as Portland cement, which, owing to its physical properties, has a beneficial effect on some properties of concrete, such as workability, density, permeability, capillarity, bleeding, or cracking tendency. Fillers are

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usually chemically inert but there is no disadvantage if they have some hydraulic properties or if they enter into harmless reactions with the products of reaction in the hydrated cement paste. Indeed, it has been found by Zielinska[2.44] that $CaCO_3$, which is a common filler, reacts with C_3A and C_4AF to produce $3CaO.Al_2O_3.CaCO_3.11H_2O$.

Fillers can enhance the hydration of Portland cement by acting as nucleation sites. This effect was observed in concrete containing fly ash and titanium dioxide in the form of particles smaller than 1 μm.[2.72] Ramachandran[2.74] found that, in addition to its nucleation role in the hydration of cement, $CaCO_3$ becomes partly incorporated into the C-S-H phase. This effect on the structure of the hydrated cement paste is beneficial.

Fillers can be naturally occurring materials or processed inorganic mineral materials. What is essential is that they have uniform properties, and especially fineness. They must not increase the water demand when used in concrete, unless used

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with a water-reducing admixture, or adversely affect the resistance of concrete to weathering or the protection against corrosion which concrete provides to the reinforcement. Clearly, they must not lead to a long-term retrogression of strength of concrete, but such a problem has not been encountered.

Because the action of fillers is predominantly physical, they have to be physically compatible with the cement in which they are included. Because the filler is softer than clinker, it is necessary to grind the composite material longer so as to ensure the presence of some very fine cement particles, which are necessary for early strength.

Although BS EN 197-1 : 2000 limits the filler content to 5 per cent, it allows the use of limestone up to 35 per cent, provided the remaining cementitious material is Portland cement only. This cement is known as *Portland limestone cement* (Class II/B-L). As limestone is in effect a type of filler, the limestone cement can be said to have a filler content of up to 35 per cent. It

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can be expected that, for some purposes, blended cements with a filler content of 15, or even 20, per cent are likely to be popular in the future. A higher filler content may cause a reduction in the strength of concrete: 10 per cent for a filler content of 10 per cent, and 12 per cent for 20 per cent.[1.97] These losses can be compensated for by a reduced ration of water to total cementitious material (w/cm).

Other cements

Among the numerous cements developed for special uses, *anti-bacterial cement* is of interest. It is a Portland cement interground with an anti-bacterial agent which prevents microbiological fermentation. This bacterial action is encountered in concrete floors of food processing plants where the leaching out of cement by acids is followed by fermentation caused by bacteria in the presence of moisture. Anti-bacterial cement can also be successfully used in swimming pools and similar places where bacteria or fungi are present.

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Another special cement is the so-called *hydrophobic cement*, which deteriorates very little during prolonged storage under unfavourable conditions. This cement is obtained by intergrinding Portland cement with 0.1 to 0.4 per cent of oleic acid. Stearic acid or pentachlorophenol can also be used.[2.10] These additions increase the grindability of clinker, probably due to electrostatic forces resulting from a polar orientation of the acid molecules on the surface of the cement particles. Oleic acid reacts with alkalis in cement to form calcium and sodium oleates which foam, so that air-entraining results. When this is not desired a detraining agent, such as tri-n-butyl phosphate, has to be added during grinding.[2.11]

The hydrophobic properties are due to the formation of a water-repellent film around each particle of cement. This film is broken during the mixing of the concrete, and normal hydration takes place but early strength is rather low.

Hydrophobic cement is similar in appearance to ordinary Portland cement but has a character-

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istic musty smell. In handling, the cement seems more flowing than other Portland cements.

Masonry cement, used in mortar in brickwork, is made by integrinding Portland cement, limestone and an air-entraining agent, or alternatively Portland cement and hydrated lime, granulated slag or an inert filler, and an air-entraining agent; other ingredients are usually also present. Masonry cements make a more plastic mortar than ordinary Portland cement; they also have a greater water-retaining property and lead to lower shrinkage. The strength of masonry cements is lower than that of ordinary Portland cement, particularly because a high air content is introduced, but this low strength is generally an advantage in brick construction. Masonry cement must not be used in structural concrete. The specification for masonry cement is given in ASTM C 91-05.

Three further cements should be mentioned. One is *expansive cement*, which has the property of expanding in its early life so as to counteract contraction induced by drying shrinkage. For this

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reason, expanding cement will be considered in Chapter 9.

The second cement is *oil-well cement*. This is a highly specialized product, based on Portland cement, used for grout or slurry to be pumped to depths of up to thousands of metres in the earth's crust where temperature can exceed 150 °C (or 300 °F) and pressure can be 100 MPa (or 15 000 psi). These values would apply typically to depths of about 5000 m (or 16 000 ft) but exploration holes to a depth of 10 000 m (or 33 000 ft) have been drilled and grouted.

The cements to be used in grout under these conditions must not set before reaching distant locations but subsequently they have to gain strength rapidly so as to allow resumption of the drilling operations. Sulfate resistance is often also required. Several classes of oil-well cement are recognized by the American Petroleum Institute, which prepares specifications for oil-well cements.[2.21]

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Essentially, oil-well cements have to have certain special features: (a) to have a particular fineness (to 'hold' a large amount of water); (b) to contain retarders or accelerators (see Chapter 5); (c) to contain friction reducers (to improve flow); (d) to contain lightweight additives (such as bentonite) to lower the density of the grout or densifying additives (such as barytes or haematite) to increase the density of the grout; and (e) to contain pozzolana or silica fume (to improve the strength at high temperatures).

Finally, we should mention *natural cement*. This is the name given to a cement obtained by calcining and grinding a so-called cement rock, which is a clayey limestone containing up to 25 per cent of argillaceous material. The resulting cement is similar to Portland cement, and is really intermediate between Portland cement and hydraulic lime. Because natural cement is calcined at temperatures too low for sintering, it contains practically no C_3S and is therefore slow hardening. Natural cements are rather variable in quality

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as adjustment of composition by blending is not possible. Because of this, as well as for economic reasons, natural cements are nowadays rarely used.

Which cement to use

The wide variety of cement Types (in American nomenclature) and cement Classes (in European classification) and, above all, of cementitious and other materials used in blended cements, may result in a bewildering impression. Which cement is best? Which cement should be used for a given purpose?

There is no simple answer to these questions but a rational approach will lead to satisfactory solutions.

First of all, no single cement is the best one under all circumstances. Even if cost is ignored, pure Portland cement is not the all-round winner, although in the past commercial interests extolled it as the true unadulterated product, second to none. As far back as 1985, about one-half of all

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cement produced in Western Europe and in China was blended, about two-thirds in India and in what was the Soviet Union, but only a minimal proportion in North America and in the United Kingdom,[2.29] possibly because of the influence of the Portland cement lobby there.

The use of blended cements has been steadily increasing in the 1980s and the 1990s, and it can be confidently expected that blended cements will eventually form the bulk of cements used world-wide. In the words of Dutron,[2.29] "pure Portland cements will be regarded as special cements reserved for applications where exceptional performance is required, particularly as far as mechanical strength is concerned". Even this last caveat is no longer valid as high performance concrete is best made with blended cements. Moreover, the durability of blended cements is equal to, and often better than, that of pure Portland cement.

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So, if no single cement is the best all round, we should look at the question: which cement should be used for a given purpose?

The chapters which follow discuss the properties of concrete both in the fresh state and when hardened. Many of these properties depend, to a greater or lesser extent, on the properties of the cement used: it is on this basis that the choice of cement can be made. However, in many cases, no one cement is the best one: more than one Type or Class can be used. The choice depends on availability, on cost – that important element in engineering decision-making – and on the particular circumstances of equipment, skilled labour force, speed of construction and, of course, on the exigencies of the structure and its environment.

It is intended to refer to the relevant properties of the different cements in the chapters dealing with fresh concrete, strength and, especially, durability, and also in Chapter 13, dealing with concretes with particular properties. Thus, it is there

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that views on the choice or appropriateness of various cements can be found.

High-alumina cement

The search for a solution to the problem of attack by gypsum-bearing waters on Portland cement concrete structures in France led Jules Bied to the development of a high-alumina cement, at the beginning of the twentieth century. This cement is very different in its composition, and also in some properties, from Portland cement and Portland blended cements so that its structural use is severely limited, but the concreting techniques are similar. For full treatment of the topic, the reader may consult a specialized book.[*]

[*] A. M. Neville, in collaboration with P. J. Wainwright, *High-alumina Cement Concrete* (Construction Press, Longman Group, 1975).

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Manufacture

From the name of the cement – high-alumina – it can be inferred that it contains a large proportion of alumina: typically, about 40 per cent each of alumina and lime, with about 15 per cent of ferrous and ferric oxides, and about 5 per cent of silica. Small amounts of TiO_2, magnesia, and the alkalis can also be present.

The raw materials are usually limestone and bauxite. Bauxite is a residual deposit formed by the weathering, under tropical conditions, of rocks containing aluminium, and consists of hydrated alumina, oxides of iron and titanium, and small amounts of silica.

There are several processes of manufacture of high-alumina cement. In one process, bauxite is crushed into lumps not larger than 100 mm (or 4 in.). Dust and small particles formed during this fragmentation are cemented into briquettes of similar size because dust would tend to damp the furnace. The second main raw material is usually

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limestone, also crushed to lumps of about 100 mm (or 4 in.).

Limestone and bauxite, in the required proportions, are fed into the top of a furnace which is a combination of the cupola (vertical stack) and reverberatory (horizontal) types. Pulverized coal is used for firing, its quantity being about 22 per cent of the mass of the cement produced. In the furnace, the moisture and carbon dioxide are driven off and the materials are heated by the furnace gases to the point of fusion at about 1600 °C. The fusion takes place at the lower end of the stack so that the molten material falls into the reverberatory furnace and thence through a spout into steel pans. The melt is now solidified into pigs, fragmented in a rotary cooler, and then ground in a tube mill. A very dark grey powder with a fineness of 290 to 350 m^2/kg is produced.

Because of the high hardness of high-alumina cement clinker, the power consumption and the wear of tube mills are considerable. This, coupled with the high prime cost of bauxite and the high

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temperature of firing, leads to a high price of high-alumina cement, compared with Portland cement. The price is, however, compensated for by some valuable properties for specific purposes.

It may be noted that, unlike the case of Portland cement, the materials used in the manufacture of high-alumina cement are completely fused in the kiln. This fact gave rise to the French name *ciment fondu*, and 'fondu cement' is sometimes used as a colloquial name in English.

Because of adverse publicity, associated with high-alumina cement in the United Kingdom in the 1970s (see p. 99), there have been attempts to use an alternative name of *aluminous cement*. However, this name is not correct because other cements, such as supersulfated cement and slag cements, also contain alumina in significant proportions. Yet a third name, *calcium aluminate cement*, is more appropriate but then, by contrast, we should refer to Portland cement as calcium silicate cement; this appellation is never used. In

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this book, we shall therefore use the traditional name of high-alumina cement.

High-alumina cement is no longer manufactured in the United Kingdom. However, there exists a British Standard for high alumina cement, BS 915 : 1972 (1983), which refers to BS 4550-3.1 : 1978 for fineness, strength, setting time and soundness. There is a European standard BS EN 14647 : 2005.

Composition and hydration

The main cementitious compounds are calcium aluminates of low basicity primarily CA and also $C_{12}A_7$.[2.32] Other compounds are also present: C_6A_4.FeO.S and an isomorphous C_6A_4.MgO.S.[2.13] The amount of C_2S or C_2AS does not account for more than a few per cent, and there are, of course, minor compounds present, but no free lime can exist. Thus unsoundness is never a problem in high-alumina cement although BS 915 : 1972 (1983) prescribes the conventional Le Chatelier test.

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The hydration of CA, which has the highest rate of strength development, results in the formation of CAH_{10}, a small quantity of C_2AH_8, and of alumina gel (Al_2O_3.aq). With time, these hexagonal CAH_{10} crystals, which are unstable both at normal and at higher temperatures, become transformed into cubic crystals of C_3AH_6 and alumina gel. This transformation is encouraged by a higher temperature and a higher concentration of lime or a rise in alkalinity.[2.14]

$C_{12}A_7$, which also hydrates rapidly, is believed to hydrate to C_2AH_8. The compound C_2S forms C-S-H, the lime liberated by hydrolysis reacting with excess alumina; no $Ca(OH)_2$ exists. The reactions of hydration of the other compounds, particularly those containing iron, have not been determined with any degree of certainty, but the iron held in glass is known to be inert.[2.15] Iron compounds are useful as a flux in the manufacture of high-alumina cement.

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The water of hydration of high-alumina cement is calculated to be up to 50 per cent of the mass of the dry cement,[2.6] which is about twice as much as the water required for the hydration of Portland cement, but mixes with a water/cement ratio as low as 0.35 are practicable and indeed desirable. The pH of pore solution in high-alumina cement paste is between 11.4 and 12.5.[2.8]

Resistance to chemical attack

As mentioned earlier, high-alumina cement was first developed to resist sulfate attack, and it is indeed highly satisfactory in this respect. This resistance to sulfates is due to the absence of $Ca(OH)_2$ in hydrated high-alumina cement and also to the protective influence of the relatively inert alumina gel formed during hydration.[2.16] However, lean mixes are very much less resistant to sulfates.[2.6] Also, the chemical resistance decreases drastically after conversion (see p. 95).

High-alumina cement is not attacked by CO_2 dissolved in pure water. The cement is not acid-

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resisting but it can withstand tolerably well very dilute solutions of acids (pH greater than about 4) found in industrial effluents, but not of hydrochloric, hydrofluoric or nitric acids. On the other hand, caustic alkalis, even in dilute solutions, attack high-alumina cement with great vigour by dissolving the alumina gel. The alkalis may have their origin outside (e.g. by percolation through Portland cement concrete) or in the aggregate. The behaviour of this cement in the presence of many agents has been studied by Hussey and Robson.[2.16]

It may be noted that, although high-alumina cement stands up extremely well to sea water, this water should not be used as mixing water; the setting and hardening of the cement are adversely affected, possibly because of the formation of chloroaluminates. Likewise, calcium chloride must never be added to high-alumina cement.

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Physical properties of high-alumina cement

A feature of high-alumina cement is its very high rate of strength development. About 80 per cent of its ultimate strength is achieved at the age of 24 hours, and even at 6 to 8 hours the concrete is strong enough for the side formwork to be struck and for the preparation for further concreting to take place. Concrete made with high-alumina cement, at a content of 400 kg/m^3 (or 680 lb/yd^3) and a water/cement ratio of 0.40, at 25 °C (77 °F), can reach a compressive strength (measured on cubes) of about 30 MPa (or 4500 psi) at 6 hours, and more than 40 MPa (or 6000 psi) at 24 hours. The high rate of gain of strength is due to the rapid hydration, which in turn means a high rate of heat development. This can be as high as 38 J/g per hour (9 cal/g per hour) whereas for rapid-hardening Portland cement the rate is never higher than 15 J/g per hour (3.5 cal/g per hour). However, the *total* heat of hydration is about the same for both types of cement.

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It should be stressed that the rapidity of hardening is not accompanied by rapid setting. In fact, high-alumina cement is slow setting but the final set follows the initial set more rapidly than is the case in Portland cement. Typical values for high-alumina cement are: initial set at $2\frac{1}{2}$ hours, and final set, 30 minutes later. Of the compounds present in the high-alumina cement, $C_{12}A_7$ sets in a few minutes, whereas CA is considerably more slow-setting, so that the higher the C: A ratio in the cement the more rapid the set. On the other hand, the higher the glass content of the cement the slower the set. It is likely that, because of its rapid setting properties, $C_{12}A_7$ is responsible for the loss of workability of many high-alumina cement concretes, which takes place within 15 or 20 minutes of mixing. Temperatures between 18 and 30 °C (64 to 86 °F) slow down the setting but, above about 30 °C (86 °F), the setting is rapidly accelerated; the reasons for this anomalous behaviour are not clear.[2.40]

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The setting time of high-alumina cement is greatly affected by the addition of plaster, lime, Portland cement and organic matter and for this reason no additives should be used.

In the case of Portland cement–high-alumina cement mixtures, when either cement constitutes between 20 and 80 per cent of the mixture, flash set may occur. Typical data[2.81] are shown in Fig. 2.4 but actual values vary for different cements, and trial tests should be made with any given cements. When the Portland cement content is low, the accelerated setting is due to the formation of a hydrate of C_4A by the addition of lime from the Portland cement to calcium aluminate from the high-alumina cement. When the high-alumina cement content is low, gypsum contained in the Portland cement reacts with hydrated calcium aluminates, and as a consequence the now non-retarded Portland cement may exhibit a flash set.

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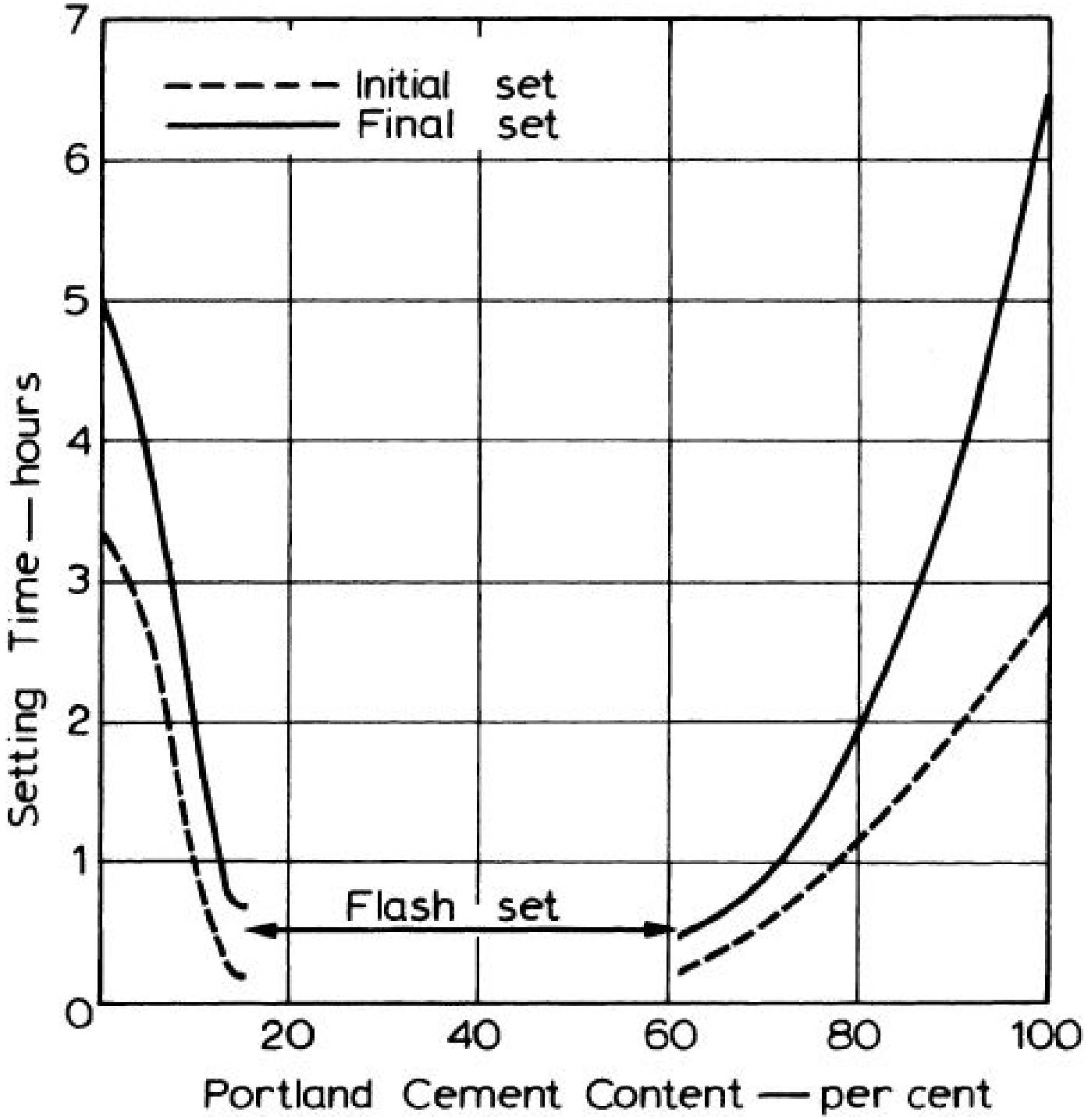


Fig. 2.4. Setting time of Portland-high-alumina cement mixtures[2.81]

Mixtures of the two cements in suitable proportions are used when rapid setting is of vital

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importance, e.g. for stopping the ingress of water, or for temporary construction between the tides, but the ultimate strength of such pastes is quite low except when the high-alumina cement content is very high. However, the use of high-alumina cement for the purpose of shortening the setting time of concrete made with Portland cement is discouraged by ACI 517.2R-87 (Revised 1991).[2.43] To accelerate the setting of high-alumina cement, lithium salts can be used.[2.57]

Because of the rapid setting just described, in construction it is essential to make sure that the two cements do not accidentally come in contact with one another. Thus, placing concrete made with one type of cement against concrete made with the other must be delayed by at least 24 hours if high-alumina cement was cast first, or 3 to 7 days if the earlier concrete was made with Portland cement. Contamination through plant or tools must also be avoided.

It may be noted that, for equal mix proportions, high-alumina cement produces a somewhat

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more workable mix than when Portland cement is used. This may be due to the lower total surface area of high-alumina cement particles, which have a 'smoother' surface than Portland cement particles, because high-alumina cement is produced by complete fusion of the raw materials. On the other hand, superplasticizers do not give good mobility and also adversely affect strength.[2.74]

Creep of high-alumina cement concrete has been found to differ little from the creep of Portland cement concretes when the two are compared on the basis of the stress/strength ratio.[2.22]

Conversion of high-alumina cement

The high strength of high-alumina cement concrete referred to on p. 93 is reached when the hydration of CA results in the formation of CAH_{10} with a small quantity of C_2AH_8 and of alumina gel (Al_2O_3.aq). The hydrate CAH_{10} is, however, chemically unstable both at higher and normal temperatures and becomes transformed into

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C_3AH_6 and alumina gel. This change is known as conversion, and, because the symmetry of the crystal systems is pseudo-hexagonal for the decahydrate and cubic for the sesquihydrate, one can refer to it as the change from the hexagonal to cubic form.

An important feature of hydration of high-alumina cement is that, at higher temperatures, only the cubic form of the calcium aluminate hydrate can exist; at room temperature, either form can exist, but the hexagonal crystals *spontaneously*, albeit slowly, convert to the cubic form. Because they undergo a spontaneous change, the hexagonal crystals can be said to be unstable at room temperature, the final product of the reactions of hydration being the cubic form. Higher temperature speeds up the process; when the periods of exposure to a higher temperature are intermittent, their effect is cumulative.[2.18] This then is conversion: an unavoidable change of one form of calcium aluminate hydrate to another, and it is only reas-

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onable to add that this type of change is not an uncommon phenomenon in nature.

Before discussing the significance of conversion, we should briefly describe the reaction. Conversion both of CAH_{10} and of C_2AH_8 proceeds direct; for instance:

$$3CAH_{10} \rightarrow C_3AH_6 + 2AH_3 + 18H.$$

It should be noted that, although water appears as a product of the reaction, conversion can take place only in the presence of water and not in desiccated concrete because redissolving and reprecipitation are involved. As far as neat cement paste is concerned, it has been found[2.46] that, in sections thicker than 25 mm, the interior of the hydrating cement has an equivalent relative humidity of 100 per cent regardless of the environmental humidity, so that conversion can take place. The influence of the ambient humidity is thus only on concrete near the surface.

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The cubic product of conversion, C_3AH_6, is stable in a solution of calcium hydroxide at 25 °C but reacts with a mixed $Ca(OH)_2$–$CaSO_4$ solution to form $3CaO.Al_2O_3.3CaSO_4.31H_2O$ both at 25 °C and at higher temperatures.[2.47]

The degree of conversion is estimated from the percentage of C_3AH_6 present as a proportion of the sum of the cubic and hexagonal hydrates taken together, i.e. the degree of conversion (per cent) is

$$\frac{\text{mass of } C_3AH_6}{\text{mass of } C_3AH_6 + \text{mass of } CAH_{10}} \times 100.$$

The relative masses of the compounds are derived from the measurements of endothermic peaks in a differential-thermal analysis thermogram.

However, unless the determination can be made under CO_2-free conditions, there is a risk of decomposition of C_3AH_6 into AH_3. The degree of conversion can be determined also in terms of the latter compound because, fortuitously, the

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masses of C_3AH_6 and AH_3 produced in conversion are not very different. Thus we can write: the degree of conversion (per cent) is

$$\frac{\text{mass of } AH_3}{\text{mass of } AH_3 + \text{mass of } CAH_{10}} \times 100.$$

While the two expressions do not give exactly the same result, at high degrees of conversion the difference is not significant. Most laboratories report the result to the nearest 5 per cent. Concrete which has converted about 85 per cent would be considered as fully converted.

The rate of conversion depends on temperature; some actual data are shown in Table 2.9. The relation[2.46] between the time necessary for one-half of the CAH_{10} to convert and the temperature of storage of 13 mm ($\frac{1}{2}$ in.) cubes of neat cement paste with a water/cement ratio of 0.26 is shown in Fig. 2.5. It is likely that, for the more porous concretes of practical mix proportions, the periods are much shorter as full conversion has been observed after some 20 years at 20 °C or there-

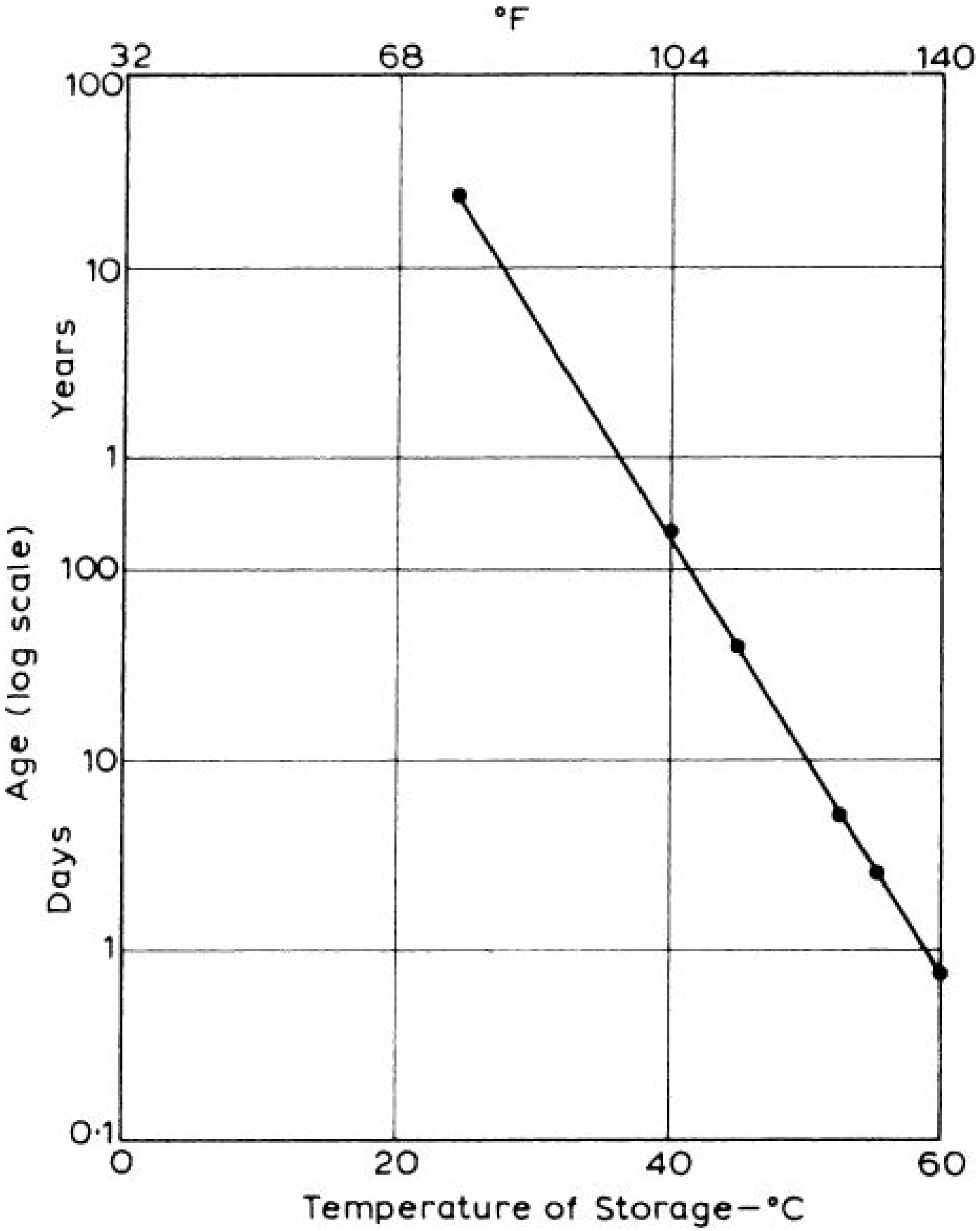
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abouts. Thus data on neat cement pastes with very low water/cement ratios should be used circumspectly, but they are nevertheless of scientific interest.

Table 2.9. Development of Conversion with Age[2.51] (Crown copyright)

Range of free water/cement ratio	*Temperature of storage °C (°F)*	*Average degree of conversion (per cent) at the age of:*				
		28 days	*3 months*	*1 year*	*5 years*	*8½ years*
0.27–0.40	18 (64)	20	20	25	30	45
	38 (100)	55	85	80	85	90
0.42–0.50	18 (64)	20	20	25	40	50
	38 (100)	60	80	80	80	90
0.52–0.67	18 (64)	20	20	25	50	65
	38 (100)	65	80	80	85	90

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Fig. 2.5. Time for half-conversion of neat high-alumina cement pastes cured at various temperatures (13 mm ($\frac{1}{2}$ in.) cubes)[2.46]
(Crown copyright)

The practical interest in conversion lies in the fact that it leads to a loss of strength of high-alumina cement concrete. The explanation of this is in terms of the densification of the calcium aluminate hydrates: typically, the density would be 1.72 g/ml for CAH_{10} and 2.53 for C_3AH_6. Thus, under conditions such that the overall dimensions of the body are constant (as is the case in set cement paste), conversion, with the concomitant internal release of water, results in an increase in the porosity of the paste. Numerous proofs of this are available, a particularly convincing one being the measurement of air permeability of converted compared with unconverted high-alumina cement concrete[2.48] (see Fig. 2.6).

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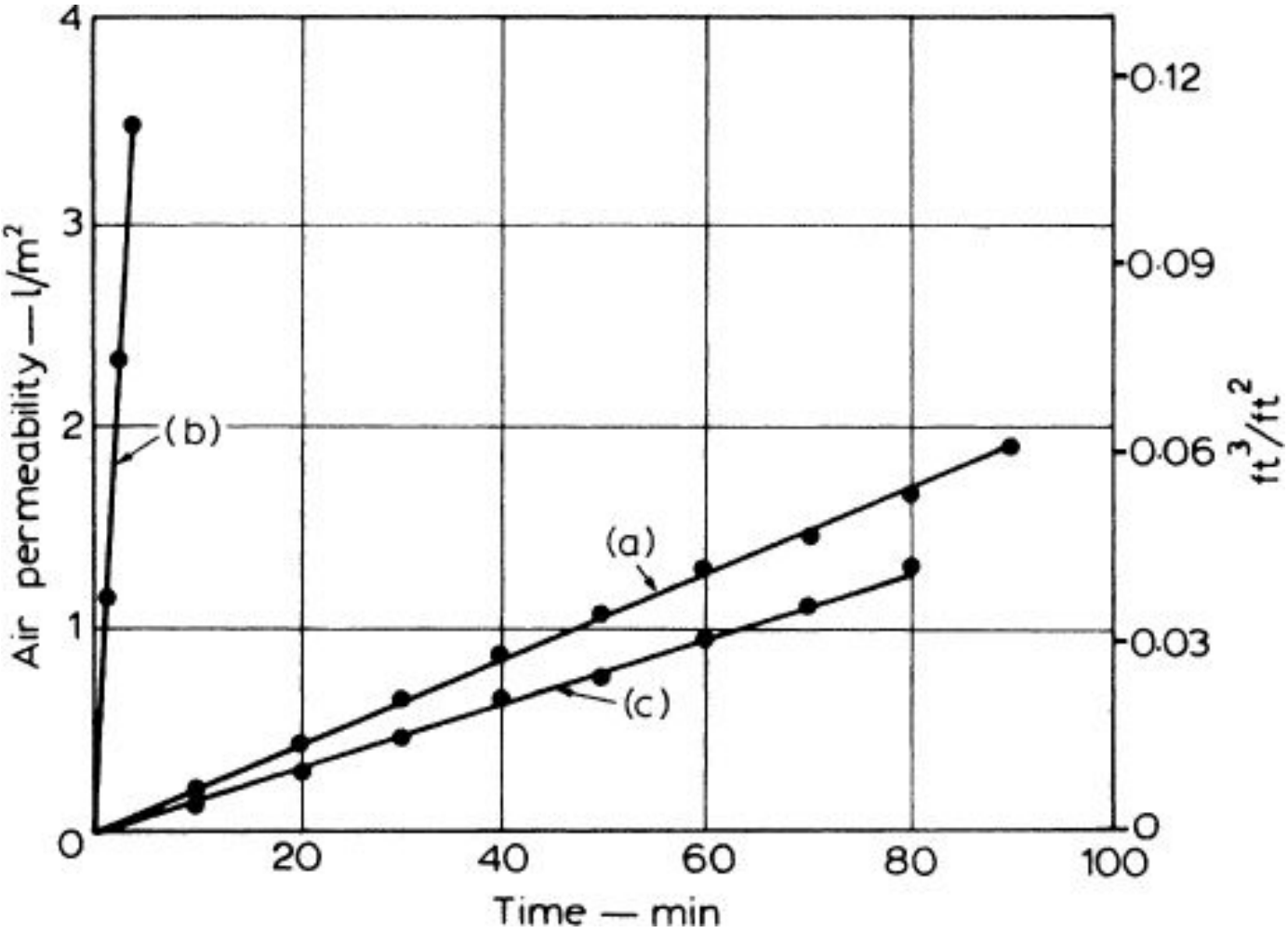


Fig. 2.6. Air flow through concrete: (*a*) unconverted high-alumina cement concrete; (*b*) converted high-alumina cement concrete; (*c*) Portland cement concrete (temperature 22 to 24 °C (72 to 75 °F), relative humidity 36 to 41 per cent; pressure difference 10.7 kPa)[2.48]

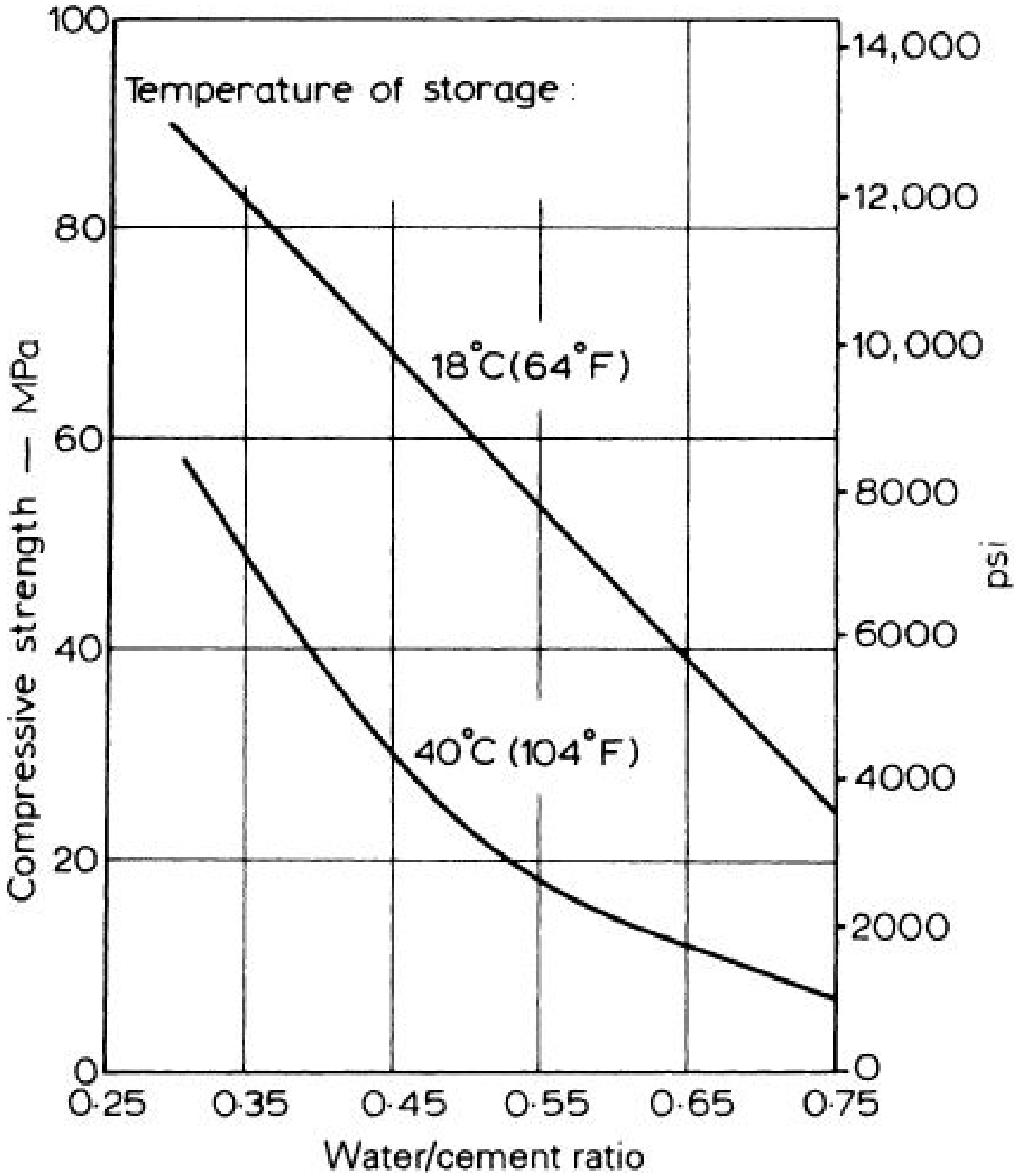
As shown on p. 279, the strength of hydrated cement paste or of concrete is very strongly affected by its porosity; porosity of 5 per cent can re-

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duce the strength by more than 30 per cent, and a 50 per cent reduction in strength would be caused by a porosity of about 8 per cent. This magnitude of porosity of concrete can be induced by conversion in high-alumina cement concrete.

It follows that, because conversion takes place in concretes and mortars of any mix proportions, they lose strength when exposed to a higher temperature, and the general pattern of the strength loss versus time is similar in all cases. However, the degree of loss is a function of the water/cement ratio of the mix, as shown in Fig. 2.7. The mix proportions and percentage loss are given in Table 2.10. It is clear that the loss, either in megapascals (or psi) or as a fraction of the strength of cold-cured concrete, is smaller in mixes with low water/cement ratios than in mixes with high water/cement ratios.[2.33]

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Fig. 2.7. Influence of the water/cement ratio on the strength of high-alumina cement concrete cubes cured in water at 18 and 40 °C for 100 days

Table 2.10. Influence of Water/Cement Ratio on Loss of Strength on Conversion

Cement	*Water/ cement ratio*	*Aggregate/ cement ratio**	*1-day strength at 18 °C (64 °F)† MPa*	*Strength of converted concrete as a percentage of strength at 18 °C (64 °F)*
A	0.29	2.0	91.0	62
	0.35	3.0	84.4	61
	0.45	4.0	72.1	26
	0.65	6.2	42.8	12
B	0.30	2.1	92.4	63
	0.35	3.0	80.7	60
	0.45	4.0	68.6	43
	0.65	6.2	37.2	30
	0.75	7.2	24.5	29

*Maximum size of aggregate 9.5 mm ($\frac{3}{8}$ in.).
†76 mm (3 in.) cubes.

It may be observed that the shape of the strength versus water/cement ratio curves for storage at 18 °C (Fig. 2.7) is dissimilar from the usual curves for Portland cement concretes. This is characteristic of concretes made with high-alumina cement, and has been confirmed also for

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cylinders both of standard size[2.17] and other height/diameter ratios.[2.22]

The values shown in Fig. 2.7 are no more than typical, and clearly some variation would be found with different cements, but the pattern of behaviour is the same in all cases. It is important to note that the residual strength of mixes with moderate and high water/cement ratios, say over 0.5, may be so low as to be unacceptable for most structural purposes.

A brief historical note on the structural use of high-alumina cement may be in order. Because of the very high early strength of concrete made with high-alumina cement, it was used in the manufacture of prestressed concrete units. Neville's warnings[2.33] about the dangers consequent upon conversion were ignored, but they were shown to be true. Structural failures occurred in England in the early 1970s, and consequently, all structural use of high-alumina cement was withdrawn from British codes. In most other countries, too, high-alumina cement is not used in

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structural concrete. Nevertheless, failures of old high-alumina cement concrete occurred in Spain in the early 1990s. The European standard BS EN 14647-2006 deals with HAC but it contains an annex giving advice on *use* of HAC; in my opinion, such advice on structural use of HAC from a body writing a material specification is outside its remit.

Arguments to the effect that, at a water/cement ratio not exceeding 0.40 and a cement content of not less than 400 kg/m^3 (680 lb/yd^3), the strength after conversion is still adequate are not convincing. To begin with, under practical conditions of manufacture of concrete, it is not possible to *guarantee* that the specified water/cement ratio will not be occasionally exceeded by 0.05 or even by 0.10; this has been repeatedly demonstrated[2.49] (see also p. 744). It should be noted that the strength of converted high-alumina cement is more sensitive to changes in the water/cement ratio than before conversion; this is illustrated in Fig. 2.8 based on the data of George.[2.50]

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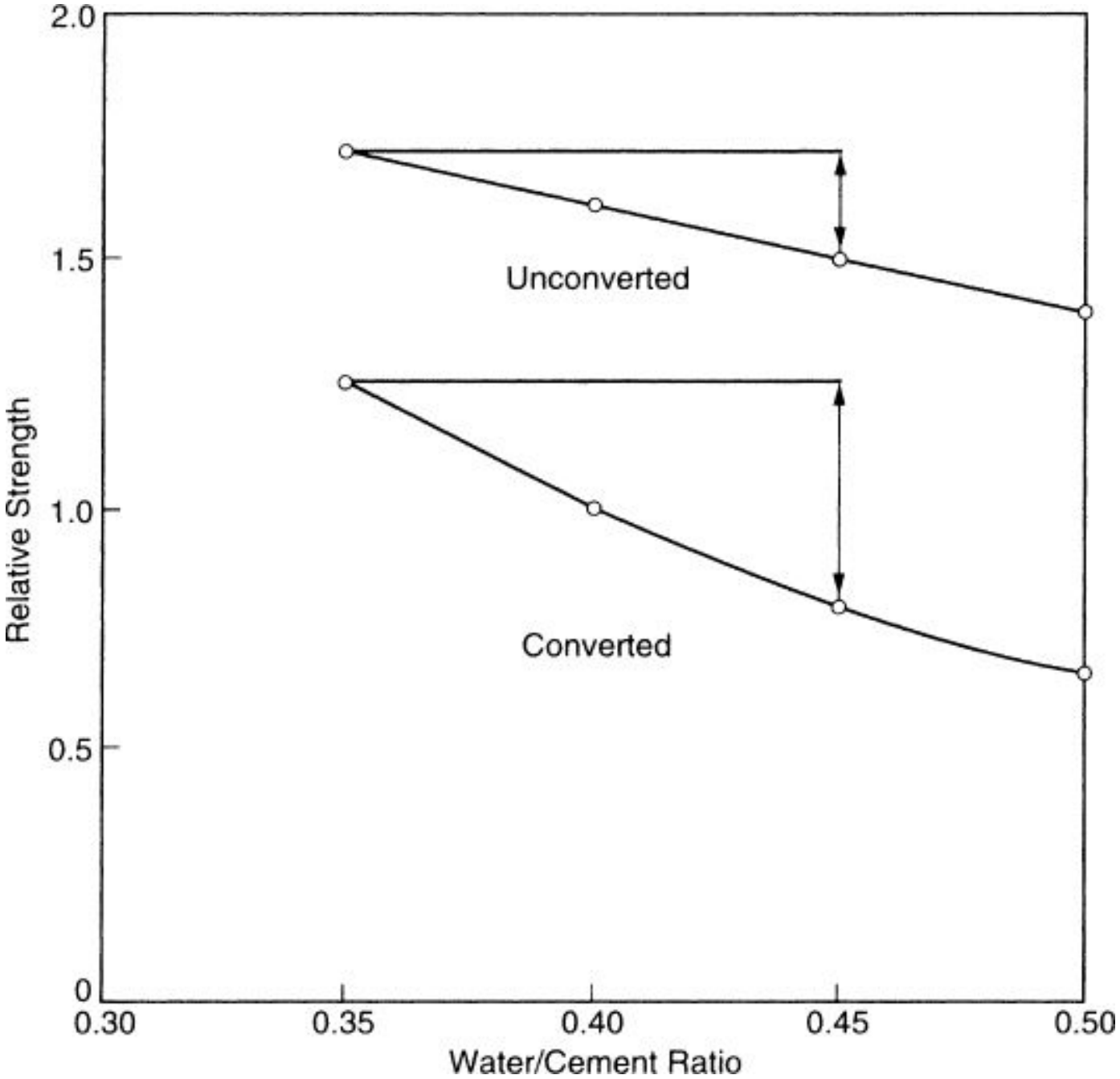


Fig. 2.8. Influence of the water/cement ratio upon the strength of high-alumina cement concrete, before and after conversion, relative to the strength after conversion of concrete with a water/cement ratio of 0.4 (based on ref. 2.50)

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Under certain moisture conditions, following conversion, hydration of the hitherto unhydrated cement leads to some increase in strength. However, the conversion of the newly formed hexagonal hydrates leads to a renewed and continuing loss of strength. Thus, the strength drops below the 24-hour value. This occurs at the age of 8 to 10 years in concrete with a water/cement ratio of 0.4, and even later if the water/cement ratio is lower.[2.78] In any case, from the structural point of view, it is the lowest strength at *any* time in the life of the concrete that is critical.

The loss of strength is lower under dry conditions, but in concrete of substantial thickness the conditions are not dry. An indirect proof that within a large mass of rich concrete there is present adequate water for chemical reaction is afforded by Hobbs[2.75] who found that, in concrete with a Portland cement content of 500 to 550 kg/m^3, kept sealed, there was enough water available for expansive alkali–silica reaction to take place. Collins and Gutt[2.78] reported that wet,

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or perhaps even occasionally wet, concrete may have a strength 10 to 15 MPa lower than dry concrete. Occasional wetting, by accident or, for instance, applied to extinguish a fire may occur in almost every building.

These results of Collins and Gutt[2.78] were the outcome of an investigation at the Building Research Establishment, started in 1964, which confirmed in essence the statements and extrapolations made in 1963 by Neville.[2.33] Menzies[2.84] described the advice given in earlier codes of practice as an error. The saga of problems with use of HAC is given in ref.[2.85]

The second argument concerning the structural use of converted high-alumina cement concrete, even if it is of adequate strength, is that converted high-alumina cement paste is more porous and therefore more liable to chemical attack than before conversion. This applies in particular to sulfate attack. If the sulfate ions penetrate through the outer protective skin of high-alumina cement concrete (associated with drying

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out), expansive reaction with C_3AH_6 takes place;[2.79] it is only unconverted CAH_{10} that is inert with respect to sulfates.

Moreover, chemical attack may then produce a further loss in strength,[2.81] but the chemical reactions involved necessitate the presence of water. As mentioned on p. 93, percolating water can bring with it sodium or potassium hydroxide, which accelerate conversion, in addition to decomposition of the products of hydration. If carbon dioxide is also present, calcium carbonate is formed, and the alkali hydroxide is regenerated to attack further the hydrated cement paste.[2.82] Under some circumstances, complete decomposition of calcium aluminate hydrates can result. The reactions are written as follows:[2.83]

$$K_2CO_3 + CaO.Al_2O_3.aq \rightarrow CaCO_3 + K_2O.Al_2O_3$$

$$CO_2 + K_2O.Al_2O_3 + aq \rightarrow K_2CO_3 + Al_2O_3.3H_2O.$$

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Thus, because the alkalis are only a carrier, the overall reaction can be written as:

$$CO_2 + CaO.Al_2O_3.aq \rightarrow CaCO_3 + Al_2O_3.3H_2O.$$

It can be said, therefore, that high-alumina cement undergoes carbonation, but its nature is not the same as that of Portland cement (see p. 499).

British Standards do not allow structural use of high-alumina cement. In the United States, the Strategic Highway Research Program[2.37] decided not to consider high-alumina cement concrete because of the consequences of conversion. The cement has, however, specialized applications. One of these is in mines for roof support. Here, a two-slurry system containing high-alumina cement, calcium sulfate, lime and appropriate admixtures leads to the development of ettringite, which has a substantial early strength:[2.72]

$$3CA + 3C\bar{S}H_2 + 2C + 26H \rightarrow C_6A\bar{S}_3H_{32}.$$

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$\bar{S}$ denotes SO_3

Madjumdar *et al.*[2.73] developed a blended high-alumina and ground granulated blastfurnace slag cement (in equal proportions by mass) in an attempt to avoid conversion problems. The slag removes lime from the solution so that the formation of C_3AH_6 is hindered and C_2ASH_8 is the main hydrate formed in the longer term. However, such a blended cement does not develop the very high early strength which is characteristic of high-alumina cement – its apparent *forte*; this may be the reason why this blended cement has not been commercially produced.

Refractory properties of high-alumina cement

High-alumina cement concrete is one of the foremost refractory materials, but it is important to be clear about its performance over the full temperature range. Between room temperature and about 500 °C, high-alumina cement concrete

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loses strength to a greater extent than Portland cement concrete; then up to 800 °C the two are comparable; but above about 1000 °C high-alumina cement gives excellent performance. Figure 2.9 shows the behaviour of high-alumina cement concrete made with four different aggregates at temperatures up to 1100 °C.[2.52] The minimum strength varies between 5 and 26 per cent of the original value but, depending on the type of aggregate, above 700 to 1000 °C, there is a gain in strength due to the development of the ceramic bond. This bond is established by solid reactions between the cement and the fine aggregate, and increases with an increase in temperature and with the progress of the reactions.

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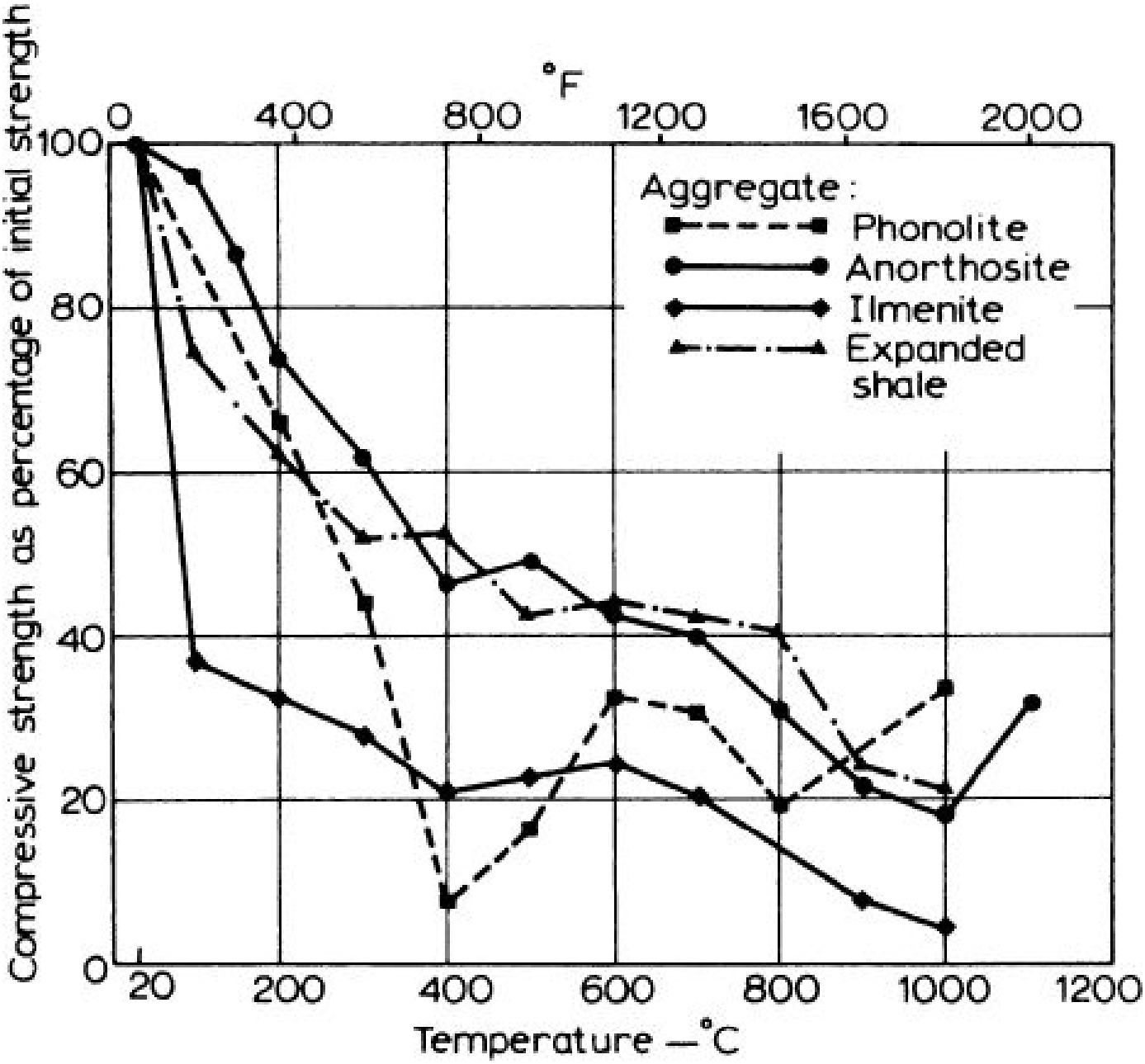


Fig. 2.9. Strength of high-alumina cement concretes made with different aggregates as a function of temperature[2.52]

As a result, high-alumina cement concrete can withstand very high temperatures: with crushed firebrick aggregate up to about 1350 °C, and with

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special aggregates, such as fused alumina or carborundum, up to 1600 °C. A temperature as high as 1800 °C can be withstood over prolonged periods of time by concrete made from special white calcium aluminate cement with fused alumina aggregate. This cement is made using alumina as a raw material and contains 70 to 80 per cent of $A1_2O_3$, 20 to 25 per cent of lime, and only about 1 per cent of iron and silica; the composition of the cement approaches C_3A_5. It is appropriate to mention that the price of such cement is very high.

Refractory concrete made with high-alumina cement has a good resistance to acid attack (e.g. acids in flue gases), the chemical resistance being in fact increased by firing at 900 to 1000 °C.[2.16] The concrete can be brought up to service temperature as soon as it has hardened, that is, it does not have to be pre-fired. While refractory brickwork expands on heating and, therefore, needs expansion joints, high-alumina cement concrete can be cast monolithically, or with butt joints

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only (at 1 to 2 m), to exactly the required shape and size. The reason for this is that the loss of water on first firing results in a contraction approximately equal to the thermal expansion on heating, so that the net dimensional change (depending on aggregate) is small. Upon subsequent cooling, for instance, during the shut-down of a plant, butt joints would open slightly due to the thermal contraction but they would close up again on re-heating. It is worth noting that refractory high-alumina cement concrete can withstand a considerable thermal shock. Refractory linings can be made by shotcreting high-alumina cement mortar.

For insulating purposes, when temperatures up to about 950 °C are expected, lightweight concrete can be made with high-alumina cement and lightweight aggregate. Such concrete has a density of 500 to 1000 kg/m^3 (30 to 60 lb/ft^3) and a thermal conductivity of 0.21 to 0.29 J/m^{2}s °C/m (0.12 to 0.17 Btu/ft^{2}h °F/ft).

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Chapter 3. Properties of aggregate

Because at least three-quarters of the volume of concrete is occupied by aggregate, it is not surprising that its quality is of considerable importance. Not only may the aggregate limit the strength of concrete, as aggregate with undesirable properties cannot produce strong concrete, but the properties of aggregate greatly affect the durability and structural performance of concrete.

Aggregate was originally viewed as an inert material dispersed throughout the cement paste largely for economic reasons. It is possible, however, to take an opposite view and to look on aggregate as a building material connected into a cohesive whole by means of the cement paste, in a manner similar to masonry construction. In fact, aggregate is not truly inert and its physical, thermal, and sometimes also chemical properties influence the performance of concrete.

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Aggregate is cheaper than cement and it is, therefore, economical to put into the mix as much of the former and as little of the latter as possible. But economy is not the only reason for using aggregate: it confers considerable technical advantages on concrete, which has a higher volume stability and better durability than hydrated cement paste alone.

General classification of aggregates

The size of aggregate used in concrete ranges from tens of millimetres down to particles less than one-tenth of a millimetre in cross-section. The maximum size actually used varies but, in any mix, particles of different sizes are incorporated, the particle size distribution being referred to as grading. In making low-grade concrete, aggregate from deposits containing a whole range of sizes, from the largest to the smallest, is sometimes used; this is referred to as *all-in* or *pit-run* aggregate. The alternative, always used in the manufacture of good quality concrete, is to ob-

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tain the aggregate in at least two size groups, the main division being between *fine aggregate*, often called sand (for example, in BS EN 12620 : 2002), not larger than 4 mm or $\frac{3}{16}$ in., and *coarse aggregate*, which comprises material at least 5 mm or $\frac{3}{16}$ in. in size. In the United States, the division is made at No. 4 ASTM sieve, which is 4.75 mm ($\frac{3}{16}$ in.) in size (see Table 3.14). More will be said about grading later, but this basic division makes it possible to distinguish in the ensuing description between fine and coarse aggregate. It should be noted that the use of the term aggregate (to mean coarse aggregate) in contradistinction to sand is not correct.

Natural sand is generally considered to have a lower size limit of 70 or 60 μm. Material between 60 μm and 2 μm is classified as silt, and particles smaller still are termed clay. Loam is a soft deposit consisting of sand, silt, and clay in about equal proportions. Although the content of particles smaller than 75 μm is usually reported globally, the influence of silt and of clay on the

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properties of the resultant concrete is often significantly different not only because these particles differ in size but also in composition. Methods of determining the proportion of material smaller than 75 μm and 20 μm, respectively, are prescribed in BS 812 : 103.1 : 1985 (2000) and BS 812 : 103.2 (2000).

All natural aggregate particles originally formed a part of a larger parent mass. This may have been fragmented by natural processes of weathering and abrasion or artificially by crushing. Thus, many properties of the aggregate depend entirely on the properties of the parent rock, e.g. chemical and mineral composition, petrological character, specific gravity, hardness, strength, physical and chemical stability, pore structure, and colour. On the other hand, there are some properties possessed by the aggregate but absent in the parent rock: particle shape and size, surface texture, and absorption. All these properties may have a considerable influence on

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the quality of the concrete, either fresh or in the hardened state.

It is only reasonable to add, however, that, although these different properties of aggregate *per se* can be examined, it is difficult to define a good aggregate other than by saying that it is an aggregate from which good concrete (for the given conditions) can be made. While aggregate whose properties all appear satisfactory will always make good concrete, the converse is not necessarily true and this is why the criterion of performance in concrete has to be used. In particular, it has been found that aggregate may appear to be unsatisfactory on some count but no trouble need be experienced when it is used in concrete. For instance, a rock sample may disrupt on freezing but need not do so when embedded in concrete, especially when the aggregate particles are well covered by a hydrated cement paste of low permeability. However, aggregate considered poor in more than one respect is unlikely to make a satisfactory concrete, so that tests on aggregate alone

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are of help in assessing its suitability for use in concrete.

Classification of natural aggregates

So far, we have considered only aggregate formed from naturally occurring materials, and the present chapter deals almost exclusively with this type of aggregate. Aggregate can, however, also be manufactured from industrial products: because these artificial aggregates are generally either heavier or lighter than ordinary aggregate they are considered in Chapter 13. Aggregates made from waste are referred to on p. 696.

A further distinction can be made between aggregate reduced to its present size by natural agents and crushed aggregate obtained by a deliberate fragmentation of rock.

From the petrological standpoint, the aggregates, whether crushed or naturally reduced in size, can be divided into several groups of rocks having common characteristics. The classification of BS 812 : 1 : 1975 is most convenient and

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is given in Table 3.1. The group classification does not imply suitability of any aggregate for concrete-making: unsuitable material can be found in any group, although some groups tend to have a better record than others. It should also be remembered that many trade and customary names of aggregates are in use, and these often do not correspond to the correct petrographic classification. Rock types commonly used for aggregates are listed in BS 812 : 102 : 1989, and BS 812 : 104 : 1994 (2000) covers the methods of petrographic examination. BS 812 has been replaced by BS EN 932 and 933.

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Table 3.1. Classification of Natural Aggregates According to Rock Type (BS 812 : 1 : 1975)

Basalt group	*Flint group*	*Gabbro group*
Andesite	Chert	Basic diorite
Basalt	Flint	Basic gneiss
Basic porphyrites		Gabbro
Diabase		Hornblende-rock
Dolerites of all kinds including theralite and teschenite		Norite
		Peridotite
Epidiorite		Picrite
Lamprophyre		Serpentinite
Quartz-dolerite		
Spilite		
Granite group	*Gritstone group (including fragmental volcanic rocks)*	*Hornfels group*
Gneiss	Arkose	Contact-altered rocks of all kinds except marble
Granite	Greywacke	
Granodiorite	Grit	
Granulite	Sandstone	
Pegmatite	Tuff	
Quartz-diorite		
Syenite		
Limestone group	*Porphyry group*	*Quartzite group*
Dolomite	Aplite	Ganister
Limestone	Dacite	Quartzitic sandstones
Marble	Felsite	Re-crystallized quartzite
	Granophyre	
	Keratophyre	
	Microgranite	
	Porphyry	
	Quartz-porphyrite	
	Rhyolite	
	Trachyte	
Schist group		
Phyllite		
Schist		
Slate		
All severely sheared rocks		

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ASTM Standard C 294-05 gives a description of some of the more common or important minerals found in aggregates. Mineralogical classification is of help in recognizing properties of aggregate but cannot provide a basis for predicting its performance in concrete as there are no minerals universally desirable and few invariably undesirable ones. The ASTM classification of minerals is summarized below:

Silica minerals (quartz, opal, chalcedony, tridymite, cristobalite)
Feldspars
Ferromagnesian minerals
Micaceous minerals
Clay minerals
Zeolites
Carbonate minerals
Sulfate minerals
Iron sulfide minerals
Iron oxides

The details of petrological and mineralogical methods are outside the scope of this book, but

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it is important to realize that geological examination of aggregate is a useful aid in assessing its quality and, in particular, in comparing a new aggregate with one for which service records are available. Furthermore, adverse properties, such as the presence of some unstable forms of silica, can be detected. Even small amounts of minerals or of rocks may have a large influence on the quality of aggregate. In the case of artificial aggregates, the influence of manufacturing methods and of processing can also be studied. Detailed information on aggregate for concrete can be found in ref. 3.38.

Sampling

Tests of various properties of aggregate are perforce performed on samples of the material and, therefore, the results of the tests apply, strictly speaking, only to the aggregate in the sample. Since, however, we are interested in the bulk of the aggregate as supplied, or as available for supply, we should ensure that the sample is typical

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of the average properties of the aggregate. Such a sample is said to be representative and, to obtain it, certain precautions in procuring the sample have to be observed.

No detailed procedures can, however, be laid down because the conditions and situations involved in taking samples in the field can vary widely from case to case. Nevertheless, an intelligent experimenter can obtain reliable results if he or she bears in mind at all times that the sample taken is to be representative of the bulk of the material considered. An instance of such care would be to use a scoop rather than a shovel so as to prevent rolling off of particles of some sizes when the shovel is lifted.

The main sample is made up of a number of portions drawn from different parts of the whole. The minimum number of these portions, called increments, is ten, and they should add up to a mass not less than that given in Table 3.2 for particles of different sizes, as prescribed by BS 812 : 102 : 1989 (replaced by BS EN 932-1 :

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1997). If, however, the source from which the sample is being obtained is variable or segregated, a larger number of increments should be taken and a larger sample ought to be dispatched for testing. This is particularly the case in stockpiles when increments have to be taken from all parts of the pile, not only below its surface but also from the centre of the pile.

Table 3.2. Minimum Mass of Samples for Testing (BS 812 : 102 : 1989)

Maximum particle size present in substantial proportion mm	*Minimum mass of sample dispatched for testing kg*
28 or larger	50
Between 5 and 28	25
5 or smaller	13

It is clear from Table 3.2 that the main sample can be rather large, particularly when large-size aggregate is used, and so the sample has to be reduced before testing. At all stages of reduction, it is necessary to ensure that the representative character of the sample is retained so that the ac-

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tual test sample has the same properties as the main sample and *ipso facto* as the bulk of the aggregate.

There are two ways of reducing the size of a sample, each essentially dividing it into two similar parts: quartering and riffling. For quartering, the main sample is thoroughly mixed and, in the case of fine aggregate, dampened in order to avoid segregation. The material is heaped into a cone and then turned over to form a new cone. This is repeated twice, the material always being deposited at the apex of the cone so that the fall of particles is evenly distributed round the circumference. The final cone is flattened and divided into quarters. One pair of diagonally opposite quarters is discarded, and the remainder forms the sample for testing or, if still too large, can be reduced by further quartering. Care must be taken to include all fine material in the appropriate quarter.

As an alternative, the sample can be split into halves using a riffler (Fig. 3.1). This is a box with

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a number of parallel vertical divisions, alternate ones discharging to the left and to the right. The sample is discharged into the riffler over its full width, and the two halves are collected in two boxes at the bottom of the chutes on each side. One half is discarded, and riffling of the other half is repeated until the sample is reduced to the desired size. BS EN 12420 : 2000 describes a typical riffler. Riffling gives less variable results than quartering.

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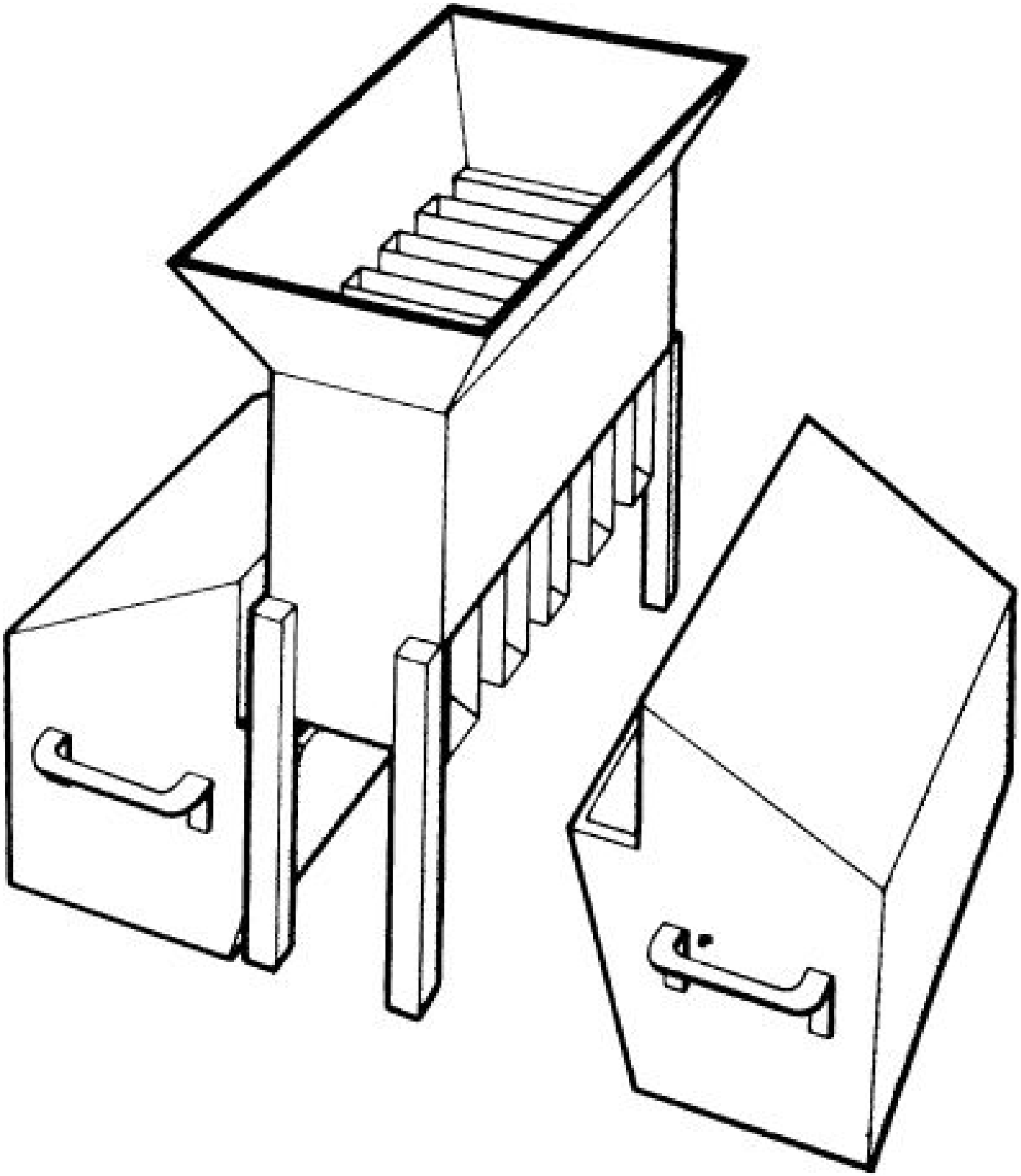


Fig. 3.1. Riffler

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Particle shape and texture

In addition to the petrological character of aggregate, its external characteristics are of importance, in particular the particle shape and surface texture. The shape of three-dimensional bodies is rather difficult to describe, and it is, therefore, convenient to define certain geometrical characteristics of such bodies.

Roundness measures the relative sharpness or angularity of the edges and corners of a particle. Roundness is controlled largely by the strength and abrasion resistance of the parent rock and by the amount of wear to which the particle has been subjected. In the case of crushed aggregate, the particle shape depends not only on the nature of the parent material but also on the type of crusher and its reduction ratio, i.e. the ratio of the size of material fed into the crusher to the size of the finished product. A convenient broad classification of roundness is that of BS 812 : 1 : 1975, given in Table 3.3. The relevant current standard is BS EN 933-4 : 2008.

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Table 3.3. Particle Shape Classification of BS 812-1 : 1975* with Examples

Classification	*Description*	*Examples*
Rounded	Fully water-worn or completely shaped by attrition	River or seashore gravel; desert, seashore and wind-blown sand
Irregular	Naturally irregular, or partly shaped by attrition and having rounded edges	Other gravels; land or dug flint
Flaky	Material of which the thickness is small relative to the other two dimensions	Laminated rock
Angular	Possessing well-defined edges formed at the intersection of roughly planar faces	Crushed rocks of all types; talus; crushed slag
Elongated	Material, usually angular, in which the length is considerably larger than the other two dimensions	—
Flaky and elongated	Material having the length considerably larger than the width, and the width considerably larger than the thickness	—

*Replaced by BS EN 933-3 : 1997

A classification sometimes used in the United States is as follows:

Well-rounded – no original faces left
Rounded – faces almost gone
Subrounded – considerable wear, faces reduced in area
Subangular – some wear but faces untouched
Angular – little evidence of wear

Because the degree of packing of particles, all of one size, depends on their shape, the angular-

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ity of aggregate can be estimated from the proportion of voids in a sample compacted in a prescribed way. British Standard BS 812-1 : 1995 defines the concept of angularity number; this can be taken as 67 minus the percentage of solid volume in a vessel filled with aggregate in a standard manner. The size of particles used in the test must be controlled within narrow limits.

The number 67 in the expression for the angularity number represents the percentage solid volume of the most rounded gravel, so that the angularity number measures the percentage of voids in excess of that in the rounded gravel (i.e. 33). The higher the number the more angular the aggregate, the range for practical aggregate being between 0 and 11. The test for angularity is rarely used.

A development in measurement of angularity of aggregate, both coarse and fine but of single size, is an angularity factor defined as the ratio of the solid volume of loose aggregate to the solid volume of glass spheres of specified grading;[3.41]

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thus, no packing is involved and the attendant error is avoided. Various other indirect methods of determination of the shape of fine aggregate have been critically reviewed by Gaynor and Meininger[3.63] but no generally accepted method is available.

The void content of aggregate can be calculated from the change in the volume of air when a known decrease in pressure is applied; hence, the volume of air, i.e. the volume of interstitial space, can be calculated.[3.52]

A simple proof of the dependence of the percentage of voids on the shape of particles is obtained from Fig. 3.2, based on Shergold's[3.1] data. The sample consisted of a mixture of two aggregates, one angular, the other rounded, in varying proportions, and it can be seen how increasing the proportion of rounded particles decreases the percentage of voids. The volume of voids influences the density of the concrete which can be achieved.

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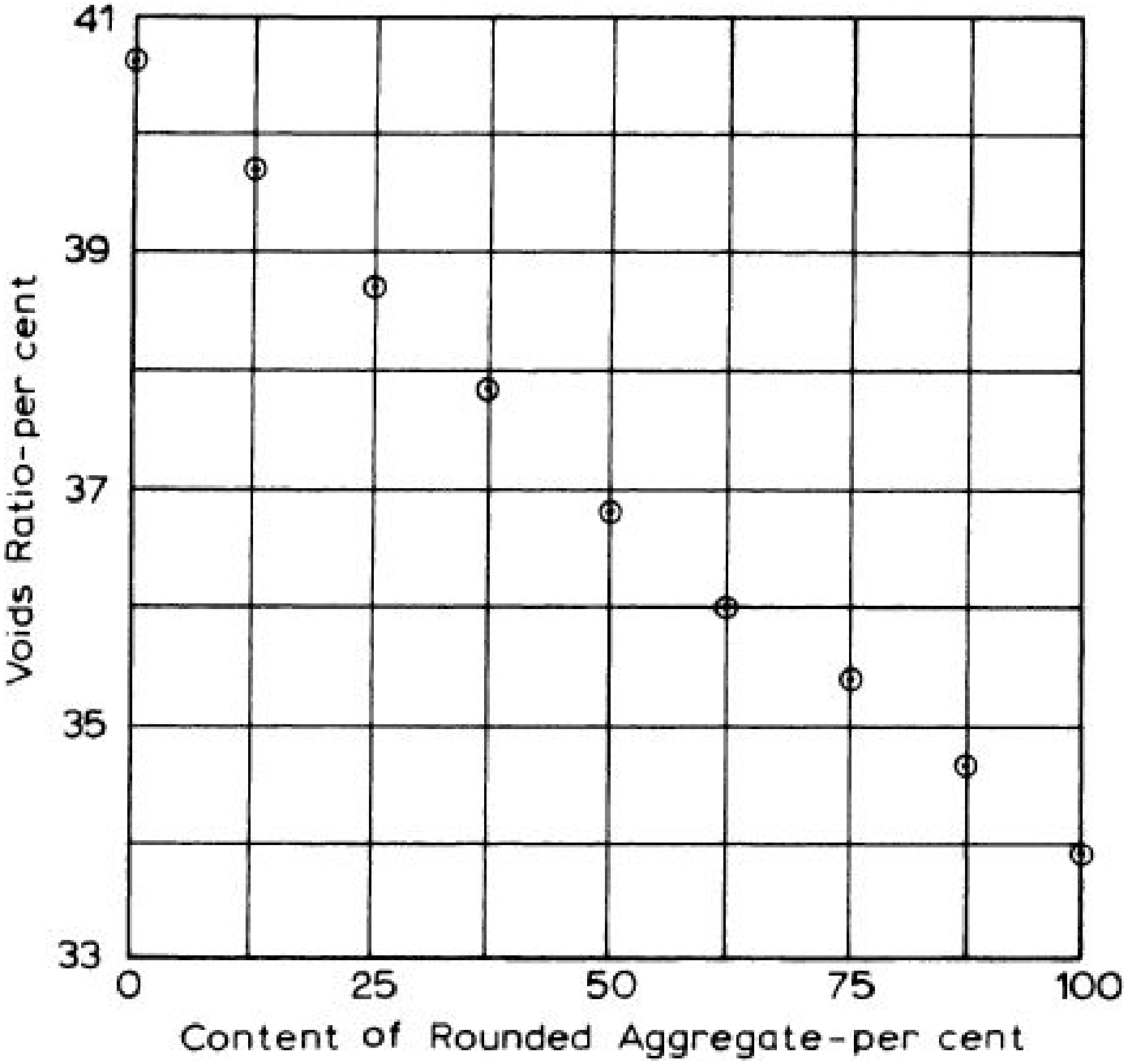


Fig. 3.2. Influence of angularity of aggregate on voids ratio[3.1] (Crown copyright)

Another aspect of the shape of coarse aggregate is its *sphericity*, defined as a function of the ratio of the surface area of the particle to its volume. Sphericity is related to the bedding and

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cleavage of the parent rock, and is also influenced by the type of crushing equipment when the size of particles has been artificially reduced. Particles with a high ratio of surface area to volume are of particular interest as they increase the water demand for a given workability of the concrete mix.

That the shape of fine aggregate particles influences the mix properties is without doubt, angular particles requiring more water for a given workability, but an objective method of measuring and expressing shape is not yet available despite attempts using measurement of the projected surface area and other geometrical approximations.

As far as coarse aggregate is concerned, equidimensional shape of particles is preferred because particles which significantly depart from such a shape have a larger surface area and pack in an anisotropic manner. Two types of particles which depart from equidimensional shape are of interest: elongated and flaky. The latter type can also affect adversely the durability of concrete

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because flaky particles tend to be oriented in one plane, with bleeding water and air voids forming underneath.

The mass of flaky particles expressed as a percentage of the mass of the sample is called the *flakiness index. Elongation index* is similarly defined. Some particles are both flaky and elongated, and are, therefore, counted in both categories.

The classification is made by means of simple gauges described in BS 812-105.1 : 1989 and BS EN 933-3 : 1997. BS EN 12620 : 2002 uses different dimensional ratios. The division is based on the rather arbitrary assumption that a particle is flaky if its thickness (least dimension) is less than 0.6 times the mean sieve size of the size fraction to which the particle belongs. Similarly, a particle whose length (largest dimension) is more than 1.8 times the mean sieve size of the size fraction is said to be elongated. The mean size is defined as the arithmetic mean of the sieve size on which the particle is just retained and the sieve

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size through which the particle just passes. As closer size control is necessary, the sieves considered are not those of the standard concrete aggregate series but: 75.0, 63.0, 50.0, 37.5, 28.0, 20.0, 14.0, 10.0, and 6.30 mm (or about 3, $2\frac{1}{2}$, 2, $1\frac{1}{2}$, 1, $\frac{3}{4}$, $\frac{1}{2}$, $\frac{3}{8}$, and $\frac{1}{4}$ in.) sieves. The flakiness and elongation tests are useful for general assessment of aggregates but they do not adequately describe the particle shape.

The presence of elongated particles in excess of 10 to 15 per cent of the mass of coarse aggregate is generally considered undesirable, but no recognized limits are laid down. British Standard BS 882 : 1992 limits the flakiness index of the coarse aggregate to 50 for natural gravel and to 40 for crushed or partially crushed coarse aggregate. However, for wearing surfaces, lower values of the flakiness index are required. Newer standards do not prescribe absolute limits on flakiness.

Surface texture of the aggregate affects its bond to the cement paste and also influences the

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water demand of the mix, especially in the case of fine aggregate.

The classification of the surface texture is based on the degree to which the particle surfaces are polished or dull, smooth or rough; the type of roughness has also to be described. Surface texture depends on the hardness, grain size and pore characteristics of the parent material (hard, dense and fine-grained rocks generally having smooth fracture surfaces) as well as on the degree to which forces acting on the particle surface have smoothed or roughened it. Visual estimate of roughness is quite reliable but, in order to reduce misunderstanding, the classification of BS 812-1 : 1975, given in Table 3.4, should be followed. This standard has been superseded by BS EN 12620 : 2002. There is no recognized method of measuring the surface roughness but Wright's approach[3.2] is of interest: the interface between the particle and a resin in which it is set is magnified, and the difference between the length of the profile and the length of an unevenness line drawn as

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a series of chords is determined. This is taken as a measure of roughness. Reproducible results are obtained, but the method is laborious and is not widely used.

Table 3.4. Surface Texture of Aggregates (BS 812 : 1 : 1975, with Examples

Group	*Surface texture*	*Characteristics*	*Examples*
1	Glassy	Conchoidal fracture	Black flint, vitreous slag
2	Smooth	Water-worn, or smooth due to fracture of laminated or fine-grained rock	Gravels, chert, slate, marble, some rhyolites
3	Granular	Fracture showing more or less uniform rounded grains	Sandstone, oolite
4	Rough	Rough fracture of fine- or medium-grained rock containing no easily visible crystalline constituents	Basalt felsite, porphyry, limestone
5	Crystalline	Containing easily visible crystalline constituents	Granite, gabbro, gneiss
6	Honeycombed	With visible pores and cavities	Brick, pumice, foamed slag, clinker, expanded clay

Another approach is to use a shape coefficient and a surface texture coefficient evaluated from a Fourier series method which *a priori* assumes ranges of the harmonic system and also of a modified total roughness coefficient.[3.53] It is doubtful

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whether this type of approach is useful in evaluating and comparing the wide range of shapes and texture properties encountered in practice. Some other approaches are reviewed by Ozol.[3.65]

It seems that the shape and surface texture of aggregate influence considerably the strength of concrete. The flexural strength is more affected than the compressive strength, and the effects of shape and texture are particularly significant in the case of high strength concrete. Some data of Kaplan's[3.3] are reproduced in Table 3.5 but this gives no more than an indication of the type of influence, as some other factors may not have been taken into account. The full role of shape and texture of aggregate in the development of concrete strength is not known, but possibly a rougher texture results in a larger adhesive force between the particles and the cement matrix. Likewise, the larger surface area of angular aggregate means that a larger adhesive force can be developed.

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Table 3.5. Average Relative Importance of the Aggregate Properties Affecting the Strength of Concrete[3.3]

	Relative effect of aggregate properties per cent		
Property of concrete	*Shape*	*Surface texture*	*Modulus of elasticity*
Flexural strength	31	26	43
Compressive strength	22	44	34

N.B. Values represent the ratio of variance due to each property to the total variance accounted for by the three characteristics of aggregate in tests on three mixes made with 13 aggregates.

The shape and texture of fine aggregate have a significant effect on the water requirement of the mix made with the given aggregate. If these properties of fine aggregate are expressed indirectly by its packing, i.e. by the percentage voids in a loose condition (see p. 128), then the influence on the water requirement is quite definite[3.42] (see Fig. 3.3). The influence of the voids in coarse aggregate is less definite.[3.42]

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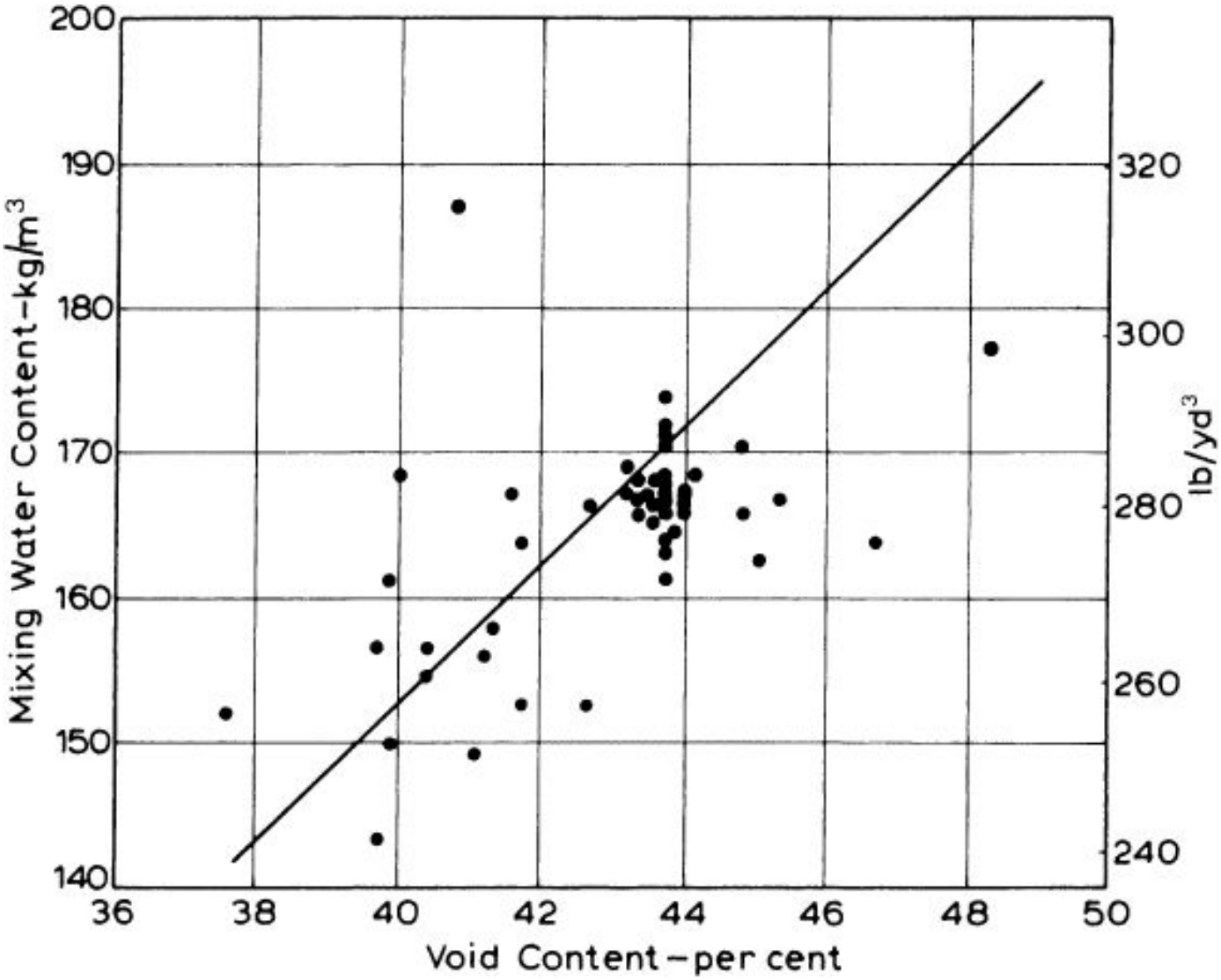


Fig. 3.3. Relation between void content of sand in a loose condition and the water requirement of concrete made with the given sand[3.42]

Flakiness and the shape of coarse aggregate in general have an appreciable effect on the workability of concrete. Figure 3.4, reproduced from Kaplan's[3.4] paper, shows the pattern of the rela-

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tion between the angularity of coarse aggregate and the compacting factor of concrete made with it. An increase in angularity from minimum to maximum would reduce the compacting factor by about 0.09 but, in practice, clearly there can be no unique relation between the two factors because other properties of aggregate also affect the workability. Kaplan's experimental results,[3.4] however, do not confirm that the surface texture is a factor.

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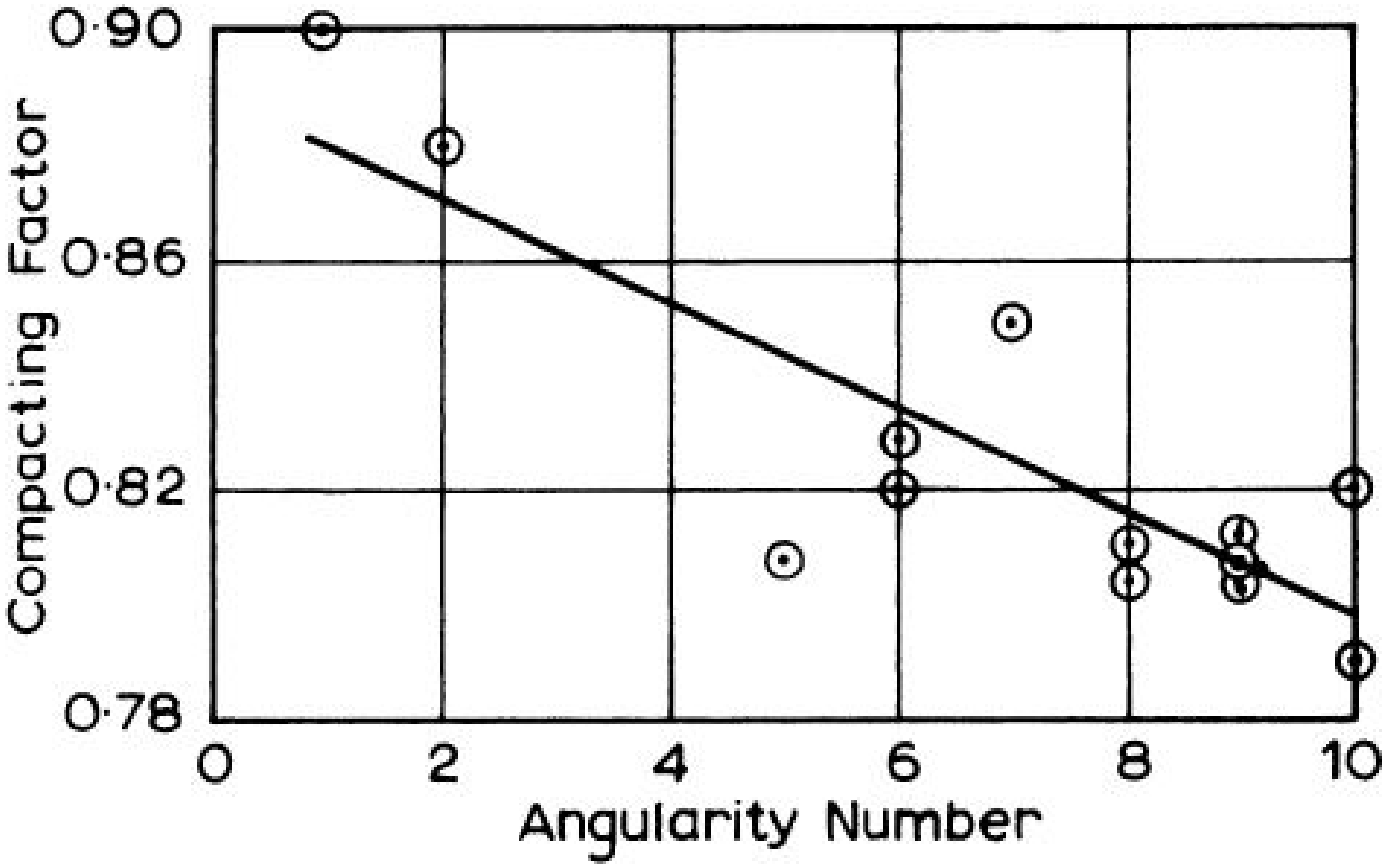


Fig. 3.4. The relation between the angularity number of aggregate and the compacting factor of concrete made with the given aggregate[3.4]

Bond of aggregate

Bond between aggregate and cement paste is an important factor in the strength of concrete, especially the flexural strength, but the nature of bond is not fully understood. Bond is due, in part, to the interlocking of the aggregate and the hy-

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drated cement paste due to the roughness of the surface of the former. A rougher surface, such as that of crushed particles, results in a better bond due to mechanical interlocking; better bond is also usually obtained with softer, porous, and mineralogically heterogeneous particles. Generally, texture characteristics which permit no penetration of the surface of the particles are not conducive to good bond. In addition, bond is affected by other physical and chemical properties of aggregate, related to its mineralogical and chemical composition, and to the electrostatic condition of the particle surface. For instance, some chemical bond may exist in the case of limestone, dolomite,[3.54] and possibly siliceous aggregates, and at the surface of polished particles some capillary forces may develop. However, little is known about these phenomena, and relying on experience is still necessary to predict the bond between the aggregate and the surrounding hydrated cement paste. In any case, for good development of

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bond, it is necessary that the aggregate surface be clean and free from adhering day particles.

As concrete is a composite material, consisting of aggregate and of hydrated cement paste matrix, the modulus of elasticity of each component influences the modulus of the composite. The difference between the moduli has a considerable influence on the bond of the aggregate.[3.91]

The determination of the quality of bond of aggregate is rather difficult and no accepted tests exist. Generally, when bond is good, a crushed specimen of normal strength concrete should contain some aggregate particles broken right through, in addition to the more numerous ones pulled out from their sockets. An excess of fractured particles, however, might suggest that the aggregate is too weak. Because it depends on the strength of the hydrated cement paste as well as on the properties of aggregate surface, bond strength increases with the age of concrete; it seems that the ratio of bond strength to the strength of the hydrated cement paste increases

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with age.[3.43] Thus, providing it is adequate, the bond strength *per se* may not be a controlling factor in the strength of ordinary concrete. However, in high strength concrete, there is probably a tendency for the bond strength to be lower than the tensile strength of the hydrated cement paste so that preferential failure in bond takes place. Indeed, the interface between the aggregate and the surrounding cement paste is of importance, if only because coarse aggregate represents a discontinuity and introduces a wall effect.

Barnes *et al.*[3.64] found plates of calcium hydroxide oriented parallel to the interface, with C-S-H behind. Also, the interface zone is rich in the finer particles of cement and has a higher water/cement ratio than the bulk of the cement paste. These observations explain the particular role of silica fume in improving the strength of concrete (see p. 672).

The problem of failure of concrete is discussed more fully in Chapter 6. What is relevant at this stage is the path of cracks under high

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stress. In a homogeneous material, a crack would be normal to the tension force causing the crack to open and therefore be straight or nearly so. However, in a grossly heterogeneous material such as concrete, the crack path may be affected by the presence of coarse aggregate. Thus the crack may pass through the aggregate, or around the aggregate through the interface zone, or through the matrix.

A recent study by Neville[3.91] showed that, beyond a very early age, the crack path is not influenced by the strength of the mortar matrix; the main factors are the strength of the parent rock, and the shape and surface texture of the particles. However, no simple and measurable parameters have been established. Nevertheless, the topic is of practical interest with reference to the so-called aggregate interlock under shear. Specifically, limestone aggregate fracture produces surfaces too smooth for shear transfer.[3.92]

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Strength of aggregate

Clearly, the compressive strength of concrete cannot significantly exceed that of the *major* part of the aggregate contained therein, although it is not easy to state what is the strength of the individual particles. Indeed, it is difficult to test the crushing strength of individual aggregate particles, and the required information has to be obtained usually from indirect tests: crushing value of bulk aggregate, force required to compact bulk aggregate, and performance of aggregate in concrete.

The latter simply means either previous experience with the given aggregate or a trial use of the aggregate in a concrete mix known to have a certain strength with previously proven aggregates. If the aggregate under test leads to a lower compressive strength of concrete, and in particular if numerous individual aggregate particles appear fractured after the concrete specimen has been crushed, then the strength of the aggregate is lower than the nominal compressive strength of

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the concrete mix in which the aggregate was incorporated. Clearly, such aggregate can be used only in a concrete of lower strength. This is, for instance, the case with laterite, a material widely spread in Africa, South Asia and South America, which can rarely produce concrete stronger than 10 MPa (1500 psi).

Inadequate strength of aggregate represents a limiting case because the physical properties of aggregate have some influence on the strength of concrete even when the aggregate by itself is strong enough not to fracture prematurely. If we compare concretes made with different aggregates, we can observe that the influence of aggregate on the strength of concrete is qualitatively the same whatever the mix proportions, and is the same regardless of whether the concrete is tested in compression or in tension.[3.5] It is possible that the influence of aggregate on the strength of concrete is due not only to the mechanical strength of the aggregate but also, to a considerable degree, to its absorption and bond characteristics.

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In general, the strength and elasticity of aggregate depend on its composition, texture, and structure. Thus, a low strength may be due to the weakness of constituent grains or the grains may be strong but not well knit or cemented together.

The modulus of elasticity of aggregate is rarely determined; this is, however not unimportant because the modulus of elasticity of concrete is generally higher the higher the modulus of the constituent aggregate, but depends on other factors as well. The modulus of elasticity of aggregate affects also the magnitude of creep and shrinkage that can be realized by the concrete (see p. 453). A large incompatibility between the moduli of elasticity of the aggregate and of the hydrated cement paste adversely affects the development of microcracking at the aggregate–matrix interface.

A good average value of the crushing strength of aggregate is about 200 MPa (30 000 psi) but many excellent aggregates range in strength down to 80 MPa (12 000 psi). One of the highest

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values recorded is 530 MPa (77 000 psi) for a certain quartzite. Values for other rocks are given in Table 3.6.[3.6] It should be noted that the required strength of aggregate is considerably higher than the normal range of concrete strengths because the actual stresses at the interface of individual particles within the concrete may be far in excess of the nominal compressive stress applied.

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Table 3.6. Compressive Strength of American Rocks Commonly Used as Concrete Aggregates[3.6]

Type of rock	Number of samples*	Compressive strength					
		Average†		After deletion of extremes‡			
				Maximum		Minimum	
		MPa	psi	MPa	psi	MPa	psi
Granite	278	181	26 200	257	37 300	114	16 600
Felsite	12	324	47 000	526	76 300	120	17 400
Trap	59	283	41 100	377	54 700	201	29 200
Limestone	241	159	23 000	241	34 900	93	13 500
Sandstone	79	131	19 000	240	34 800	44	6 400
Marble	34	117	16 900	244	35 400	51	7 400
Quartzite	26	252	36 500	423	61 300	124	18 000
Gneiss	36	147	21 300	235	34 100	94	13 600
Schist	31	170	24 600	297	43 100	91	13 200

*For most samples, the compressive strength is an average of 3 to 15 specimens.
†Average of all samples.
‡10 per cent of all samples tested with highest or lowest values have been deleted as not typical of the material.

On the other hand, aggregate of moderate or low strength and modulus of elasticity can be valuable in preserving the integrity of concrete. Volume changes of concrete, arising from hygral or thermal causes, lead to a lower stress in the hydrated cement paste when the aggregate is compressible. Thus, compressibility of aggregate

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would reduce distress in concrete while a strong and rigid aggregate might lead to cracking of the surrounding cement paste.

It may be noted that no general relation exists between the strength and modulus of elasticity of different aggregates.[3.3] Some granites, for instance, have been found to have a modulus of elasticity of 45 GPa (6.5×10^6 psi), and gabbro and diabase a modulus of 85.5 GPa (12.4×10^6 psi), the strength of all these rocks ranging between 145 and 170 MPa (21 000 to 25 000 psi). Values of the modulus in excess of 160 GPa (23×10^6 psi) have been encountered.

A test to measure the compressive strength of prepared rock cylinders used to be prescribed. However, the results of such a test are affected by the presence of planes of weakness in the rock which may not be significant once the rock has been comminuted to the size used in concrete. In essence, the crushing strength test measures the quality of the parent rock rather than the qual-

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ity of the aggregates as used in concrete. For this reason the test is rarely used.

Sometimes, the strength of a wet, as well as of a dry, specimen of rock is determined. The ratio of wet to dry strengths measures the softening effect, and when this is high, poor durability of the rock may be suspected.

A test on the crushing properties of bulk aggregate is the so-called *crushing value test* of BS 812-110 : 1990, which measures resistance to pulverization.[3.38] The crushing value is a useful guide when dealing with aggregates of unknown performance, particularly when lower strength may be suspected. There is no obvious physical relation between this crushing value and the compressive strength, but the results of the two tests are usually in agreement.[3.75]

The material to be tested for crushing value should pass a 14.0 mm ($\frac{1}{2}$ in.) test sieve and be retained on a 10.0 mm ($\frac{3}{8}$ in.) sieve. When, however, this size is not available, particles of

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other sizes may be used, but those larger than standard will in general give a higher crushing value, and the smaller ones a lower value, than would be obtained with the same rock of standard size. The sample to be tested should be dried in an oven at 100 to 110 °C (212 to 230 °F) for four hours, and then placed in a cylindrical mould and tamped in a prescribed manner. A plunger is put on top of the aggregate and the whole assembly is placed in a compression testing machine and subjected to a load of 400 kN (40 ton) (pressure of 22.1 MPa (3200 psi)) over the gross area of the plunger, the load being increased gradually over a period of 10 minutes. After the load has been released, the aggregate is removed and sieved on a 2.36 mm (No. 8 ASTM*) test sieve in the case of a sample of the 14.0 to 10.0 mm ($\frac{1}{2}$ to $\frac{3}{8}$ in.) standard size; for aggregates of other sizes, the sieve size is prescribed in BS 812 : 110 : 1990. The ratio of mass of the material passing the smaller sieve to the total mass of the sample is called the aggregate crushing value. Attempts to

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develop a test for lightweight aggregates, similar to the crushing value test described above, have been made but no test has been standardized.

*For *numbers* of sieves see Table 3.14.

The crushing value test is rather insensitive to the variation in strength of weaker aggregates, i.e. those with a crushing value greater than about 25. This is so because, having been crushed before the full load of 400 kN (40 ton) has been applied, these weaker materials become compacted so that the amount of crushing during later stages of the test is reduced. Likewise, flaky particles increase the crushing value.[3.38] For these reasons, a *ten per cent fines value test* has been introduced and is included in the BS 812-111 : 1990. In this test, the apparatus of the standard crushing test is used to determine the load required to produce 10 per cent fines from the 14.0 to 10.0 mm ($\frac{1}{2}$ to $\frac{3}{8}$ in.) particles compacted in a cylinder. This is achieved by applying a progressively increasing

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load on the plunger so as to cause its penetration in 10 minutes of about:

15 mm (0.60 in.) for rounded aggregate,
20 mm (0.80 in.) for crushed aggregate, and
24 mm (0.95 in.) for honeycombed aggregate (such as expanded shale or foamed slag).

These penetrations should result in a percentage of fines passing a 2.36 mm (No. 8 ASTM) sieve of between 7.5 and 12.5 per cent. If y is the actual percentage of fines due to a maximum load of x tons, then the load required to give 10 per cent fines is given by $14x/(y + 4)$.

It should be noted that in this test, unlike the standard crushing value test, a higher numerical result denotes a higher strength of the aggregate. British Standard BS 882 : 1992 (superseded by BS EN 12620 : 2002) prescribes a minimum ten per cent fines value of 150 kN (15 ton) for aggregate to be used in heavy duty floors, 100 kN (10 ton) for concrete for wearing surfaces, and 50 kN (5 ton) when used in other concretes.

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The ten per cent fines value test shows a fairly good correlation with the standard crushing value test for strong aggregates, while for weaker aggregates the ten per cent fines value test is more sensitive and gives a truer picture of differences between more or less weak aggregates. For this reason, the test is of use in assessing lightweight aggregates but there is no simple relation between the test result and the upper limit of strength of concrete made with the given aggregate.

Other mechanical properties of aggregate

Several mechanical properties of aggregate are of interest, especially when the aggregate is to be used in pavement construction or is to be subjected to high wear.

The first of these is *toughness*, which can be defined as the resistance of a sample of rock to failure by impact. Although this test would dis-

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close adverse effects of weathering of the rock, it is not used.

It is possible also to determine the *impact value* of bulk aggregate, and toughness determined in this manner is related to the crushing value, and can, in fact, be used as an alternative test. The size of the particles tested is the same as in the crushing value test, and the permissible values of the crushed fraction smaller than a 2.36 mm (No. 8 ASTM) test sieve are also the same. The impact is applied by a standard hammer falling 15 times under its own weight upon the aggregate in a cylindrical container. This results in fragmentation in a manner similar to that produced by the pressure of the plunger in the aggregate crushing value test. Details of the test are prescribed in BS 812-112 : 1990 (2000), and BS 882 : 1992 prescribes the following maximum values: 25 per cent when the aggregate is to be used in heavy duty floors; 30 per cent when the aggregate is to be used in concrete for wearing surfaces; and 45 per cent when it is to be used in other con-

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cretes. These figures serve as useful guides, but it is clear that a direct correlation between the crushing value and the performance of aggregate in concrete or the strength of the concrete is not possible.

One advantage of the impact value test is that it can be performed in the field with some modifications, such as the measurement of quantities by volume rather than by mass, but the test may not be adequate for compliance purposes.

In addition to strength and toughness, hardness or resistance to wear is an important property of concrete used in pavements and in floor surfaces subjected to heavy traffic. Several tests are available because it is possible to cause wear of aggregate by abrasion, i.e. by rubbing of a foreign material against the stone under test, or by attrition of stone particles against one another.

It may be worth noting that some limestone rocks are subject to wear, and their use in concrete pavement should be conditional on abrasion testing. In other respects, many limestone aggreg-

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ates, even when porous, can produce satisfactory concrete.[3.67]

Abrasion of rock specimens is no longer determined and, in keeping with the tendency to test aggregate in bulk, an *abrasion value test* on aggregate particles is prescribed by BS 812-113 : 1990. Aggregate particles between 14.0 and 10.2 mm, with flaky particles removed, are embedded in resin in a single layer. The sample is subjected to abrasion in a standard machine, the grinding lap being turned 500 revolutions with Leighton Buzzard sand (see p. 54) being fed continuously at a prescribed rate. The aggregate abrasion value is defined in terms of the percentage loss in mass on abrasion, so that a high value denotes a low resistance to abrasion. BS 812-113 : 1990 has been withdrawn, and superseded by BS EN 12620 : 2002 where the value of the coefficient has to be declared. PD 6682-1 : 2009 gives information about abrasion tests.

European standard BS EN 12620 : 2002 prescribes the determination of a so-called micro-

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Deval coefficient, which is a measure of wear of 10 to 14 mm aggregate particles produced by friction between those particles and an abrasive charge in a rotating drum. The coefficient represents the percentage loss in mass in the form of particles reduced to a size smaller than 1.6 mm.

The *attrition* (*Deval*) *test* also uses aggregate in bulk but is no longer used because it gives only small numerical differences between widely differing aggregates.

An American test combining attrition and abrasion is the *Los Angeles test*; it is quite frequently used in other countries, too, because its results show good correlation not only with the actual wear of aggregate when used in concrete but also with the compressive and flexural strengths of concrete made with the given aggregate. In this test, aggregate of specified grading is placed in a cylindrical drum, mounted horizontally, with a shelf inside. A charge of steel balls is added, and the drum is rotated a specified number of revolutions. The tumbling and drop-

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ping of the aggregate and the balls results in abrasion and attrition of the aggregate, and this is measured in the same way as in the attrition test.

The Los Angeles test can be performed on aggregates of different sizes, the same wear being obtained by an appropriate mass of the sample and of the charge of steel balls, and by a suitable number of revolutions. The various quantities are prescribed by ASTM C 131-06. The Los Angeles test is, however, not very suitable for the assessment of the behaviour of fine aggregate when subjected to attrition on prolonged mixing; limestone fine aggregate is probably one of the more common materials to undergo this degradation. For this reason, unknown fine aggregates should, in addition to standard tests, be subjected to a wet attrition test to see how much material smaller than 75 μm (No. 200 ASTM) sieve is produced. The degree to which fine aggregate can be subject to degradation in the mixer can be determined by the method of ASTM C 1137-05.

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Table 3.7 gives average values of crushing strength, aggregate crushing value, and abrasion, impact, and attrition values for the different rock groups of BS 812-1 : 1975 which has been superseded by BS EN 12620 : 2002. That standard deals also with recycled aggregate, that is aggregate resulting from processing inorganic materials previously used in construction. Recycled aggregate is not discussed in this book. It should be noted that the values for hornfels and schists are based on a few specimens only; these groups would appear to be better than they really are, presumably because only good quality hornfels and schists were tested. As a rule, they are not suitable for use in concrete. Likewise, chalk is not included in the limestone group data as it is not generally suitable as a concrete aggregate.

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Table 3.7. Average Test Values for British Rocks of Different Groups*

Rock group	Crushing strength		Aggregate crushing value	Abrasion value	Impact value	Attrition value†		Specific gravity
	MPa	psi				Dry	Wet	
Basalt	200	29 000	12	17.6	16	3.3	5.5	2.85
Flint	205	30 000	17	19.2	17	3.1	2.5	2.55
Gabbro	195	28 500	—	18.7	19	2.5	3.2	2.95
Granite	185	27 000	20	18.7	13	2.9	3.2	2.69
Gritstone	220	32 000	12	18.1	15	3.0	5.3	2.67
Hornfels	340	49 500	11	18.8	17	2.7	3.8	2.88
Limestone	165	24 000	24	16.5	9	4.3	7.8	2.69
Porphyry	230	33 500	12	19.0	20	2.6	2.6	2.66
Quartzite	330	47 500	16	18.9	16	2.5	3.0	2.62
Schist	245	35 500	—	18.7	13	3.7	4.3	2.76

*Courtesy of the late Professor J. F. Kirkaldy.
†Lower value denotes a better quality.

As far as the crushing strength is concerned, basalt is extremely variable, fresh basalts with little olivine reaching some 400 MPa (60 000 psi), while decomposed basalt at the other end of the scale may have a strength of no more than 100 MPa (15 000 psi). Limestone and porphyry show much less variation in strength, and in Great Britain porphyry has a good general performance – rather better than that of granites, which tend to be variable.

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An indication of the accuracy of the results of the different tests is given in Table 3.8, listing the number of samples to be tested in order to ensure a 90 per cent probability that the mean value for the samples is within ±3 per cent and also within ±10 per cent of the true mean.[3.40] The aggregate crushing value shows up as particularly consistent. On the other hand, the prepared specimens show a greater scatter of results than the bulk samples, which is of course to be expected. While the various tests described in this and succeeding sections give an indication of the quality of the aggregate, it is not possible to predict from the properties of aggregate the potential strength development of concrete made with the given aggregate, and indeed it is not yet possible to translate physical properties of aggregate into its concrete-making properties.

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Table 3.8. Reproducibility of Test Results on Aggregate[3.40] (Crown copyright)

Test	*Coefficient of variation, per cent*	*Number of samples to be tested to ensure 0.9 probability that mean will be:*	
		Within ±3 per cent of true mean	*Within ±10 per cent of true mean*
Dry attrition	5.7	10	1
Wet attrition	5.6	9	1
Abrasion	9.7	28	3
Impact of prepared specimen	17.1	90	8
Impact of bulk aggregate	3.0	—	—
Crushing strength	14.3	60	6
Aggregate crushing value	1.8	1	—
Los Angeles test	1.6	1	—

Specific gravity

Because aggregate generally contains pores, both permeable and impermeable (see p. 129), the meaning of the term specific gravity has to be carefully defined, and there are indeed several types of specific gravity.

The *absolute* specific gravity refers to the volume of the solid material excluding all pores, and can, therefore, be defined as the ratio of the mass of the solid, referred to vacuum, to the mass

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of an equal volume of gas-free distilled water, both taken at a stated temperature. Thus, in order to eliminate the effect of totally enclosed impermeable pores the material has to be pulverized, and the test is both laborious and sensitive. Fortunately, it is not normally required in concrete technology work.

If the volume of the solid is deemed to include the impermeable pores, but not the capillary ones, the resulting specific gravity is prefixed by the word *apparent*. The apparent specific gravity is then the ratio of the mass of the aggregate dried in an oven at 100 to 110 °C (212 to 230 °F) for 24 hours to the mass of water occupying a volume equal to that of the solid including the impermeable pores. The latter mass is determined using a vessel which can be accurately filled with water to a specified volume. Thus, if the mass of the oven-dried sample is D, the mass of the vessel full of water is B, and the mass of the vessel with the sample and topped up with water is A, then the mass of the water occupying the same volume

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as the solid is $B - (A - D)$. The apparent specific gravity is then

$$\frac{D}{B - A + D}.$$

The vessel referred to earlier, and known as a pycnometer, is usually a one-litre jar with a watertight metal conical screwtop having a small hole at the apex. The pycnometer can thus be filled with water so as to contain precisely the same volume every time.

Calculations with reference to concrete are generally based on the *saturated and surface-dry* condition of the aggregate (see p. 132) because the water contained in *all* the pores in the aggregate does not take part in the chemical reactions of cement and can, therefore, be considered as part of the aggregate. Thus, if a sample of the saturated and surface-dry aggregate has a mass C, the gross apparent specific gravity is

$$\frac{C}{B - A + C}.$$

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This is the specific gravity most frequently and easily determined, and it is used for calculations of yield of concrete or of the quantity of aggregate required for a given volume of concrete.

The apparent specific gravity of aggregate depends on the specific gravity of the minerals of which the aggregate is composed and also on the amount of voids. The majority of natural aggregates have a specific gravity of between 2.6 and 2.7, and the range of values is given in Table 3.9.[3.7] The values for artificial aggregates extend from considerably below to very much above this range (see Chapter 13).

Table 3.9. Apparent Specific Gravity of Different Rock Groups[3.7] (Crown copyright)

Rock group	*Average specific gravity*	*Range of specific gravities*
Basalt	2.80	2.6–3.0
Flint	2.54	2.4–2.6
Granite	2.69	2.6–3.0
Gritstone	2.69	2.6–2.9
Hornfels	2.82	2.7–3.0
Limestone	2.66	2.5–2.8
Porphyry	2.73	2.6–2.9
Quartzite	2.62	2.6–2.7

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As mentioned earlier, specific gravity of aggregate is used in the calculation of quantities but the actual value of the specific gravity of aggregate is not a measure of its quality. Thus, the value of specific gravity should not be specified unless we are dealing with a material of a given petrological character when a variation in specific gravity would reflect the porosity of the particles. An exception to this is the case of mass construction, such as a gravity dam, where a minimum density of concrete is essential for the stability of the structure.

Bulk density

It is well known that in the SI system the density of a material is numerically equal to its specific gravity although, of course, the latter is a ratio while density is expressed in kilograms per litre. However, in concrete practice, expressing the density of kilograms per cubic metre is more common. In the American or Imperial system, specific gravity has to be multiplied by the unit

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mass of water (approximately 62.4 lb/ft^3) in order to be converted into *absolute density* (specific mass) expressed in pounds per cubic foot.

This absolute density, it must be remembered, refers to the volume of the individual particles only, and of course it is not physically possible to pack these particles so that there are no voids between them. When aggregate is to be actually batched by volume it is necessary to know the mass of aggregate that would fill a container of unit volume. This is known as the *bulk density* of aggregate, and this density is used to convert quantities by mass to quantities by volume.

The bulk density clearly depends on how densely the aggregate is packed, and it follows that, for a material of a given specific gravity, the bulk density depends on the size distribution and shape of the particles: particles all of one size can be packed to a limited extent, but smaller particles can be added in the voids between the larger ones, thus increasing the bulk density of the packed material. The shape of the particles

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greatly affects the closeness of packing that can be achieved.

For a coarse aggregate of given specific gravity, a higher bulk density means that there are fewer voids to be filled by fine aggregate and cement, and the bulk density test was at one time used as a basis of proportioning of mixes.

The actual bulk density of aggregate depends not only on the various characteristics of the material which determine the potential degree of packing, but also on the actual compaction achieved in a given case. For instance, using spherical particles all of the same size, the densest packing is achieved when their centres lie at the apexes of imaginary tetrahedra. The bulk density is then 0.74 of the absolute density (specific mass) of the material. For the loosest packing, the centres of spheres are at the corners of imaginary cubes and the bulk density is only 0.52 of the specific mass of the solid.

Thus, for test purposes, the degree of compaction has to be specified. British Standard BS 812

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: 2 : 1995 recognizes two degrees: loose (or uncompacted) and compacted. The test is performed in a metal cylinder of prescribed diameter and depth, depending on the maximum size of the aggregate and also on whether compacted or uncompacted bulk density is being determined.

For the determination of loose bulk density, the dried aggregate is gently placed in the container to overflowing and then levelled by rolling a rod across the top. In order to find the compacted or rodded bulk density, the container is filled in three stages, each third of the volume being tamped a prescribed number of times with a 16 mm ($\frac{5}{8}$ in.) diameter round-nosed rod. Again, the overflow is removed. The net mass of the aggregate in the container divided by its volume then represents the bulk density for either degree of compaction. The ratio of the loose bulk density to the compacted bulk density lies usually between 0.87 and 0.96.[3.55]

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Knowing the apparent specific gravity for the saturated and surface-dry condition, s, the voids ratio can be calculated from the expression:

$$\text{voids ratio} = 1 - \frac{\text{bulk density}}{s \times \text{unit mass of water}}.$$

If the aggregate contains surface water, it will pack less densely owing to the bulking effect. This is discussed on p. 134. Moreover, the bulk density as determined in the laboratory may not be directly suitable for conversion of mass to volume of aggregate for purposes of volume batching as the degree of compaction in the laboratory and on the site may not be the same.

The bulk density of aggregate is of interest in connection with the use of lightweight and heavy aggregates (see p. 763).

Porosity and absorption of aggregate

The presence of internal pores in the aggregate particles was mentioned in connection with the specific gravity of aggregate, and indeed the characteristics of these pores are very important

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in the study of its properties. The porosity of aggregate, its permeability, and absorption influence such properties of aggregate as the bond between it and the hydrated cement paste, the resistance of concrete to freezing and thawing, as well as its chemical stability and resistance to abrasion. As stated earlier, the apparent specific gravity of aggregate also depends on its porosity and, as a consequence, the yield of concrete for a given mass of aggregate is affected (see p. 764).

The pores in aggregate vary in size over a wide range, the largest being large enough to be seen under a microscope or even with the naked eye, but even the smallest aggregate pores are larger than the gel pores in the cement paste. Pores smaller than 4 μm are of special interest as they are generally believed to affect the durability of aggregates subjected to alternating freezing and thawing (see 545).

Some of the aggregate pores are wholly within the solid; others open onto the surface of the particle. The cement paste, because of its viscos-

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ity, cannot penetrate to a great depth any but the largest of the aggregate pores, so that is the gross volume of the particle that is considered solid for the purpose of calculating the aggregate content in concrete. However, water can enter the pores, the amount and rate of penetration depending on their size, continuity and total volume. The values of porosity of some common rocks are given in Table 3.10 and, since aggregate represents some three-quarters of the volume of concrete, it is clear that the porosity of aggregate materially contributes to the overall porosity of concrete.

Table 3.10. Porosity of Some Common Rocks

Rock group	*Porosity, per cent*
Gritstone	0.0–48.0
Quartzite	1.9–15.1
Limestone	0.0–37.6
Granite	0.4–3.8

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When all the pores in the aggregate are full, it is said to be saturated and surface-dry. If aggregate in this condition is allowed to stand free in dry air, e.g. in the laboratory, some of the water contained in the pores will evaporate and the aggregate will be less than saturated, i.e. air-dry. Prolonged drying in an oven would reduce the moisture content of the aggregate still further until, when no moisture whatever is left, the aggregate is said to be bone-dry. These various stages are shown diagrammatically in Fig. 3.5, and some typical values of absorption are given in Table 3.11. At the extreme right of Fig. 3.5, the aggregate contains surface moisture and is darker in colour.

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Table 3.11. Typical Values of Absorption of Different British Aggregates[3.8]

Aggregate size (with ASTM sieve designation) and type	Shape	Moisture contained in air-dry aggregate as a percentage of dry mass	Absorption (moisture contained in saturated and surface-dry aggregate) as a percentage of dry mass
19.0–9.5 mm ($\frac{3}{4}$–$\frac{3}{8}$ in.) Thames Valley river gravel	Irregular	0.47	2.07
9.5–4.8 mm ($\frac{3}{8}$–$\frac{3}{16}$ in.) Thames Valley river gravel	Irregular	0.84	3.44
4.8–2.4 mm ($\frac{3}{16}$ in. – No. 8) Thames Valley river sand	Irregular	0.50	3.15
2.4–1.2 mm (No. 8–16)	Irregular	0.30	2.90
1.2 mm–600 μm (No. 16–30)	Irregular	0.30	1.70
600–300 μm (No. 30–50)	Irregular	0.40	1.10
300–150 μm (No. 50–100)	Irregular	0.50	1.25
150–75 μm (No. 100–200)	Irregular	0.60	1.60
4.8 mm–150 μm ($\frac{3}{16}$ in. – No. 100) Thames Valley river sand zone 2	Irregular	0.80	1.80
19.0–9.5 mm ($\frac{3}{4}$–$\frac{3}{8}$ in.) Test river gravel	Irregular	1.13	3.30
9.5–4.8 mm ($\frac{3}{8}$–$\frac{3}{16}$ in.) Test river gravel	Irregular	0.53	4.53
19.0–9.5 mm ($\frac{3}{4}$–$\frac{3}{8}$ in.) Bridport gravel	Rounded	0.40	0.93
9.5–4.8 mm ($\frac{3}{8}$–$\frac{3}{16}$ in.) Bridport gravel	Rounded	0.50	1.17
19.0–9.5 mm ($\frac{3}{4}$–$\frac{3}{8}$ in.) Mountsorrel granite	Angular	0.30	0.57
9.5–4.8 mm ($\frac{3}{8}$–$\frac{3}{16}$ in.) Mountsorrel granite	Angular	0.45	0.80
19.0–9.5 mm ($\frac{3}{4}$–$\frac{3}{8}$ in.) crushed limestone	Angular	0.15	0.50
9.5–4.8 mm ($\frac{3}{8}$–$\frac{3}{16}$ in.) crushed limestone	Angular	0.20	0.73
850–600 μm (No. 20–30) Leighton Buzzard standard sand	Rounded	0.05	0.20

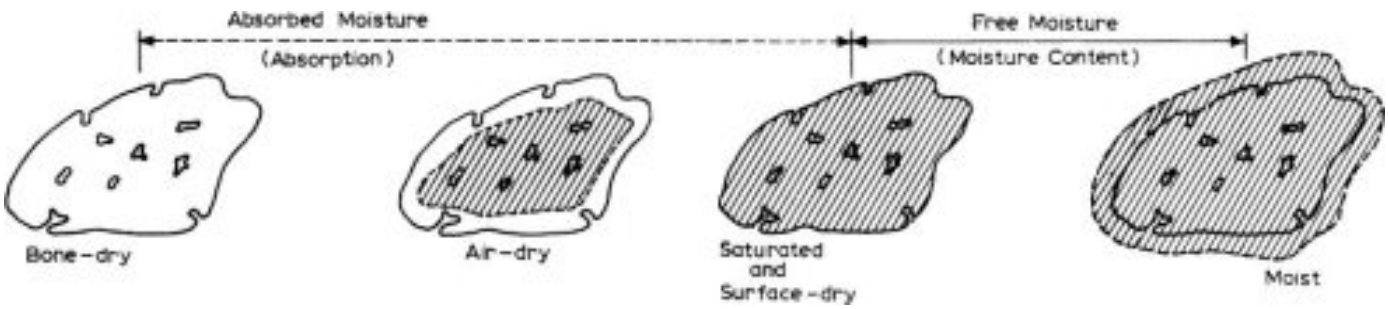


Fig. 3.5. Diagrammatic representation of moisture in aggregate

The water absorption of aggregate is determined by measuring the increase in mass of an oven-dried sample when immersed in water for 24 hours (the surface water being removed). The ratio of the increase in mass to the mass of the

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dry sample, expressed as a percentage, is termed absorption. Standard procedures are prescribed in BS 812-2 : 1995.

Some typical values of absorption of different aggregates are given in Table 3.11, based on Newman's data.[3.8] The moisture content in the air-dry condition is also tabulated. It may be noted that gravel has generally a higher absorption than crushed rock of the same petrological character because weathering results in the outer layer of the gravel particles being more porous and absorbent.

Although there is no clear-cut relation between the strength of concrete and the water absorption of aggregate used, the pores at the surface of the particle affect the bond between the aggregate and the cement paste, and may thus exert some influence on the strength of concrete.

Normally, it is assumed that, at the time of setting of concrete, the aggregate is in a saturated and surface-dry condition. If the aggregate is batched in a dry condition, it is assumed that

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sufficient water will be absorbed from the mix to bring the aggregate to a saturated condition, and this absorbed water is not included in the free or effective mixing water. Such a situation can be encountered in a hot, dry climate. It is possible, however, that, when dry aggregate is used, the particles become quickly coated with cement paste which prevents further ingress of water necessary for saturation. This is particularly so with coarse aggregate, where water has further to travel from the surface of the particle. As a result, the effective water/cement ratio is higher than would be the case had full absorption of water by the aggregate been possible. This effect is significant mainly in rich mixes where rapid coating of aggregate can take place; in lean, wet mixes the saturation of aggregate proceeds undisturbed. In practical cases, the actual behaviour of the mix is affected also by the order of feeding the ingredients into the mixer.

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The absorption of water by aggregate results also in some loss of workability with time, but beyond about 15 minutes the loss becomes small.

Because absorption of water by dry aggregate slows down or is stopped owing to the coating of particles with cement paste, it is often useful to determine the quantity of water absorbed in 10 to 30 minutes instead of the total water absorption, which may never be achieved in practice.

Moisture content of aggregate

It was mentioned in connection with the specific gravity that, in fresh concrete, the volume occupied by the aggregate is the volume of the particles including all the pores. If no water movement into the aggregate is to take place, the pores must be full of water, i.e. the aggregate must be in a saturated condition. On the other hand, any water on the surface of the aggregate will contribute to the water in the mix and will occupy a volume in excess of that of the aggreg-

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ate particles. The basic state of the aggregate is thus *saturated and surface-dry*.

Aggregate exposed to rain collects a considerable amount of moisture on the surface of the particles and, except at the surface of the stockpile, keeps this moisture over long periods. This is particularly true of fine aggregate, and the surface- or free moisture (in excess of that held by aggregate in a saturated and surface-dry condition) must be allowed for in the calculation of batch quantities. Coarse aggregate rarely contains more than 1 per cent of surface moisture but fine aggregate can contain in excess of 10 per cent. The surface moisture is expressed as a percentage of the mass of the saturated and surface-dry aggregate, and is termed moisture content.

Since absorption represents the water contained in aggregate in a saturated and surface-dry condition, and the moisture content is the water in excess of that state, the total water content of a moist aggregate is equal to the sum of absorption and moisture content.

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As the moisture content of aggregate changes with weather, and changes also from one part of a stockpile to another, the value of the moisture content has to be determined frequently and a number of methods have been developed. The oldest one consists simply of finding the loss in mass of an aggregate sample when dried on a tray over a source of heat. Care is necessary to avoid over-drying: the sand should be brought to a just free-flowing condition, and must not be heated further. This stage can be determined by feel or by forming the sand into a pile by means of a conical mould; when the mould has been removed the material should slump freely. If the sand has acquired a brownish tinge, this is a sure sign that excessive drying has taken place. This method of determining the moisture content of aggregate, colloquially referred to as the 'frying-pan method', is simple, can be used in the field, and is quite reliable. Microwave ovens can also be used but care is necessary to avoid overheating.

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In the laboratory, the moisture content of aggregate can be determined by means of a pycnometer. The apparent specific gravity of the aggregate on a saturated and surface-dry basis, s, must be known. Then, if B is the mass of the pycnometer full of water, C the mass of the moist sample, and A the mass of the pycnometer with the sample and topped up with water, the moisture content of the aggregate is:

$$\left[\frac{C}{A-B}\left(\frac{s-1}{s}\right)-1\right]\times 100.$$

The test is slow and requires great care in execution (e.g. all air must be expelled from the sample) but can yield accurate results. This method is described in BS 812-109 : 1990.

In the siphon can test[3.9] the volume of water displaced by a known mass of moist aggregate is measured, the siphon making this determination more accurate. Preliminary calibration for each aggregate is required because the results depend

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on its specific gravity but, once this has been done, the test is rapid and accurate.

The moisture content of aggregate can also be found using a steelyard moisture meter: the moist aggregate is added to a vessel containing a fixed amount of water and suspended at one end of a steelyard until it balances. We measure thus the quantity of water that has to be replaced by the moist aggregate for a constant weight and *total* volume. For this condition, it can be shown that the amount of displaced water is proportional to the moisture content of the aggregate. A calibration curve for any aggregate used has to be obtained. The moisture content can be determined with an accuracy of $\frac{1}{2}$ per cent.

In the buoyancy meter test,[3.10] the moisture content of the aggregate of known specific gravity is determined from the apparent loss in weight on immersion in water. The balance can read the moisture content direct if the size of the sample is adjusted, according to the specific gravity of the aggregate, to such a value that a saturated and

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surface-dry sample has a standard weight when immersed. The test is rapid and gives the moisture content to the nearest $\frac{1}{2}$ per cent. A simple version of the test is prescribed by ASTM C 70-06 but is not widely used.

Numerous other methods have been developed. For instance, moisture can be removed by burning the aggregate with methyl alcohol, the resulting loss in mass of the sample being measured. There are also proprietary meters based on the measurement of pressure of gas formed in a closed vessel by the reaction of calcium carbide with the moisture in the sample. ASTM C 566-97 (2004) prescribes a method for the determination of the total moisture content of the aggregate. Although this method is not highly accurate, the error involved is smaller than the sampling error.

It can be seen that a great variety of tests is available but, however accurate the test, its result is useful only if a representative sample has been used. Furthermore, if the moisture content of aggregate varies between adjacent parts of a

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stockpile, the adjustment of mix proportions becomes laborious. Because the variation in moisture content occurs mainly in the vertical direction from a waterlogged bottom of a pile to its drying or dry surface, care in laying out of stockpiles is necessary: storing in horizontal layers, having at least two stockpiles and allowing each pile to drain before use, and not using the bottom 300 mm (12 in.) or so, all help to keep the variation in moisture content to a minimum. Coarse aggregate holds very much less water than fine aggregate, has a less variable moisture content, and generally causes fewer difficulties.

Electrical devices which give instantaneous or continuous reading of the moisture content of aggregate in a storage bin, on the basis of the variation of resistance or capacitance with a change in the moisture content of the aggregate, have been developed. In some batching plants, meters of this type are used in automatic devices which regulate the quantity of water to be added to the mixer but an acuracy greater than 1 per cent of

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moisture cannot in practice be achieved; frequent calibration is necessary. Measurement of the dielectric constant has the advantage of not being affected by the presence of salts. Microwave absorption meters have been developed: these are accurate and stable but expensive. Instruments emitting fast neutrons which are thermalized by hydrogen atoms in the water are also used. All these instruments need to be carefully positioned.

There is no doubt that continuous measurement of moisture and automatic adjustment of the amount of water added into the mixer greatly reduce the variability of the concrete produced when the moisture content of the aggregate is variable. However, widespread adoption of the determination of the moisture content in the aggregate in any given batch is still in the future.

Bulking of fine aggregate

The presence of moisture in aggregate necessitates correction of the actual mix proportions: the mass of water added to the mix has to be de-

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creased by the mass of the free moisture in the aggregate, and the mass of the wet aggregate must be increased by a like amount. In the case of sand, there is a second effect of the presence of moisture: bulking. This is the increase in the volume of a given mass of sand caused by the films of water pushing the sand particles apart. While bulking *per se* does not affect the proportioning of materials by mass, in the case of volume batching, bulking results in a smaller mass of sand occupying the fixed volume of the measuring box. For this reason, the mix becomes deficient in fine aggregate and appears 'stony', and the concrete may be prone to segregation and honeycombing. Also, the yield of concrete is reduced. The remedy, of course, lies in increasing the apparent volume of fine aggregate (sand) to allow for bulking.

The extent of bulking depends on the percentage of moisture present in the sand and on its fineness. The increase in volume relative to that occupied by a saturated and surface-dry sand in-

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creases with an increase in the moisture content of the sand up to a value of some 5 to 8 per cent, when the bulking of 20 to 30 per cent occurs. Upon further addition of water, the films merge and the water moves into the voids between the particles so that the total volume of sand decreases until, when fully saturated (flooded), its volume is approximately the same as the volume of dry sand for the same method of filling the container. This is apparent from Fig. 3.6, which also shows that finer sand bulks considerably more and reaches maximum bulking at a higher water content than does coarse sand. Crushed fine aggregate bulks even more than natural sand. Extremely fine sand (which contains a larger number of particles) has been known to bulk as much as 40 per cent at a moisture content of 10 per cent, but such a sand is, in any case, not used for the production of good quality concrete.

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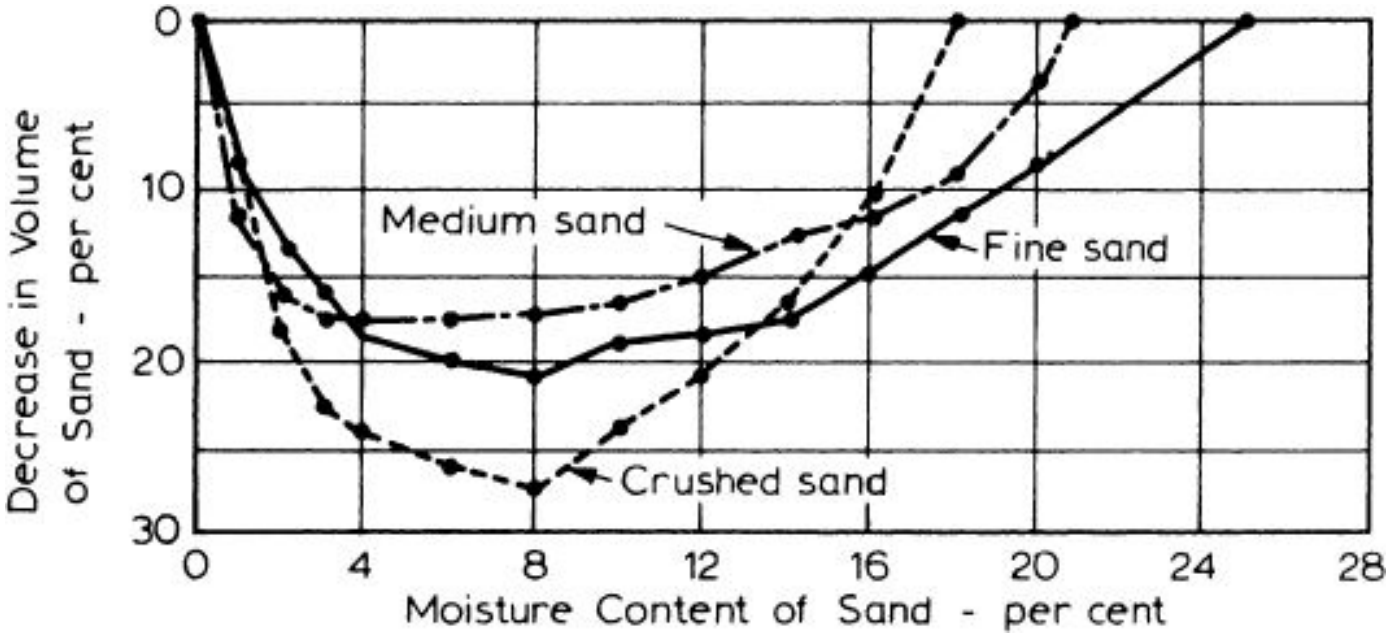


Fig. 3.6. Decrease in true volume of sand due to bulking (for a constant apparent volume of moist sand)

Coarse aggregate shows only a negligible increase in volume due to the presence of free water, as the thickness of moisture films is very small compared with the particle size.

Because the volume of saturated sand is the same as that of dry sand, the most convenient way of determining bulking is by measuring the decrease in volume of the given sand when inundated. A container of known volume is filled with loosely packed moist sand. The sand is then

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tipped out, the container is partially filled with water and the sand is gradually fed back, with stirring and rodding to expel all air bubbles. The volume of sand in the saturated state, V_s, is now measured. If V_m is the initial apparent volume of the sand (i.e. the volume of the container), then bulking is given by $(V_m - V_s)/V_s$.

With volume batching, bulking has to be allowed for by increasing the total volume of (moist) sand used. Thus, volume V_s is multiplied by the factor:

$$1 + \frac{V_m - V_s}{V_s} = \frac{V_m}{V_s},$$

sometimes known as the *bulking factor*; a plot of bulking factor against moisture of three typical sands is shown in Fig. 3.7.

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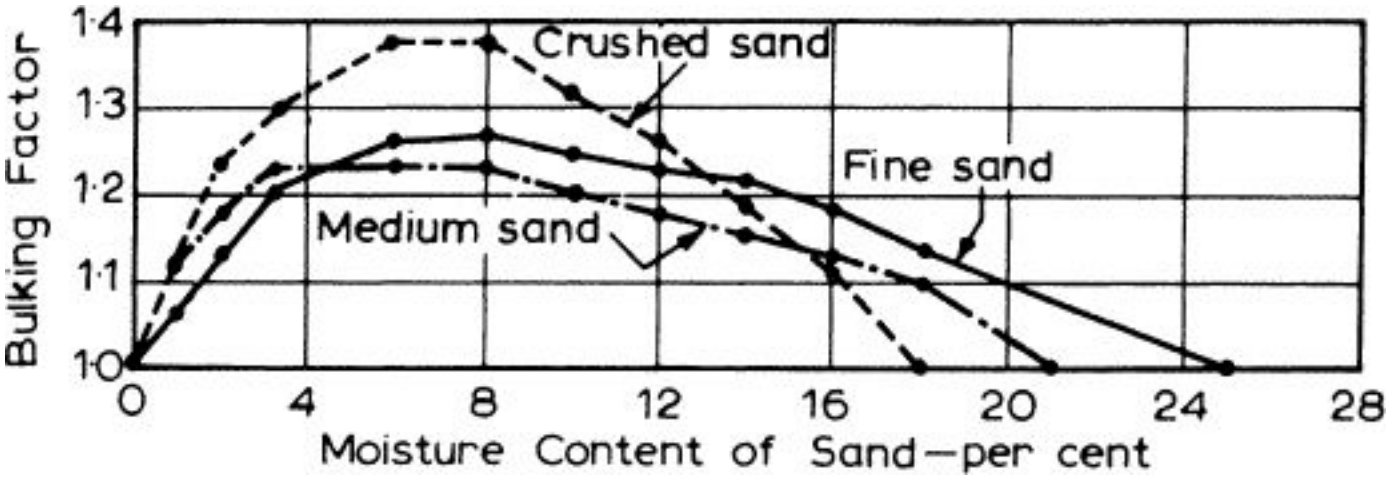


Fig. 3.7. Bulking factor for sands with different moisture contents

The bulking factor can also be found from the bulk densities of dry and moist sand, D_d and D_m, respectively, and the moisture content per unit volume of sand, m/V_m. The bulking factor is then

$$\frac{D_d}{D_m - \frac{m}{V_m}}.$$

Since D_d represents a ratio of the mass of dry sand, w, to its bulk volume V_s (the volumes of dry and inundated sand being the same),

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$$\frac{D_d}{D_m - \frac{m}{V_m}} = \frac{\frac{w}{V_s}}{\frac{(w+m)}{V_m} - \frac{m}{V_m}} = \frac{V_m}{V_s},$$

i.e. the two factors are identical.

Deleterious substances in aggregate

There are three broad categories of deleterious substances that may be found in aggregates: *impurities* which intefere with the processes of hydration of cement; *coatings* preventing the development of good bond between aggregate and the hydrated cement paste; and certain individual particles which are *weak* or *unsound* in themselves. All or part of an aggregate can also be harmful through the development of chemical reactions between the aggregate and the cement paste: these chemical reactions are discussed on p. 144.

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Organic impurities

Natural aggregates may be sufficiently strong and resistant to wear and yet they may not be satisfactory for concrete-making if they contain organic impurities which interfere with the chemical reactions of hydration. The organic matter found in aggregate consists usually of products of decay of vegetable matter (mainly tannic acid and its derivatives) and appears in the form of humus or organic loam. Such materials are more likely to be present in sand than in coarse aggregate, which is easily washed.

Not all organic matter is harmful and it is best to check its effects by making actual compression test specimens. Generally, however, it saves time to ascertain first whether the amount of organic matter is sufficient to warrant further tests. This is done by the so-called colorimetric test of ASTM C 40-04. The acids in the sample are neutralized by a 3 per cent solution of NaOH, prescribed quantities of aggregate and of solution being placed in a bottle. The mixture is vigor-

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ously shaken to allow the intimate contact necessary for chemical reaction, and then left to stand for 24 hours, when the organic content can be judged by the colour of the solution: the greater the organic content the darker the colour. If the colour of the liquid above the test sample is not darker than the standard yellow colour defined by the ASTM Standard, the sample can be assumed to contain only a harmless amount of organic impurities.

If the observed colour is darker than the standard yellow, i.e. if the solution appears brownish or brown, the aggregate has a rather high organic content, but this does not necessarily mean that the aggregate is not fit for use in concrete. The organic matter present may not be harmful to concrete or the colour may be due to some iron-bearing minerals. For this reason, further tests are necessary: ASTM C 87-05 recommends strength tests on mortar with the suspect sand as compared with mortar made with the same, but washed,

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sand. The colorimetric test is no longer specified in British Standards.

In some countries, the quantity of organic matter in aggregate is determined from the loss of mass of a sample on treating with hydrogen peroxide.

It is interesting to note that, in some cases, the effects of organic impurities may be only temporary. In one investigation,[3.11] concrete made with a sand containing organic matter had a 24-hour strength equal to 53 per cent of the strength of similar concrete made with clean sand. At 3 days, this ratio rose to 82 per cent, then to 92 per cent at 7 days, and at 28 days equal strengths were recorded.

Clay and other fine material

Clay may be present in aggregate in the form of surface coatings which interfere with the bond between aggregate and the cement paste. Because good bond is essential to ensure a satisfactory

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strength and durability of concrete, the problem of clay coatings is an important one.

There are two more types of fine material which can be present in aggregate: silt and crusher dust. Silt is material between 2 and 60 μm, reduced to this size by natural processes of weathering; silt may thus be found in aggregate won from natural deposits. On the other hand, crusher dust is a fine material formed during the process of comminution of rock into crushed stone or, less frequently, of gravel into crushed fine aggregate. In a properly laid out processing plant, this dust should be removed by washing. Other soft or loosely adherent coatings can also be removed during the processing of the aggregate. Well-bonded coatings cannot be so removed but, if they are chemically stable and have no deleterious effect, there is no objection to the use of aggregate with such a coating, although shrinkage may be increased. However, aggregates with chemically reactive coatings, even if physically stable, can lead to serious trouble.

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Silt and fine dust may form coatings similar to those of clay, or may be present in the form of loose particles not bonded to the coarse aggregate. Even when they are in the latter form, silt and fine dust should not be present in excessive quantities because, owing to their fineness and therefore large surface area, silt and fine dust increase the amount of water necessary to wet all the particles in the mix.

In view of the above, it is necessary to control the clay, silt and fine dust contents of aggregate. As no test is available to determine separately the clay content, this is not prescribed in British Standards. However, BS 882 : 1992 (withdrawn 2004 and replaced by BS EN 12620 : 2002 and PD 6682-1 : 2009) imposes a limit on the maximum amount of material passing the 75 μm (No. 200) sieve:

in coarse aggregate: 2 per cent, increased to 4 per cent when it consists wholly of crushed rock;

in fine aggregate: 4 per cent, increased to 16 per cent when it consists wholly of crushed rock; and

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in all-in aggregate: 11 per cent.

For heavy duty floors, the limit is 9 per cent. ASTM 33-08 also specifies grading requirements. European standard BS EN 12620 : 2002 requires the content of fines to be declared.

In the BS standard, the content of clay lumps and friable particles is specified separately as 3 per cent in fine, and 2 to 10 per cent in coarse aggregate, depending on the use of the concrete.

It should be noted that different test methods are prescribed in different standards so that the results are not directly comparable.

The clay, silt and fine dust content of fine aggregate can be determined by the sedimentation method described in BS 812-103.2 : 1989 (2000). The sample is placed in a sodium hexametaphosphate solution in a stoppered jar and rotated with the axis of the jar horizontal for 15 minutes at approximately 80 revolutions per minute. The fine solids become dispersed and the amount of suspended material is then measured by means of a pipette. A simple calculation gives the percentage

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of clay, fine silt and fine dust in the fine aggregate, the separation size being 20 μm.

A similar method, with suitable modifications, can be used for coarse aggregate which contains very fine material, but it is simpler to wet-sieve the aggregate on a 75 μm (No. 200) test sieve, as prescribed in BS 812 : 103.1 : 1985 (2000) and ASTM C 117-04. This type of sieving is resorted to because fine dust or clay adhering to larger particles would not be separated in ordinary dry sieving. In wet sieving, on the other hand, the aggregate is placed in water and agitated sufficiently vigorously for the finer material to be brought into suspension. By decantation and sieving, all material smaller than a 75 μm (No. 200) test sieve can be removed. To protect this sieve from damage by large particles during decantation, a 1.18 mm (No. 16 ASTM) sieve is placed above the 75 μm (No. 200) sieve.

For natural sands and crushed gravel sands, there is also a field test available which can be performed quite easily and rapidly, with very

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little laboratory equipment. In this non-standard test, 50 ml of an approximately 1 per cent solution of common salt in water is placed in a 250 ml measuring cylinder. Sand, as received, is added until its level reaches the 100 ml mark, and more solution is then added until the total volume of the mixture in the cylinder is 150 ml. The cylinder is now covered with the palm of the hand, shaken vigorously, repeatedly turned upside down, and then allowed to stand for 3 hours. The silt which became dispersed on shaking will now settle in a layer above the sand, and the height of this layer can be expressed as a percentage of the height of the sand below.

It should be remembered that this is a volumetric ratio, which cannot easily be converted to a ratio by mass since the conversion factor depends on the fineness of the material. It has been suggested that for natural sand the mass ratio is obtained by multiplying the volumetric ratio by a factor of $\frac{1}{4}$, the corresponding figure for crushed gravel sand being $\frac{1}{2}$, but with some aggregates

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an even wider variation is obtained. These conversions are not reliable, so that, when the volumetric content exceeds 8 per cent, tests by the more accurate methods, described earlier, should be made.

Salt contamination

Sand won from the seashore or dredged from the sea or a river estuary, as well as desert sand, contains salt, and has to be processed. In the United Kingdom, about 20 per cent of natural gravel and sand is sea dredged, submersible pumps making it possible to win the material from depths up to 50 m (160 ft). The simplest procedure is to wash the sand in fresh water, but special care is required with deposits just above the high-water mark in which large quantities of salt, sometimes over 6 per cent of mass of sand, can be found. Generally, sand from the sea bed, washed even in sea water, does not contain harmful quantities of salts.

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Because of the danger of chloride-induced corrosion of steel reinforcement, BS 8110-1 : 1997 (Structural use of concrete) specifies the maximum total chloride ion content in the mix. The chlorides may arise from all ingredients of the mix. As far as aggregate is concerned, BS 882 : 1992 contains guidance on the maximum chloride ion content in the aggregate which is likely to be acceptable, although the total content in the concrete mix should be verified. The BS 882 : 1992 (withdrawn) limits on the chloride ion content by mass, expressed as a percentage of the mass of the total aggregate, are as follows:

for prestressed concrete	0.01
for reinforced concrete made with sulfate-resisting cement	0.03
for other reinforced concrete	0.05.

The method of BS 812-117 : 1988 (2000) determines the content of water-soluble chlorides; this may be inadequate when the aggregate is porous and chlorides may exist within the aggregate particles.[3.38]

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Apart from the danger of corrosion of steel reinforcement, if salt is not removed, it will absorb moisture from the air and cause efflorescence – unsightly white deposits on the surface of the concrete (see p. 515).

Sea-dredged coarse aggregate may have a large shell content. This usually has no adverse effect on strength but workability of concrete made with aggregate having a large shell content is slightly reduced.[3.44] The shell content of particles larger than 5 mm can be determined by hand picking, using the method of BS 812-106 : 1985 replaced by BS EN 933-7 : 1998. British Standard BS 882 : 1992 (withdrawn) limits the shell content of coarse aggregate to 20 per cent when the maximum size is 10 mm and to 8 per cent when it is larger. Nevertheless, aggregate with a much higher shell content has been used successfully on some Pacific islands. There is no limit on the shell content of fine aggregate. The current standard is BS EN 12620 : 2002.

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Unsound particles

Tests on aggregate sometimes reveal that the majority of the component particles are satisfactory but that a few are unsound: the quantity of such particles must clearly be limited.

There are two broad types of unsound particles: those that fail to maintain their integrity, and those that lead to disruptive expansion on freezing or even on exposure to water. The disruptive properties are characteristic of certain rock groups, and will therefore be discussed in relation to the durability of aggregate in general (mainly in the next section). In this section, non-durable impurities only will be considered.

Shale and other particles of low density are regarded as unsound, and so are soft inclusions, such as clay lumps, wood, and coal as they lead to pitting and scaling. If present in large quantities (over 2 to 5 per cent of the mass of the aggregate) these particles may adversely affect the strength of concrete and should certainly not be permitted in concrete which is exposed to abrasion.

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Coal, in addition to being a soft inclusion, is undesirable for other reasons: it can swell, causing disruption of concrete and, if present in large quantities in a finely divided form, it can disturb the process of hydration of the cement paste. However, discrete particles of hard coal amounting to no more than $\frac{1}{4}$ per cent of the mass of the aggregate have no adverse effect on the strength of concrete.

The presence of coal and other materials of low density can be determined by flotation in a liquid of suitable specific gravity, for instance, by the method of ASTM C 123-04. If the danger of pitting and scaling is not thought important, and strength of concrete is the main consideration, a trial mix should be made.

Mica should be avoided because, in the presence of active chemical agents produced during the hydration of cement, alteration of mica to other forms can occur. Also, free mica in fine aggregate, even in quantities of a few per cent of the mass of the aggregate, affects adversely the wa-

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ter requirement and the strength of concrete.[3.45] Fookes and Revie[3.69] found that a 5 per cent content by mass of mica in sand reduced the 28-day strength of the resulting concrete by about 15 per cent, even when the water/cement ratio was kept constant. The reason for this is probably poor adhesion of the cement paste to the surface of mica particles. It appears that mica in the form of muscovite is much more harmful than biotite.[3.58] These facts should be borne in mind when materials such as china clay sand are considered for use in concrete.

There is no standard method of determining the amount of mica present in sand or even a test on the effect of mica on the properties of concrete. If sand contains mica, it is likely to be concentrated among the finest particles. Gaynor and Meininger[3.63] recommend a microscopical count of mica particles in the fraction of sand between 300 and 150 μm (No. 50 to No. 100) sieve sizes, and if less than about 15 per cent of mica as a number of particles is present in that fraction,

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the properties of concrete are unlikely to be significantly affected. It should be emphasized that the mica content of the larger particles should be many times smaller.

Gypsum and other sulfates must not be present; their content in aggregate can be determined by BS 812 : 118 : 1988. Other requirements for sulfate content are given in PD 6682-1 : 2009. Their existence in many Middle East aggregates leads to difficulties, but up to 5 per cent of SO_3 by mass of cement (including the SO_3 in the cement) is often tolerated there.[3.59] Water-soluble forms, such as magnesium and sodium sulfates, are particularly harmful. Special problems arising from the presence of various salts in aggregates found in arid regions, such as the Middle East, are treated by Fookes and Collis.[3.56,3.57]

Iron pyrites and marcasite represent the most common expansive inclusions in aggregate. These sulfides react with water and oxygen in the air to form a ferrous sulfate which subsequently decomposes to form a hydroxide, while

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the sulfate ions react with calcium aluminates in the cement. Sulfuric acid can also form, and this can attack the hydrated cement paste.[3.76] Surface staining of the concrete and disruption of the cement paste (pop-outs) may result, particularly under warm and humid conditions. The formation of pop-outs can be delayed for many years until water and oxygen are present.[3.76] The appearance problem of pop-outs can be ameliorated by using a smaller maximum size of aggregate.

Not all forms of pyrites are reactive and, because the decomposition of pyrites takes place only in lime water, it is possible to test a suspect aggregate for reactivity by placing the material in a saturated solution of lime.[3.12] If the aggregate is reactive, a blue-green gelatinous precipitate of ferrous sulfate appears within a few minues and, on exposure to air, this changes to brown ferric hydroxide. The absence of this reaction means that no staining need be feared. Lack of reactivity was found[3.12] to be associated with the presence of a number of metal cations, while their absence

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makes the pyrites active. Generally, particles of pyrites likely to cause trouble are those between 5 and 10 mm (or $\frac{3}{16}$ and $\frac{3}{8}$ in.) in size.

The permissible quantities of unsound particles laid down by ASTM C 33-08 are summarized in Table 3.12.

Table 3.12. Permissible Quantities of Unsound Particles Prescribed by ASTM C 33-08

Type of particles	*Maximum content (per cent of mass)*	
	In fine aggregate	*In coarse aggregate*
Friable particles and clay lumps	3.0	3.0–10.0*
Coal	0.5–1.0†	0.5–1.0‡†
Chert that will readily disintegrate	—	3.0–8.0‡

*Including chert.
†Depending on importance of appearance.
‡Depending on exposure.

The majority of impurities discussed in the present section are found in natural aggregate deposits and are much less frequently encountered in crushed aggregate. However, some processed aggregates, such as mine tailings, can contain harmful substances. For instance, small quantities

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of lead soluble in limewater (e.g. 0.1 per cent of PbO by mass of aggregate) greatly delay the set and reduce the early strength of concrete; the long-term strength is unaffected.[3.46]

Soundness of aggregate

This is the term used to describe the ability of aggregate to resist excessive changes in volume as a result of changes in physical conditions. Lack of soundness is thus distinct from expansion caused by the chemical reactions between the aggregate and the alkalis in cement.

The physical causes of large or permanent volume changes of aggregate are freezing and thawing, thermal changes at temperatures above freezing, and alternating wetting and drying.

Aggregate is said to be unsound when volume changes, induced by the above causes, result in deterioration of the concrete. This may range from local scaling and so-called pop-outs to extensive surface cracking and to disintegration over a considerable depth, and can thus vary from

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no more than impaired appearance to a structurally dangerous situation.

Unsoundness is exhibited by porous flints and cherts, especially the lightweight ones with a fine-textured pore structure, by some shales, by limestones with laminae of expansive clay, and by other particles containing clay minerals, particularly of the montmorillonite or illite group. For instance, an altered dolerite has been found to change in dimensions by as much as 600×10^{-6} with wetting and drying, and concrete containing this aggregate might fail under conditions of alternating wetting and drying, and will certainly do so on freezing and thawing. Likewise, the disruption of porous chert arises from freezing.[3.77]

A British test for soundness of aggregate is prescribed in BS 812-121 : 1989 (2000). This determines the proportion of aggregate broken up in consequence of five cycles of immersion in a saturated solution of magnesium sulfate alternating with oven drying. The original sample contains particles between 10.0 and 14.0 mm in size, and

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the mass of particles which remain larger than 10.0 mm, expressed as a percentage of the original mass, is called the *soundness value*.

The American test for soundness of aggregate is prescribed by ASTM C 88-05. A sample of graded aggregate is subjected alternately to immersion in a saturated solution of sodium or magnesium sulfate (the latter being the more severe of the two) and drying in an oven. The formation of salt crystals in the pores of the aggregate tends to disrupt the particles, probably in a manner similar to the action of ice. The reduction in size of the particles, as shown by a sieve analysis, after a number of cycles of exposure denotes the degree of unsoundness. The test is no more than qualitative in predicting the behaviour of the aggregate under actual site conditions, and cannot be used as a basis of acceptance or rejection of unknown aggregates. Specifically, there is no clear reason why soundness as tested by ASTM C 88-05 should be a measure of the performance of the

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given aggregate in concrete subjected to freezing and thawing.

Other tests consist of subjecting the aggregate to cycles of alternating freezing and thawing, and sometimes this treatment is applied to mortar or concrete made with the suspect aggregate. Unfortunately, none of the tests gives an accurate indication of the behaviour of aggregate under actual conditions of moisture and temperature changes above the freezing point.

Likewise, there are no tests which could satisfactorily predict the durability of aggregate in the concrete under conditions of freezing and thawing. The main reason for this is that the behaviour of aggregate is affected by the presence of the surrounding hydrated cement paste, so that only a service record can satisfactorily prove the durability of aggregate.

Nevertheless, certain aggregates are known to be susceptible to frost damage and it is on these that our attention is centred. These are: porous cherts, shales, some limestones, particularly lam-

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inated limestones, and some sandstones. A common characteristic of these rocks with a poor record is their high absorption (see Fig. 3.8),[3.37] but it should be emphasized that many durable rocks also exhibit high absorption.

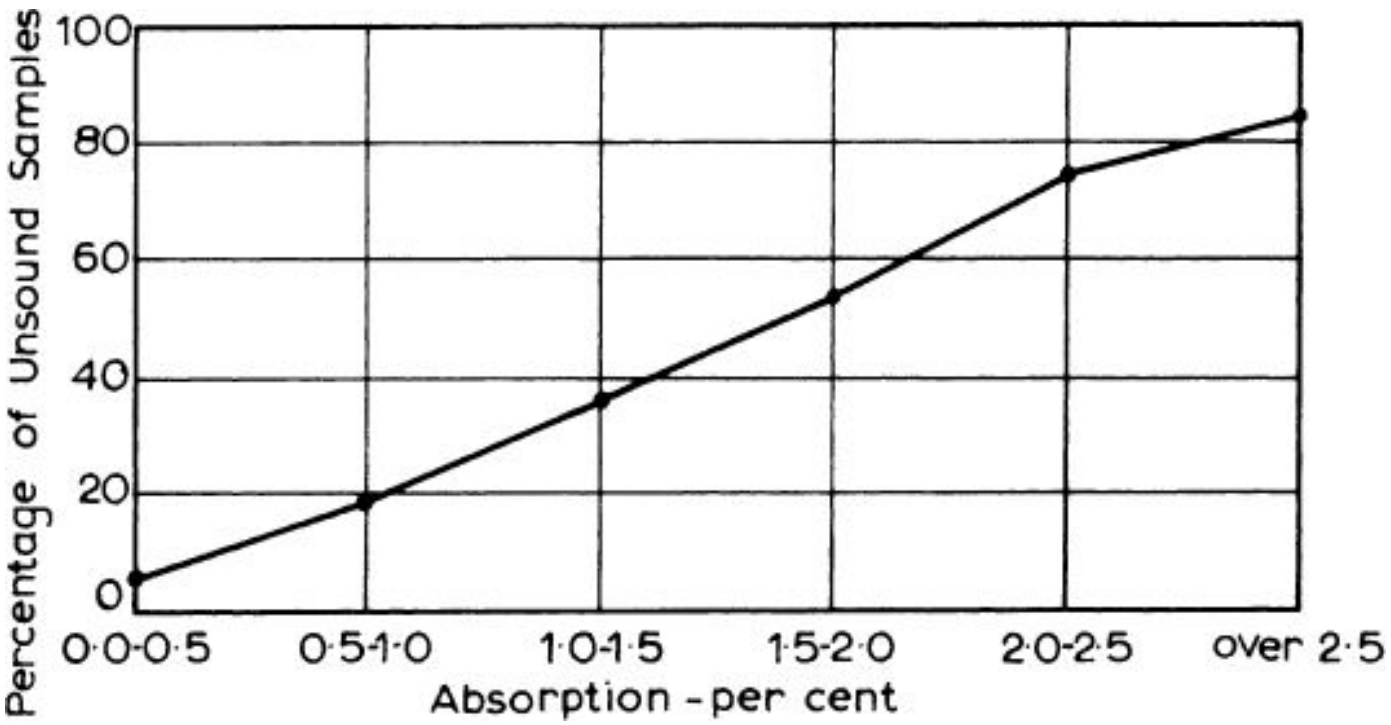


Fig. 3.8. Distribution of sound and unsound aggregate samples as a function of absorption[3.37]

For frost damage to occur, there must exist critical conditions of water content and lack of drainage. These are governed, *inter alia*, by the size, shape and continuity of pores in the ag-

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gregate because these characteristics of the pores control the rate and amount of absorption and also the rate at which water can escape from the aggregate particle. Indeed, these features of the pores are more important than merely their total volume as reflected by the magnitude of absorption.

It has been found that pores smaller than 4 to 5 μm are critical, because they are large enough to permit water to enter but not large enough to allow easy drainage under the pressure of ice. This pressure, in fully confined space at –20 °C (–4 °F), can be as high as 200 MPa (29 000 psi). Thus, if splitting of aggregate particles and disruption of the surrounding cement paste are to be avoided, flow of water towards unfilled pores within the aggregate particle, or into the surrounding paste, must be possible before the hydraulic pressure becomes high enough to cause disruption.

This argument illustrates the statement made earlier that the durability of aggregate cannot be

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fully determined other than when it is embedded in hydrated cement paste: the particle may be strong enough to resist the pressure of ice but expansion can cause disruption of the surrounding mortar.

It has been said that the pore size is an important factor in the durability of aggregate. In most aggregates, pores of different sizes are present so that we are really confronted with a pore size distribution. A means of expressing this quantitatively has been developed by Brunauer, Emmett and Teller.[3.13] The specific surface of the aggregate is determined from the amount of a gas sorbate required to form a layer one molecule thick over the entire internal surface of the aggregate pores. The total volume of the pores is measured by absorption, and the ratio of the volume of pores to their surface represents the hydraulic radius of the pores. This value, familiar from flow problems in hydraulics, gives an indication of the pressure required to produce flow.

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Alkali–silica reaction

In the latter part of the 20th century, an increasing number of deleterious chemical reactions between the aggregate and the surrounding hydrated cement paste has been observed. The most common reaction is that between the active silica constituents of the aggregate and the alkalis in cement. The reactive forms of silica are opal (amorphous), chalcedony (cryptocrystalline fibrous), and tridymite (crystalline). These reactive materials occur in: opaline or chalcedonic cherts, siliceous limestones, rhyolites and rhyolitic tuffs, dacite and dacite tuffs, andesite and andesite tuffs, and phyllites.[3.29]

The reaction starts with the attack on the siliceous minerals in the aggregate by the alkaline hydroxides in pore water derived from the alkalis (Na_2O and K_2O) in the cement. As a result, an alkali-silicate gel is formed, either in planes of weakness or pores in the aggregate (where reactive silica is present) or on the surface of the ag-

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gregate particles. In the latter case, a characteristic altered surface zone is formed. This may destroy the bond between the aggregate and the surrounding hydrated cement paste.

The gel is of the 'unlimited swelling' type: it imbibes water with a consequent tendency to increase in volume. Because the gel is confined by the surrounding hydrated cement paste, internal pressures result and may eventually lead to expansion, cracking and disruption of the hydrated cement paste. Thus, expansion appears to be due to hydraulic pressure generated through osmosis, but expansion can also be caused by the swelling pressure of the still solid products of the alkali-silica reaction.[3.30] For this reason, it is believed that it is the swelling of the hard aggregate particles that is most harmful to concrete. Some of the relatively soft gel is later leached out by water and deposited in the cracks already formed by the swelling of the aggregate. The size of the siliceous particles affects the speed with which reaction occurs, fine particles (20 to 30 μm) lead-

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ing to expansion within a month or two, larger ones only after many years.[3.60]

Reviews of the mechanisms involved in the alkali-silica reactions have been presented by Diamond[3.66] and by Helmuth.[3.78] It is believed that the gel formation takes place only in the presence of Ca^{++} ions.[3.73] This is of importance with reference to the prevention of expansive aggregate–silica reactions by the inclusion in the mix of pozzolanas, which remove $Ca(OH)_2$ (see p. 522). The progress of reactions is complex, but it is important to bear in mind that it is not the presence of the alkali–silica gel *per se* but the physicochemical response to the reactions that leads to the cracking of concrete.[3.66]

The alkali–silica reaction occurs only in the presence of water. The minimum relative humidity in the interior of concrete for the reaction to proceed is about 85 per cent at 20 °C (68 °F).[3.79] At higher temperatures, the reaction can take place at a somewhat lower relative humid-

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ity.[3.79] Generally, a higher temperature accelerates the progress of the alkali–silica reaction but does not increase the total expansion induced by the reaction.[3.79] The effect of temperature may be due to the fact that an increase in temperature lowers the solubility of $Ca(OH)_2$ and increases that of silica. The accelerating effect of temperature is exploited in tests on the reactivity of aggregate.

Because water is essential for the alkali–silica reaction to continue, drying out the concrete and prevention of future contact with water is an effective means of stopping the reaction; it is, in fact, the only means. Conversely, alternating wetting and drying aggravates the migration of the alkali ions, which move from the wet to the drier part of the concrete. A moisture gradient has a similar effect.[3.80]

The alkali–silica reaction is very slow, and its consequences may not manifest themselves until after many years. The reasons for this are complex, and the processes involved, related to the

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local concentration of various ions, are still debated.[3.66]

While we can predict that, with given materials, alkali–aggregate reaction will take place, it is not generally possible to estimate the deleterious effects from the knowledge of the quantities of the reactive materials alone. For instance, the actual reactivity of aggregate is affected by its particle size and porosity, which influence the area over which the reaction can take place. When the alkalis originate from cement only, their concentration at the reactive surface of aggregate will be governed by the magnitude of this surface. Within limits, the expansion of concrete made with a given reactive aggregate is greater the higher the alkali content of the cement and, for a given alkali content in the cement, the greater its fineness.[3.32]

Other factors influencing the progress of alkali–aggregate reaction include the permeability of the hydrated cement paste because this controls the movement of water and of the various

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ions, as well as of the silica gel. It can thus be seen that various physical and chemical factors make the problem of akali–aggregate reaction highly complex. In particular, the gel can change its constitution by absorption and thus exert a considerable pressure whereas, at other times, diffusion of the gel out of the confined area takes place.[3.32] It may be noted that, as the hydration of cement progresses, much of the alkalis becomes concentrated in the aqueous phase. As a consequence, pH rises and all silica minerals become soluble.[3.61]

Tests for aggregate reactivity

The preceding discussion explains why it is that, although we know that certain types of aggregate tend to be reactive, and their presence can be established by ASTM C 295-08, there is no simple way of determining whether a given aggregate will cause excessive expansion due to reaction with alkalis in the cement. Service record has generally to be relied upon, but as little as 0.5

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per cent of harmful aggregate can cause damage.[3.61] If no record is available, it is possible only to determine the potential reactivity of the aggregate but not to prove that a deleterious reaction will take place. A quick chemical test is prescribed by ASTM C 289-07: the reduction in the alkalinity of a normal solution of NaOH when placed in contact with pulverized aggregate at 80 °C (176 °F) is determined, and the amount of dissolved silica is measured. The interpretation of the result is in many cases not clear but, generally, a potentially deleterious reaction is indicated if the plotted test results falls to the right of the boundary line in Fig. 3.9 reproduced from ASTM C 289-07 but based on Mielenz and Witte's paper.[3.33] However, potentially deleterious aggregates represented by points lying above the dashed line in Fig. 3.9 may be extremely reactive with alkalis so that a relatively low expansion may result. These aggregates should therefore be tested further to determine whether their reactivity is deleterious by the mortar-bar test described be-

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low. The test is of little value with lightweight aggregates.[3.68]

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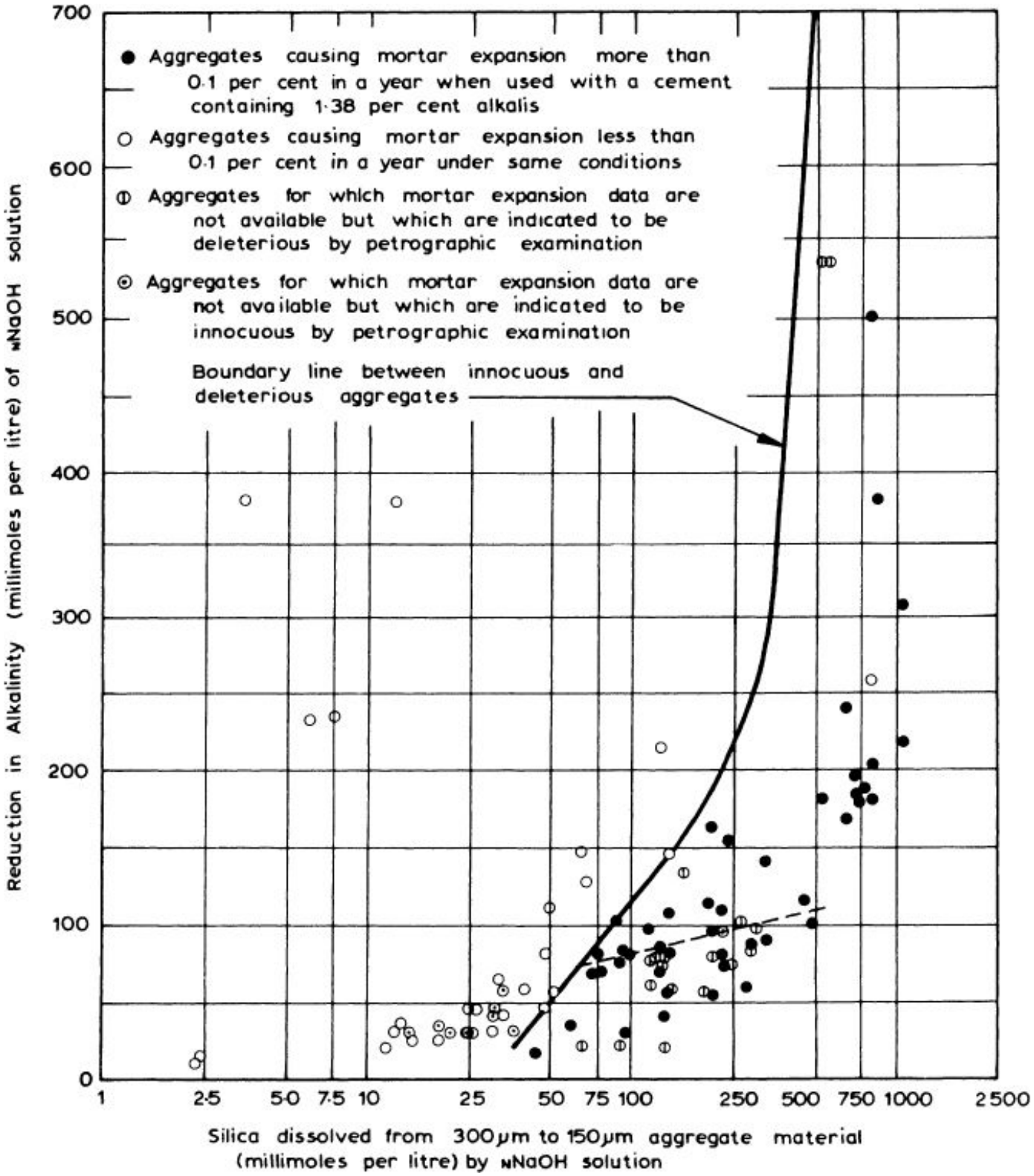


Fig. 3.9. Results of chemical test of ASTM C 289-07

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In the mortar-bar test for the physical reactivity of aggregate, prescribed by ASTM C 227-10, the suspected aggregate, crushed if need be and made up to a prescribed grading, is used in making special cement–sand mortar bars, using a cement with an equivalent alkali content of more than 0.6 per cent and preferably more than 0.8 per cent. The bars are stored over water at 38 °C (100 °F), at which temperature the expansion is more rapid and usually higher than at higher or lower temperatures.[3.34] The reaction is also accelerated by the use of a fairly high water/cement ratio. According to an appendix to ASTM C 33-08, the aggregate under test is considered harmful if it expands more than 0.10 per cent after 6 months or more than 0.05 per cent after 3 months, if a 6-month result is not available.

The mortar-bar test of ASTM C 227-10 has shown a very good correlation with field experience, but a considerable time is required before judgement on the aggregate can be made. For quartz-bearing aggregates, the test period may

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need to be as long as one year.[3.81] On the other hand, as mentioned earlier, the results of the chemical test, which is rapid, are often not conclusive. Likewise, petrographic examination, although a useful tool in identifying the mineral constituents, cannot establish that a given mineral will result in deleterious expansion. Various accelerated test methods continue to be developed but they often involve the use of a high temperature (up to 80 °C (176 °F)) which distorts the behaviour. The British Standard prescribing a test method using expansion of a concrete prism is BS 812-123 : 1999.

There is a lack of results of laboratory tests which are correlated with the field performance of concrete containing the same materials.[3.82] The probable reason for this is the very long period in service before the effects of alkali–silica reaction manifest themselves. It follows that new test methods cannot be validated in a hurry. A rapid and conclusive test for aggregate reactivity is thus still to be developed, and to use more than

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one of the existing tests is the best that can be done at the moment.

The preceding discussion of the alkali–aggregate reaction is intended to ensure an awareness of the potential problems with some aggregates. The consequences of the alkali–aggregate reaction in concrete and the means of avoiding them are discussed on p. 519, but full treatment of this vast subject cannot be included in the present book. What is important is to realize that the risk of deleterious alkali–aggregate reaction has to be considered in the selection of concrete materials.

Alkali–carbonate reaction

Another type of deleterious aggregate reaction is that between some dolomitic limestone aggregates and the alkalis in cement. The volume of the products of this reaction is smaller than the volume of the original materials so that the explanation for the deleterious reaction has to be sought in phenomena different from those in-

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volved in the alkali–silica reaction.[3.83] It is likely that the gel which is formed is subject to swelling in a manner similar to swelling clays.[3.79] Thus, under humid conditions, expansion of concrete takes place. Typically, reaction zones up to 2 mm (or 0.1 in.) are formed around the active aggregate particles. Cracking develops within these rims and leads to a network of cracks and a loss of bond between the aggregate and the cement paste.

Tests have shown that de-dolomitization, that is a change of dolomite, $CaMg(CO_3)_2$, into $CaCO_3$ and $Mg(OH)_2$, occurs. However, the reactions involved are still imperfectly understood; in particular, the role of clay in the aggregate is not clear, but expansive reaction seems to be nearly always associated with the presence of clay. Also, in expansive aggregates the dolomite and calcite crystals are very fine.[3.47] One suggestion is that the expansion is due to moisture uptake by the previously unwetted clay, the de-dolomitization being necessary only to provide access of mois-

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ture to the locked-in clay;[3.48] another suggestion is that clay increases the reactivity of the aggregate so that dolomite and calcium silicate hydrate produce $Mg(OH)_2$, silica gel, and calcium carbonate with a volume increase of about 4 per cent.[3.62] A good review of the problem is given by Walker.[3.70]

It should be stressed that only *some* dolomitic limestones cause expansive reactions in concrete. No simple test to identify them has been developed; in case of doubt, help can be obtained from an investigation of the rock texture or of rock expansion in sodium hydroxide (ASTM C 586-05). If the expansion of the rock sample in the test of ASTM C 586-05 exceeds 0.10 per cent, the length change of concrete made with the suspect aggregate and stored in moist air is determined according to ASTM C 1105-08a, which also gives guidance on the interpretation of the test results.

One distinction between the silica- and carbonate–alkali reactions which should be borne in

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mind is that, in the latter, the alkali is regenerated. It is probably for this reason that pozzolanas, including silica fume, are not effective in controlling the alkali–carbonate expansion.[3.84] However, ground granulated blastfurnace slag, which reduces the permeability of concrete (see Chapter 13) is reasonably effective.[3.84] Fortunately, reactive carbonate rocks are not very widespread and can usually be avoided.

Thermal properties of aggregate

There are three thermal properties of aggregate that may be significant in the performance of concrete: coefficient of thermal expansion, specific heat, and conductivity. The last two are of importance in mass concrete or where insulation is required, but not in ordinary structural work, and are discussed in the section dealing with the thermal properties of concrete (see p. 376).

The coefficient of thermal expansion of aggregate influences the value of such a coefficient of concrete containing the given aggregate: the

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higher the coefficient of the aggregate the higher the coefficient of the concrete, but the latter depends also on the aggregate content in the mix and on the mix proportions in general.

There is, however, another aspect of the problem. It has been suggested that if the coefficients of thermal expansion of the coarse aggregate and of the hydrated cement paste differ too much, a large change in temperature may introduce differential movement and a break in the bond between the aggregate particles and the surrounding paste. However, possibly because the differential movement is affected also by other forces, such as those due to shrinkage, a large difference between the coefficients is not necessarily detrimental when the temperature does not vary outside the range of, say, 4 to 60 °C (40 to 140 °F). Nevertheless, when the two coefficients differ by more than 5.5×10^{-6} per °C (3×10^{-6} per °F) the durability of concrete subjected to freezing and thawing may be affected.

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The coefficient of thermal expansion can be determined by means of a dilatometer devised by Verbeck and Hass[3.14] for use with both fine and coarse aggregate. The linear coefficient of thermal expansion varies with the type of parent rock, the range for the more common rocks being about 0.9×10^{-6} to 16×10^{-6} per °C (0.5×10^{-6} to 8.9×10^{-6} per °F), but the majority of aggregates lie between approximately 5×10^{-6} and 13×10^{-6} per °C (3×10^{-6} and 7×10^{-6} per °F) (see Table 3.13).[3.39] For hydrated Portland cement paste, the coefficient varies between 11×10^{-6} and 16×10^{-6} per °C (6×10^{-6} and 9×10^{-6} per °F), but values up to 20.7×10^{-6} per °C (11.5×10^{-6} per °F) have also been reported, the coefficient varying with the degree of saturation. Thus, a serious difference in coefficients occurs only with aggregates of a very low expansion; these are certain granites, limestones and marbles.

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Table 3.13. Linear Coefficient of Thermal Expansion of Different Rock Types[3.39]

Rock type	*Thermal coefficient of linear expansion*	
	10^{-6} per °C	*10^{-6} per °F*
Granite	1.8–11.9	1.0–6.6
Diorite, andesite	4.1–10.3	2.3–5.7
Gabbro, basalt, diabase	3.6–9.7	2.0–5.4
Sandstone	4.3–13.9	2.4–7.7
Dolomite	6.7–8.6	3.7–4.8
Limestone	0.9–12.2	0.5–6.8
Chert	7.3–13.1	4.1–7.3
Marble	1.1–16.0	0.6–8.9

If extreme temperatures are expected, the detailed properties of any given aggregate have to be known. For instance, quartz undergoes inversion at 574 °C and expands suddenly by 0.85 per cent. This would disrupt the concrete, and for this reason fire-resistant concrete is never made with quartz aggregate.

Sieve analysis

This somewhat grandiose name is given to the simple operation of dividing a sample of aggregate into fractions, each consisting of particles of the same size. In practice, each fraction contains

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particles between specific limits, these being the openings of standard test sieves.

The test sieves used for concrete aggregate have square openings and their properties are prescribed by BS 410-1 and 2 : 2000 and ASTM E 11-09. In the latter standard, the sieves can be described by the size of the opening (in inches) for larger sizes, and by the number of openings per lineal inch for sieves smaller than about $\frac{1}{4}$ in. Thus a No. 100 test sieve has 100 × 100 openings in each square inch. The standard approach is to designate the sieve sizes by the nominal aperture size in millimetres or micrometres.

Sieves smaller than 4 mm (0.16 in.) are normally made of wire cloth although, if required, this can be used up to 16 mm (0.62 in.). The wire cloth is made of phosphor bronze but, for some coarser sieves, brass and mild steel can also be used. The screening area, i.e. the area of the openings as a percentage of the gross area of the sieve, varies between 28 and 56 per cent, being larger for large openings. Coarse test sieves (4

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mm (0.16 in.) and larger) are made of perforated plate, with a screening area of 44 to 65 per cent.

All sieves are mounted in frames which can nest. It is thus possible to place the sieves one above the other in order of size with the largest sieve at the top, and the material retained on each sieve after shaking represents the fraction of aggregate coarser than the sieve in question but finer than the sieve above. Frames 200 mm (8 in.) in diameter are used for 5 mm ($\frac{3}{16}$ in.) or smaller sizes, and 300 or 400 mm (12 or 18 in.) diameter frames for 5 mm ($\frac{3}{16}$ in.) and larger sizes. It may be remembered that 5 mm (or $\frac{3}{16}$ in., No. 4 ASTM) or 4 mm is the dividing line between the fine and coarse aggregates.

The sieves used for concrete aggregate consist of a series in which the clear opening of any sieve is approximately one-half of the opening of the next larger sieve size. The BS test sieve sizes in Imperial units for this series were as follows: 3 in., $1\frac{1}{2}$ in., $\frac{3}{4}$ in., $\frac{3}{8}$ in., $\frac{3}{16}$ in., Nos. 7,

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14, 25, 52, 100, and 200, and results of tests on those sieves are still used. Table 3.14 gives the traditional sieve sizes according to their fundamental description by aperture in millimetres or micrometres and also the previous British and ASTM designations and approximate apertures in inches.

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Table 3.14. Traditional American and British Sieve Sizes

Aperture mm or μm	*Approximate Imperial equivalent in.*	*Previous designation of nearest size*	
		BS	*ASTM*
125 mm	5	—	5 in.
106 mm	4.24	4 in.	4.24 in.
90 mm	3.5	$3\frac{1}{2}$ in.	$3\frac{1}{2}$ in.
75 mm	3	3 in.	3 in.
63 mm	2.5	$2\frac{1}{2}$ in.	$2\frac{1}{2}$ in.
53 mm	2.12	2 in.	2.12 in.
45 mm	1.75	$1\frac{3}{4}$ in.	$1\frac{3}{4}$ in.
37.5 mm	1.50	$1\frac{1}{2}$ in.	$1\frac{1}{2}$ in.
31.5 mm	1.25	$1\frac{1}{4}$ in.	$1\frac{1}{4}$ in.
26.5 mm	1.06	1 in.	1.06 in.
22.4 mm	0.875	$\frac{7}{8}$ in.	$\frac{7}{8}$ in.
19.0 mm	0.750	$\frac{3}{4}$ in.	$\frac{3}{4}$ in.
16.0 mm	0.625	$\frac{5}{8}$ in.	$\frac{5}{8}$ in.
13.2 mm	0.530	$\frac{1}{2}$ in.	0.530 in.
11.2 mm	0.438	—	$\frac{7}{16}$ in.
9.5 mm	0.375	$\frac{3}{8}$ in.	$\frac{3}{8}$ in.
8.0 mm	0.312	$\frac{5}{16}$ in.	$\frac{5}{16}$ in.
6.7 mm	0.265	$\frac{1}{4}$ in.	0.265 in.
5.6 mm	0.223	—	No. $3\frac{1}{2}$
4.75 mm	0.187	$\frac{3}{16}$ in.	No. 4
4.00 mm	0.157	—	No. 5
3.35 mm	0.132	No. 5	No. 6
2.80 mm	0.111	No. 6	No. 7
2.36 mm	0.0937	No. 7	No. 8
2.00 mm	0.0787	No. 8	No. 10

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1.70 mm	0.0661	No. 10	No. 12
1.40 mm	0.0555	No. 12	No. 14
1.18 mm	0.0469	No. 14	No. 16
1.00 mm	0.0394	No. 16	No. 18
850 μm	0.0331	No. 18	No. 20
710 μm	0.0278	No. 22	No. 25
600 μm	0.0234	No. 25	No. 30
500 μm	0.0197	No. 30	No. 35
425 μm	0.0165	No. 36	No. 40
355 μm	0.0139	No. 44	No. 45
300 μm	0.0117	No. 52	No. 50
250 μm	0.0098	No. 60	No. 60
212 μm	0.0083	No. 72	No. 70
180 μm	0.0070	No. 85	No. 80
150 μm	0.0059	No. 100	No. 100
125 μm	0.0049	No. 120	No. 120
106 μm	0.0041	No. 150	No. 140
90 μm	0.0035	No. 170	No. 170
75 μm	0.0029	No. 200	No. 200
63 μm	0.0025	No. 240	No. 230
53 μm	0.0021	No. 300	No. 270
45 μm	0.0017	No. 350	No. 325
38 μm	0.0015	—	No. 400
32 μm	0.0012	—	No. 450

For determination of oversize and undersize aggregate, and especially for research work on aggregate grading, additional sieve sizes are required. The full sequence of test sieves is based theoretically on the ratio of $\sqrt[4]{2}$ for the openings of two consecutive sieves, 1 mm size being the base. However, both the British (BS 410 : 1986) and American (ASTM E 11-09) sieves have been standardized generally in accordance with the R40/3 sieve series of the International Standards

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Organization. Not all of these sizes form a true geometric series but follow 'preferred numbers'. British Standard BS 410-1 : 2000 also uses some sieve sizes of the R20 series of the International Standards Organization (ISO 565 – 1990). This series covers the range of sizes from 125 mm to 63 μm in steps with a ratio of approximately 1.2, based on the 1 mm size. There exists also a European Standard BS EN 933-2 : 1996, which uses the same sizes as ISO 6274 – 1982. The various standard sieve sizes are shown in Table 3.15. For grading purposes, the sieve sizes normally used are: 75.0, 50.0, 37.5, 20.0, 10.0, 5.00, 2.36, 1.18 mm, and 600, 300, and 150 μm.

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Table 3.15. Standard Sieve Sizes for Aggregate Given by Various Standards (mm or μm)

BS 410 : 1986	*BS 812-103.1 : 1985 (2000)*	*BS EN 933-2 : 1996*	*ASTM E 11-87 (2009)**
125.0			125
			100
90.0			
	75.0		75.0
63.0	63.0	63.0	
	50.0		50.0
45.0			
	37.5		37.5
31.5		31.5	
	28.0		25.0
22.4			
	20.0		
			19.0
16.0		16.0	
	14.0		
			12.5
11.2	10.0		
			9.5
8.00		8.00	
	6.30		6.30
5.60			
	5.00		
			4.75
4.00		4.00	
	3.35		
2.80			
	2.36		2.36
2.00		2.00	
	1.70		

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1.40			
	1.18		1.18
1.00		1.00	
	850		
710			
	600		600
500		500	
	425		
355			
	300		300
250		250	
	212		
180			
	150		150
125		125	
90			
	75		75
63		63	
45			
32			

*Selected common values.

We can thus see that in discussing aggregate grading we have to contend with two sets of sieve sizes. In this book, results of measurements made with Imperial size sieves will be reported by the exact metric equivalent, but grading curves for mix proportioning purposes (see Chapter 14) will, wherever available, be based on current ASTM or BS metric sieve sizes.

Before the sieve analysis is performed, the aggregate sample has to be air-dried in order to avoid lumps of fine particles being classified as large particles and also to prevent clogging of

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the finer sieves. The minimum masses of the reduced samples for sieving, as recommended by BS 812-103.1 : 1985 (2000), are given in Table 3.16, and Table 3.17 shows the maximum mass of material with which each sieve can cope. If this mass is exceeded on a sieve, material which is really finer than this sieve may be included in the portion retained. The material on the sieve in question should, therefore, be split into two parts and each should be sieved separately. The actual sieving operation can be performed by hand, each sieve in turn being shaken until not more than a trace continues to pass. The movement should be backwards and forwards, sideways left and right, circular clockwise and anticlockwise, all these motions following one another so that every particle 'has a chance' of passing through the sieve. In most laboratories a sieve shaker is available, usually fitted with a time switch so that uniformity of the sieving operation can be ensured. None the less, care is necessary in order to make sure that no sieve is overloaded (see Table 3.17). The amount of material smaller than 75 μm

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can best be determined by wet sieving in accordance with BS 812-103.1 : 1985 (2000) or ASTM C 117-04.

Table 3.16. Minimum Mass of Sample for Sieve Analysis According to BS 812-103.1 : 1985 (2000)

Nominal size of material mm	*Minimum mass of sample to be taken for sieving kg*
63	50
50	35
40	15
28	5
20	2
14	1
10	0.5
6 or 5 or 3	0.2
Less than 3	0.1

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Table 3.17. Maximum Mass to be Retained at the Completion of Sieving According to BS 812-103.1 : 1985 (2000)

BS sieve size		*Maximum mass (kg) for sieve of diameter*		
mm	*μm*	*450 mm*	*300 mm*	*200 mm*
50.0		14	5	
37.5		10	4	
28.0		8	3	
20.0		6	2.5	
14.0		4	2	
10.0		3	1.5	
6.30		2	1	
5.00		1.5	0.75	0.350
3.35		1	0.55	0.250
2.36			0.45	0.200
1.70			0.375	0.150
1.18			0.300	0.125
	850		0.260	0.115
	600		0.225	0.100
	425		0.180	0.080
	300		0.150	0.065
	212		0.130	0.060
	150		0.110	0.050
	75		0.075	0.030

The results of a sieve analysis are best reported in tabular form, as shown in Table 3.18. Column (2) shows the mass retained on each sieve. This is expressed as a percentage of the total mass of the sample and is shown in column (3). Now, working from the finest size upwards, the *cumulative* percentage (to the nearest 1 per

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cent) passing each sieve can be calculated (column (4)), and it is this percentage that is used in the plotting of grading curves.

Table 3.18. Example of Sieve Analysis

Sieve size		*Mass retained g*	*Percentage retained*	*Cumulative percentage passing*	*Cumulative percentage retained*
BS	*ASTM*				
	(1)	*(2)*	*(3)*	*(4)*	*(5)*
10.0 mm	$\frac{3}{8}$ in.	0	0.0	100	0
5.00 mm	4	6	2.0	98	2
2.36 mm	8	31	10.1	88	12
1.18 mm	16	30	9.8	78	22
600 μm	30	59	19.2	59	41
300 μm	50	107	34.9	24	76
150 μm	100	53	17.3	7	93
<150 μm	<100	21	6.8	—	—
		Total = 307		Total = 246 Fineness modulus = 2.46	

Grading curves

The results of a sieve analysis can be grasped much more easily if represented graphically and, for this reason, grading charts are very extensively used. By using a chart, it is possible to see at a glance whether the grading of a given sample conforms to that specified, or is too coarse or too fine, or deficient in a particular size.

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In the grading chart commonly used, the ordinates represent the cumulative percentage passing and the abscissae show the sieve opening plotted to a logarithmic scale. Since the openings of sieves in a standard series are in the ratio of $\frac{1}{2}$, a logarithmic plot shows these openings at a constant spacing. This is illustrated in Fig. 3.10 which represents the data of Table 3.18.

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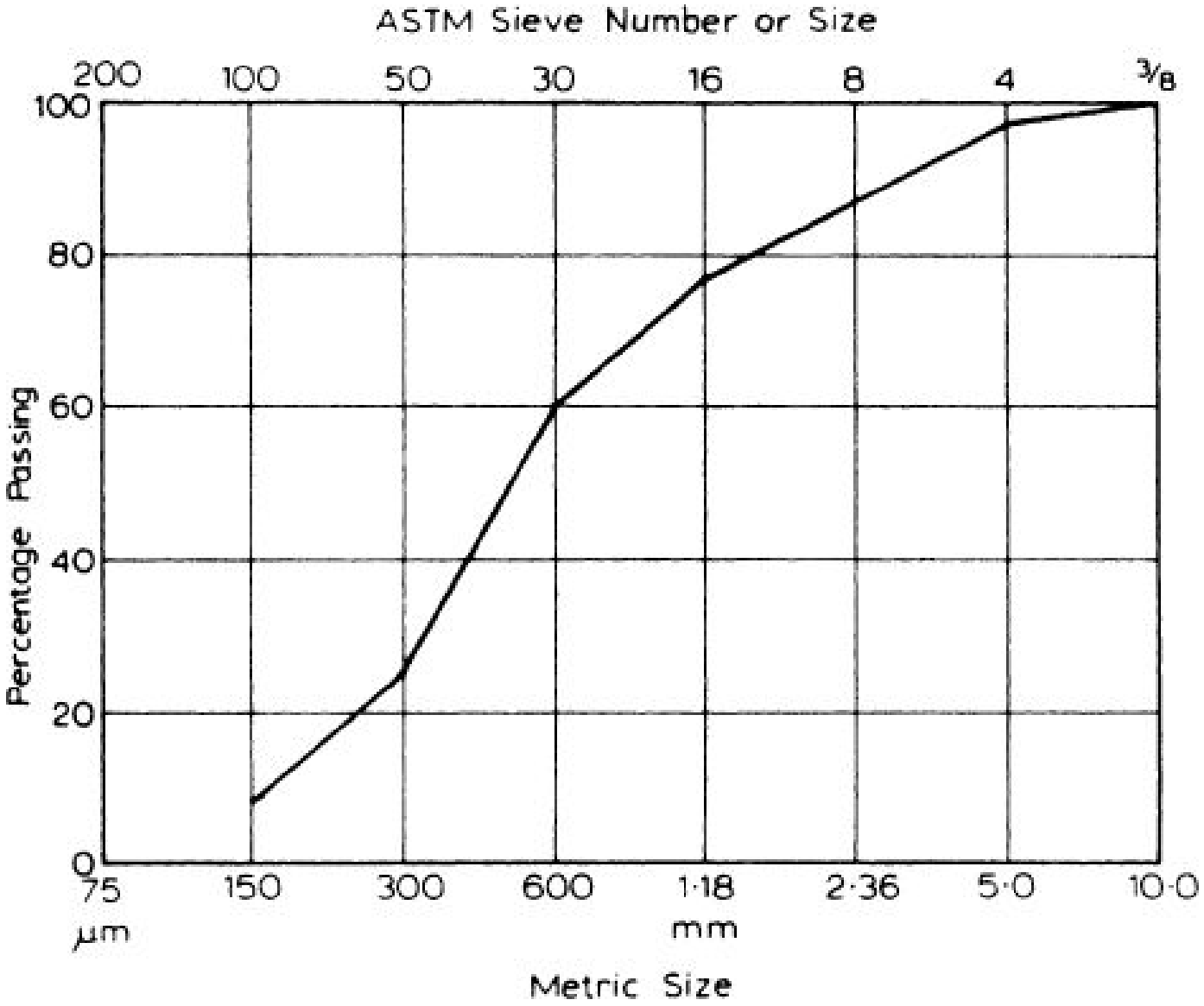


Fig. 3.10. Example of a grading curve (see Table 3.18)

It is convenient to choose a scale such that the scale spacing between two adjacent sieve sizes is approximately equal to the 20 per cent interval on the ordinate scale; a visual comparison of different grading curves can then be made from memory.

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Fineness modulus

A single factor computed from the sieve analysis is sometimes used, particularly in the United States. This is the fineness modulus, defined as $\frac{1}{100}$ of the sum of the cumulative percentages retained on the sieves of the standard series: 150, 300, 600 μm, 1.18, 2.36, 5.00 mm (ASTM Nos. 100, 50, 30, 16, 8, 4) and up to the largest sieve size used. It should be remembered that, when all the particles in a sample are coarser than, say, 600 μm (No. 30 ASTM), the cumulative percentage retained on 300 μm (No. 50 ASTM) sieve should be entered as 100; the same value, of course, would be entered for 150 μm (No. 100). The value of the fineness modulus is higher the coarser the aggregate (see column (5), Table 3.18).

The fineness modulus can be looked upon as a weighted average size of a sieve on which the material is retained, the sieves being counted from the finest. Popovics[3.49] showed it to be a logarithmic average of the particle size distribu-

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tion. For instance, a fineness modulus of 4.00 can be interpreted to mean that the fourth sieve, 1.18 mm (No. 16 ASTM) is the average size. However, it is clear that one parameter, the average, cannot be representative of a distribution: thus the same fineness modulus can represent an infinite number of totally different size distributions or grading curves. The fineness modulus cannot, therefore, be used as a single description of the grading of an aggregate, but it is valuable for measuring slight variations in the aggregate from the same source, e.g. as a day-to-day check. Nevertheless, within certain limitations, the fineness modulus gives an indication of the probable behaviour of a concrete mix made with aggregate having a certain grading, and the use of the fineness modulus in assessment of aggregates and in mix proportioning has many supporters.[3.49]

Grading requirements

We have seen how to find the grading of a sample of aggregate, but it still remains to determine

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whether or not a particular grading is suitable. A related problem is that of combining fine and coarse aggregates so as to produce a desired grading. What, then, are the properties of a 'good' grading curve?

Because the strength of fully compacted concrete with a given water/cement ratio is independent of the grading of the aggregate, grading is, in the first instance, of importance only in so far as it affects workability. As, however, achieving the strength corresponding to a given water/cement ratio requires full compaction, and this can be obtained only with a sufficiently workable mix, it is necessary to produce a mix that can be compacted to a maximum density with a reasonable amount of work.

It should be stated at the outset that there is no one *ideal* grading curve but a compromise is aimed at. Apart from the physical requirements, the economic aspects must not be forgotten: concrete has to be made of materials which can be

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produced cheaply so that no narrow limits can be imposed on aggregate.

It has been suggested that the main factors governing the desired aggregate grading are: the surface area of the aggregate, which determines the amount of water necessary to wet all the solids; the relative volume occupied by the aggregate; the workability of the mix; and the tendency to segregation.

Segregation is discussed on p. 205, but it should be observed here that the requirements of workability and absence of segregation tend to be partially opposed to one another: the easier it is for the particles of different sizes to pack, smaller particles passing into the voids between the larger ones, the easier it is also for the small particles to be shaken out of the voids, i.e. to segregate in the dry state. In actual fact, it is the mortar (i.e. a mixture of sand, cement and water) that should be prevented from passing freely out of the voids in the coarse aggregate. It is also essential for the voids in the combined aggregate to be suf-

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ficiently small to prevent the fresh cement paste from passing through and separating out.

The problem of segregation is thus rather similar to that of filters, although the requirements in the two cases are of course diametrically opposite: for the concrete to be satisfactory it is essential that segregation be avoided.

There is a further requirement for a mix to be satisfactorily cohesive and workable: it must contain a sufficient amount of material smaller than a 300 μm (No. 50 ASTM) sieve. Because the cement particles are included in this material, a richer mix requires a lower content of fine aggregate than a lean mix. If the grading of fine aggregate is such that it is deficient in finer particles, increasing the fine/coarse aggregate ratio may not prove a satisfactory remedy, as it may lead to an excess of middle sizes and possibly to harshness. (A mix is said to be harsh when one size fraction is present in excess, as shown by a steep step in the middle of a grading curve, so that particle interference results.) This need for

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an adequate amount of fines (provided they are structurally sound) explains why minimum contents of particles passing 300 μm (No. 50 ASTM) and sometimes also 150 μm (No. 100) sieves are laid down, as for instance in Tables 3.22 and 3.23 (p. 167). However, it is now thought that the U.S. Bureau of Reclamation requirements of Table 3.23 for the minimum percentage of particles passing the 300 and 150 μm (Nos 50 and 100 ASTM) sieves are too high.

It may be further added that all the cementitious materials automatically provide a certain amount of 'ultra-fines'. The ultra-fines can be, therefore, taken as materials smaller than 125 μm of all provenance, that is, aggregate, filler, and cement. However, there are some differences in behaviour in that the initial hydration of cement rapidly removes some water from the mix, while the other particles are inert. The volume of entrained air can be taken as equivalent to one-half the volume of fines. The German Standard DIN 1045 : 1988[3.86] establishes the particle size of

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125 μm as the criterion for ultra-fine material. No minima of ultra-fines are specified because they are normally found in the materials used, but the presence of adequate ultra-fines is essential for pumped concrete and for concrete to be placed in thin sections or with congested reinforcement, and also for water-retaining structures. On the other hand, an excessive amount of ultra-fines is harmful from the point of view of resistance to freezing and thawing and to de-icing salts as well as of resistance to abrasion. A maximum total content of 350 kg per cubic metre of concrete is prescribed for mixes with a cement content of not more than 300 kg/m^3. The maximum of ultra-fines is 400 kg/m^3 when the cement content is 350 kg/m^3; higher amounts of ultra-fines are permitted at higher cement contents. These values apply to mixes with a maximum aggregate size of 16 to 63 mm. The beneficial effect of ultra-fines smaller than 50 μm on the water requirement of fresh concrete, and therefore on strength, has been confirmed.[3.85]

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The requirement that the aggregate occupies as large a relative volume as possible is, in the first instance, an economic one, the aggregate being cheaper than the cement paste, but there are also strong technical reasons why too rich a mix is undesirable. It is also believed that the greater the amount of solid particles that can be packed into a given volume of concrete the higher its density and therefore the higher its strength. This maximum density theory has led to the advocacy of grading curves parabolic in shape, or in part parabolic and then straight (when plotted to a natural scale), as shown in Fig. 3.11. It was found, however, that the aggregate graded to give maximum density makes a harsh and somewhat unworkable mix. The workability is improved when there is an excess of paste above that required to fill the voids in the sand, and also an excess of mortar (fine aggregate plus cement paste) above that required to fill the voids in the coarse aggregate.

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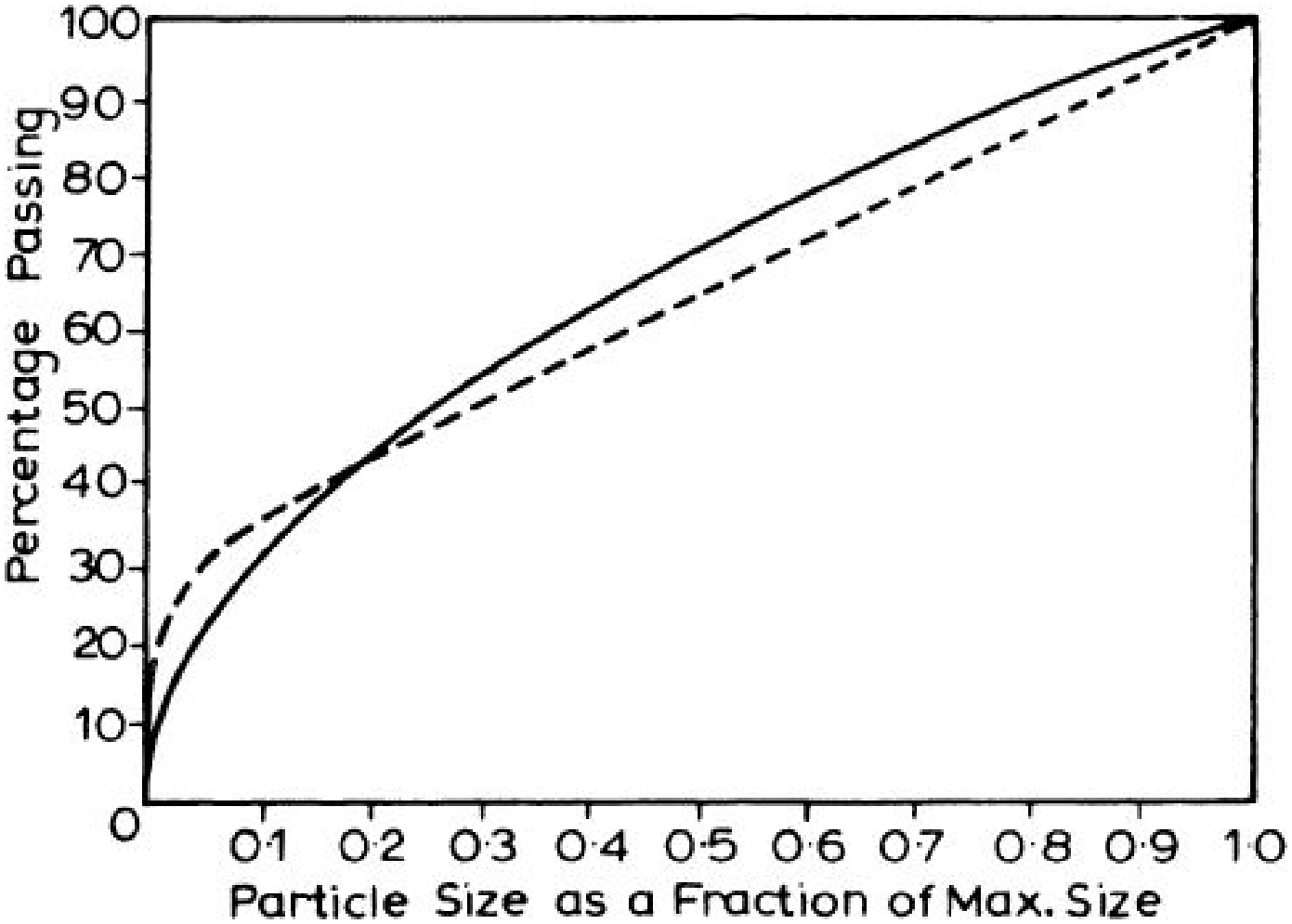


Fig. 3.11. Fuller's grading curves

The concept of an 'ideal' grading curve, such as that shown in Fig. 3.11, still finds favour, although somewhat varying shapes of 'ideal' curves are recommended by different researchers.[3.87]

One 'ideal' grading derived from the asphalt industry, in which it is important to minimize the volume of the binder, is as follows. A graph is

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constructed in which the ordinate is the cumulative percentage passing and the abscissa represents the sieve size raised to the power of 0.45. A straight line is drawn on this graph connecting a point corresponding to the largest sieve size on which some aggregate is retained, to a point corresponding to the sieve size onto which no more aggregate arrives in sieving. The 'ideal' grading should follow this line, except that the percentage passing from 600 μm (No. 30 ASTM) sieve downwards should fall below the straight line, which does not take into account the presence of cement – also a fine material. It is claimed that gradings which do not swing widely above and below the straight line produce dense concrete, but the '0.45 power grading curve' approach is not proven and not widely used.

The practical problem is that aggregates from different sources, even if nominally of the same grading, vary in the actual distribution of particle size within a given size fraction, as well as in other properties of the particles such as shape and

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texture. It has to be added that the total volume of voids in concrete is reduced when the range of particle sizes from the maximum aggregate size downward is as large as possible, that is, if extremely fine particles are included in the mix; silica fume, which is one such material, is considered on p. 86.

Let us now consider the surface area of the aggregate particles. The water/cement ratio of the mix is generally fixed from strength considerations. At the same time, the amount of the fresh cement paste has to be sufficient to cover the surface of all the particles so that the lower the surface area of the aggregate the less paste, and therefore the less water, is required.

Taking for simplicity a sphere of diameter D as representative of the shape of the aggregate, we have the ratio of the surface area to volume of $6/D$. This ratio of the surface of the particles to their volume (or, when the particles have a constant specific gravity, to their mass) is called specific surface. For particles of a different shape,

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a coefficient other than $6/D$ would be obtained but the surface area is still inversely proportional to the particle size, as shown in Fig. 3.12 reproduced from Shacklock and Walker's report.[3.15] It should be noted that a logarithmic scale is used for both the ordinates and the abscissae because the sieve sizes are in geometrical progression.

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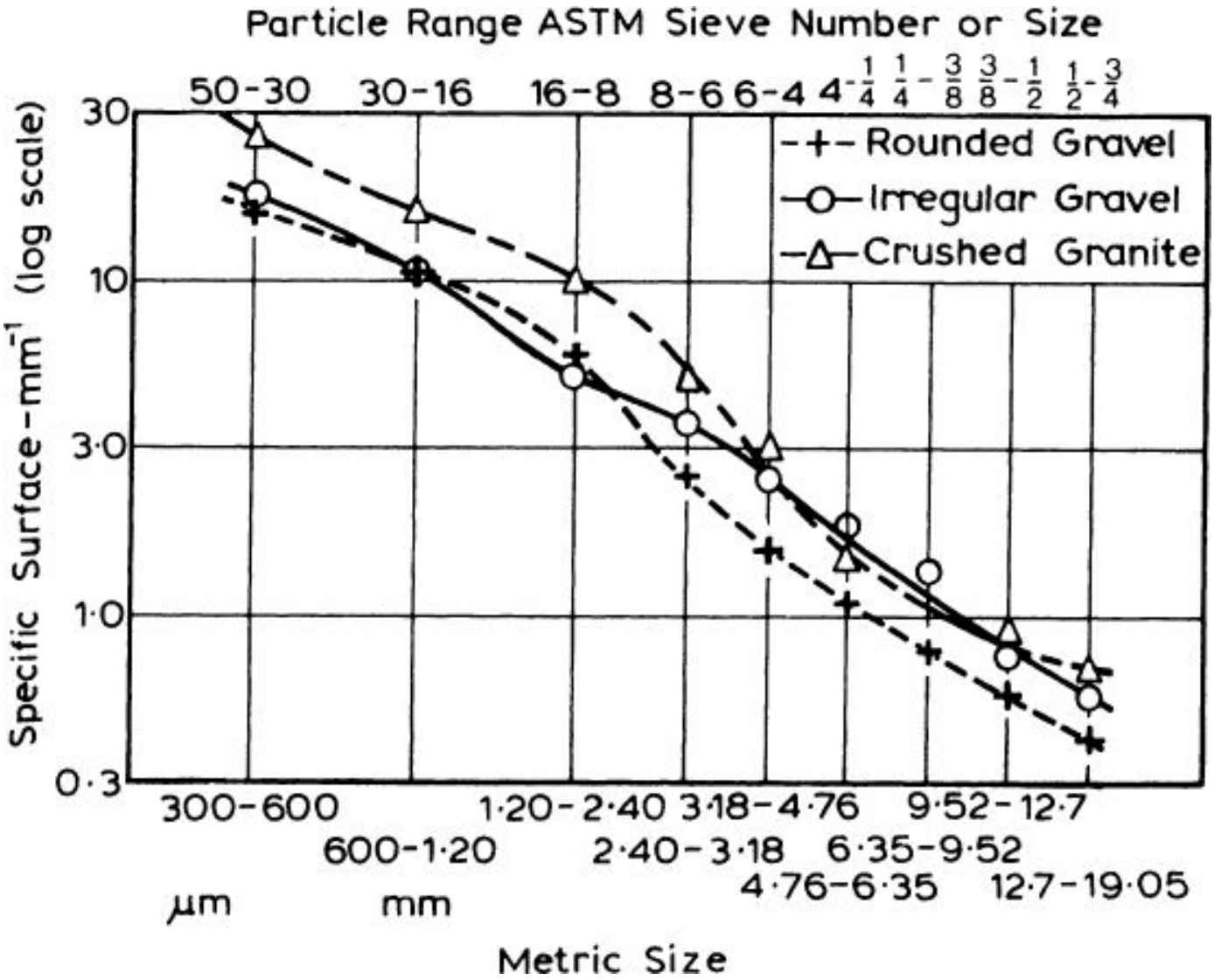


Fig. 3.12. Relation between specific surface and particle size[3.15]

In the case of graded aggregate, the grading and the overall specific surface are related to one another, although of course there are many grading curves corresponding to the same specific surface. If the grading extends to a larger maximum aggregate size, the overall specific surface

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is reduced and the water requirement decreases, but the relation is not linear. For instance, increasing the maximum aggregate size from 10 mm to 63 mm ($\frac{3}{8}$ in. to $2\frac{1}{2}$ in.) can, under certain conditions, reduce the water requirement for a constant workability by as much as 50 kg per cubic metre of concrete (85 lb/yd^3). The corresponding decrease in the water/cement ratio may be as much as 0.15.[3.16] Some typical values are shown in Fig. 3.13.

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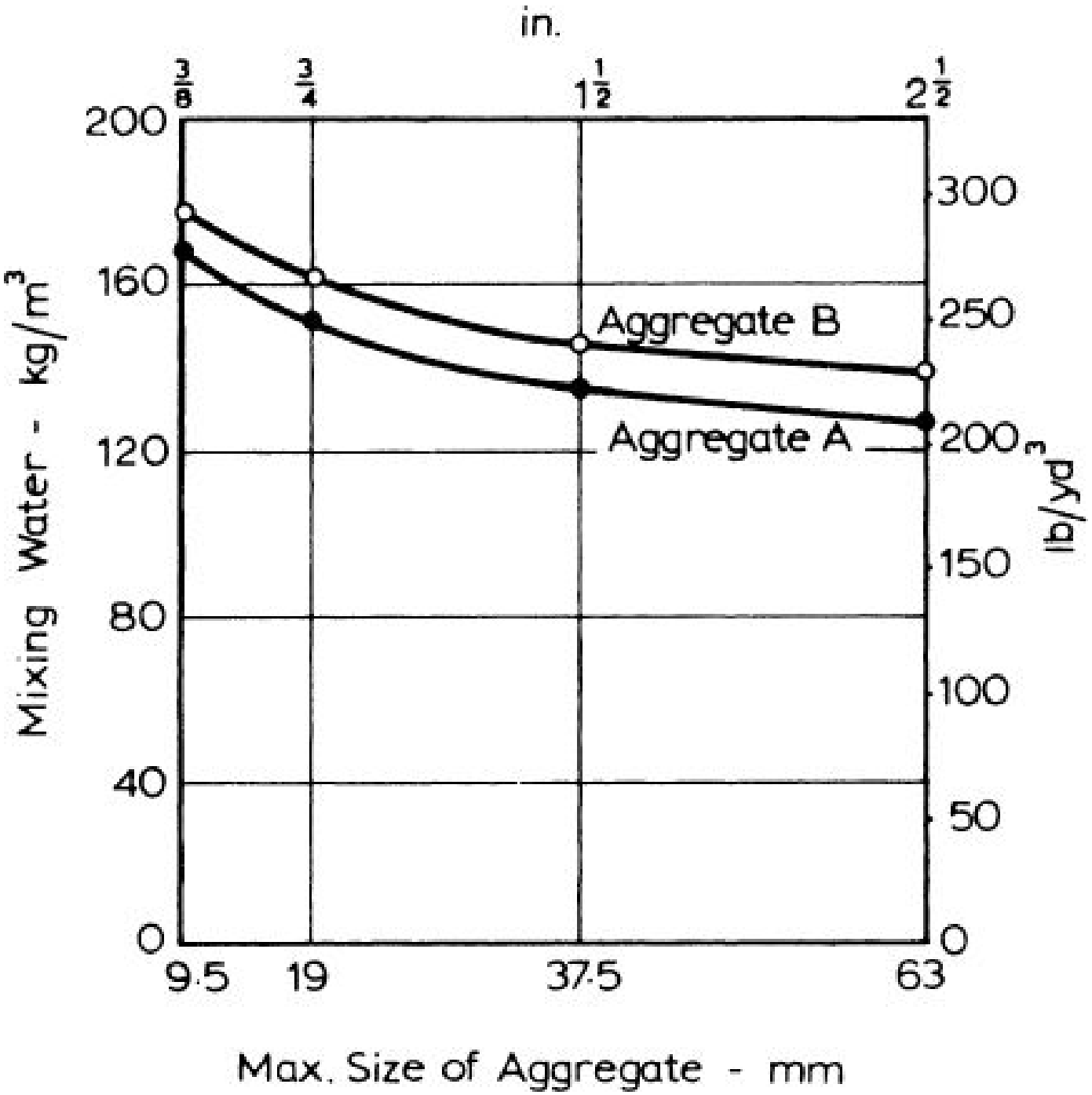


Fig. 3.13. Influence of maximum size of aggregate on mixing water requirement for a constant slump[3.16]

The practical limitations of the maximum size of aggregate that can be used under given circum-

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stances and the problem of influence of the maximum size on strength in general are discussed on p. 174.

It can be seen that, having chosen the maximum size of aggregate and its grading, we can express the total surface area of the particles using the specific surface as a parameter, and it is the total surface of the aggregate that determines the water requirement or the workability of the mix. Mix design on the basis of the specific surface of the aggregate was first suggested by Edwards[3.50] as far back as 1918, and interest in this method was renewed 40 years later. Specific surface can be determined using the water permeability method,[3.17] but no simple field test is available, and a mathematical approach is made difficult by the variability in the shape of different aggregate particles.

This, however, is not the only reason why the selection of mix proportions on the basis of the specific surface of aggregate is not universally recommended. The application of surface area cal-

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culations was found to break down for aggregate particles smaller than about 150 μm (No. 100 ASTM) sieve, and for cement. These particles, and also some larger sand particles, appear to act as a lubricant in the mix and do not seem to require wetting in quite the same way as coarse particles. An indication of this was found in some tests by Glanville *et al*.[3.18]

Because specific surface gives a somewhat misleading picture of the workability to be expected (largely owing to an overestimate of the effect of fine particles), an empirical surface index was suggested by Murdock[3.19] and its values as well as those of the specific surface are given in Table 3.19.

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Table 3.19. Relative Values of Surface Area and Surface Index

Particle size fraction	*ASTM sieve No.*	*Relative surface area*	*Murdock's surface index*[3.19]
76.2–38.1 mm	3–1½ in.	½	½
38.1–19.05 mm	1½–¾ in.	1	1
19.05–9.52 mm	¾–⅜ in.	2	2
9.52–4.76 mm	⅜–3/16 in.	4	4
4.76–2.40 mm	3/16 in.–8	8	8
2.40–1.20 mm	8–16	16	12
1.20 mm–600 μm	16–30	32	15
600–300 μm	30–50	64	12
300–150 μm	50–100	128	10
<150 μm	<100	—	1

The overall effect of the surface area of an aggregate of given grading is obtained by multiplying the percentage mass of any size fraction by the coefficient corresponding to that fraction, and summing all the products. According to Murdock,[3.19] the surface index (modified by an angularity index) should be used, and in fact the values of this index are based on empirical results. On the other hand, Davey[3.20] found that, for the same total specific surface of the aggregate, the water requirement and the compressive strength of the concrete are the same for very wide limits

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of aggregate grading. This applies both to continuously- and gap-graded aggregate, and in fact three of the four gradings listed in Table 3.20, reproduced from Davey's paper, are of the gap type.

Table 3.20. Properties of Concretes Made with Aggregates of the Same Specific Surface[3.20]

Size fraction	Aggregate grading, per cent							Specific surface m^2/kg	Water/cement ratio	Compressive strength				Modulus of rupture			
	300–150 μm	600–300 μm	1.20 mm–600 μm	2.40–1.20 mm	4.76–2.40 mm	9.52–4.76 mm	19.05–9.52 mm			7 days		28 days		7 days		28 days	
										MPa	psi	MPa	psi	MPa	psi	MPa	psi
ASTM	50–100	30–50	16–30	8–16	$\frac{3}{8}$–8	$\frac{3}{16}$–$\frac{3}{8}$	$\frac{3}{4}$–$\frac{3}{16}$										
Grading																	
A	11.2	11.2	11.2	11.2	11.2	22.0	22.0	3.2	0.575	23.7	3440	32.9	4770	3.72	539	4.38	636
B	12.9	12.9	12.9	0	0	30.6	30.7	3.2	0.575	24.2	3510	32.3	4690	3.74	543	4.48	651
C	15.4	15.4	0	0	0	34.6	34.6	3.2	0.575	24.6	3570	32.8	4760	3.84	557	4.54	659
D	25.4	0	0	0	0	0	74.6	3.2	0.575	23.3	3380	32.1	4650	3.46	502	4.16	603

An increase in the specific surface of the aggregate for a constant water/cement ratio has been found to lead to a lower strength of concrete, as shown for instance in Table 3.21, showing Newman and Teychenné's[3.21] results. The reasons for this are not quite clear, but it is possible that a reduction in density of the concrete consequent upon an increase in the fineness of

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the natural sand is instrumental in lowering the strength.[3.22]

Table 3.21. Specific Surface of Aggregate and Strength of Concrete for a 1:6 Mix with a Water/*Cement Ratio of 0.60*[3.21]

Specific surface of aggregate m^2/kg	*28-day compressive strength of concrete*		*Density of fresh concrete*	
	MPa	*psi*	kg/m^3	lb/ft^3
2.24	36.1	5240	2330	145.5
2.80	34.9	5060	2325	145.1
4.37	30.3	4390	2305	144.0
5.71	27.5	3990	2260	141.0

Workability does not seem to be a direct function of the specific surface area of aggregate; indeed, Hobbs[3.88] showed that concrete mixes containing fine aggregate with significantly varying grading led to a similar slump or compacting factor, but the percentage of fine aggregate in the total aggregate was adjusted. It seems then that the surface area of the aggregate is an important factor in determining the workability of the mix,

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but the exact role played by the finer particles has by no means been ascertained.

The type gradings of Road Note No. 4[3.23] which is an early fundamental contribution to understanding aggregate grading, represent different values of overall specific surface. For instance, when river sand and gravel are used, the four grading curves, Nos 1 to 4, of Fig. 3.14 correspond to the specific surface of 1.6, 2.0, 2.5, and 3.3 m^2/kg, respectively.[3.21] In practice, when trying to approximate type gradings, the properties of the mix will remain largely unaltered when compensation of a small deficiency of fines by a somewhat larger excess of coarser particles is applied, but the departure must not be too great. The deficiency and excess are, of course, mutually interchangeable in the above statement.

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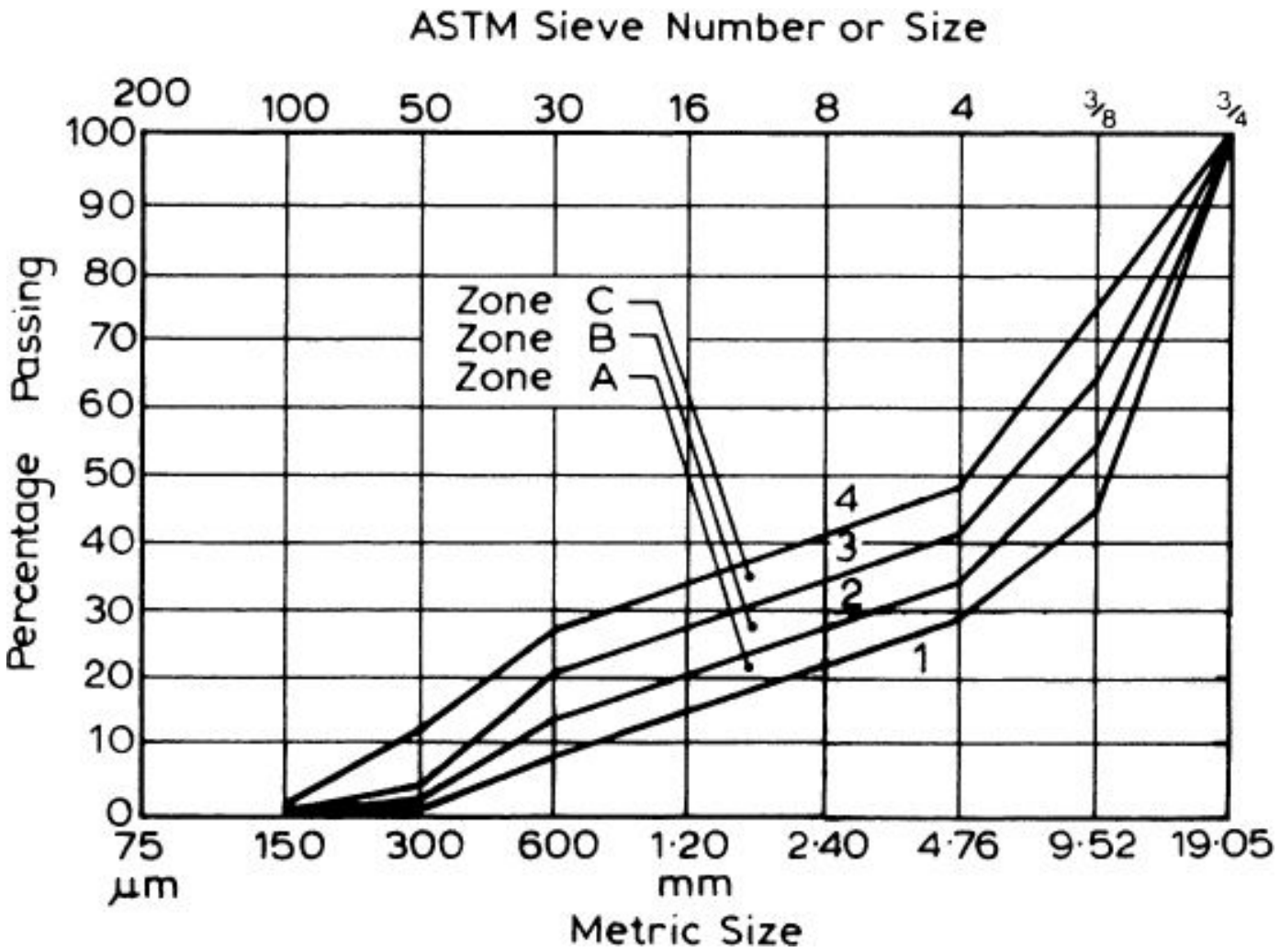


Fig. 3.14. Road Note No. 4 type grading curves for 19.05 mm ($\frac{3}{4}$ in.) aggregate[3.23] (Crown copyright)

There is no doubt then that the grading of aggregate is a major factor in the workability of a concrete mix. Workability, in turn, affects the water and cement requirements, controls segregation, has some effect on bleeding, and influences the placing and finishing of the concrete. These

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factors represent the important characteristics of fresh concrete and affect also its properties in the hardened state: strength, shrinkage, and durability.

Grading is thus of vital importance in the proportioning of concrete mixes, but its exact role in mathematical terms has not yet been established, and the behaviour of this type of semi-liquid mixture of granular materials is still imperfectly understood. Moreover, while ensuring appropriate grading of aggregate is of considerable importance, arbitrary imposition of limits which are uneconomic, or even near impossible, in a given location is inappropriate.

Finally, it must be remembered that far more important than devising a ‘good’ grading is ensuring that the grading is kept constant; otherwise, variable workability results and, when this is corrected at the mixer by a variation in the water content, concrete of variable strength is obtained.

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Practical gradings

From the brief review in the previous section, it can be seen how important it is to use aggregate with a grading such that a reasonable workability and minimum segregation are obtained. The importance of the latter requirement cannot be overemphasized: a workable mixture which *could* produce a strong and economical concrete will result in honeycombed, weak, not durable and variable end product if segregation takes place.

The process of calculation of the proportions of aggregates of different size to achieve the desired grading comes within the scope of mix proportioning and is described in Chapter 14. Here, the properties of some 'good' grading curves will be discussed. It should be remembered, however, that in practice the aggregate available locally or within an economic distance has to be used, and this can generally produce satisfactory concrete, given an intelligent approach and sufficient care. For aggregate which includes natural sand, it may be useful, as one basis of comparison, to use the

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curves of the Road Research Note No. 4 on the Design of Concrete Mixes.[3.23] They have been prepared for aggregates of 19.05 and 38.1 mm ($\frac{3}{4}$ in. and $1\frac{1}{2}$ in.) maximum size, and are reproduced in Figs 3.14 and 3.15, respectively. Similar curves for aggregate with a 9.52 mm ($\frac{3}{8}$ in.) maximum size have been prepared by McIntosh and Erntroy,[3.24] and are shown in Fig. 3.16.

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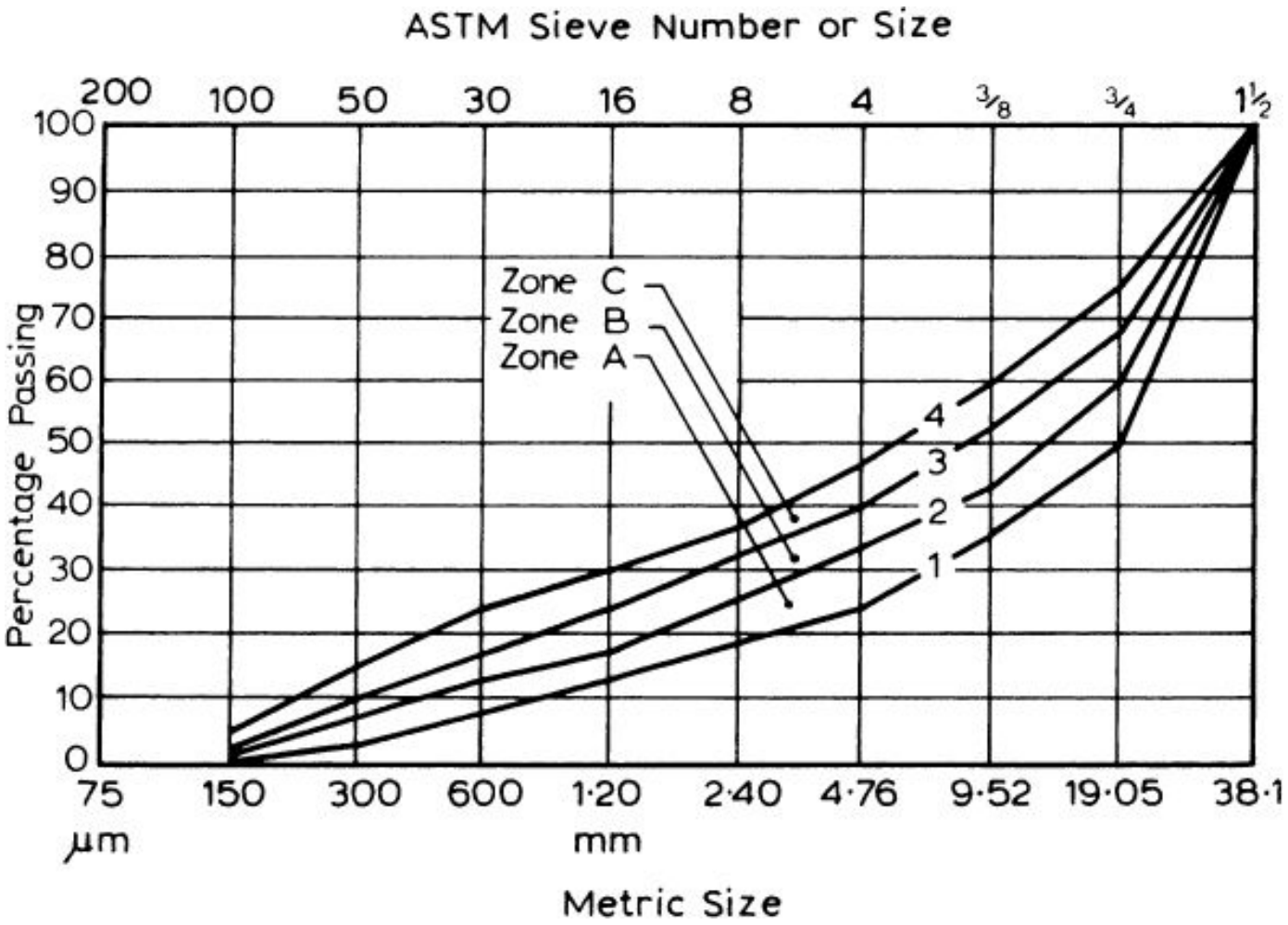


Fig. 3.15. Road Note No. 4 type grading curves for 38.1 mm ($1\frac{1}{2}$ in.) aggregate[3.23]

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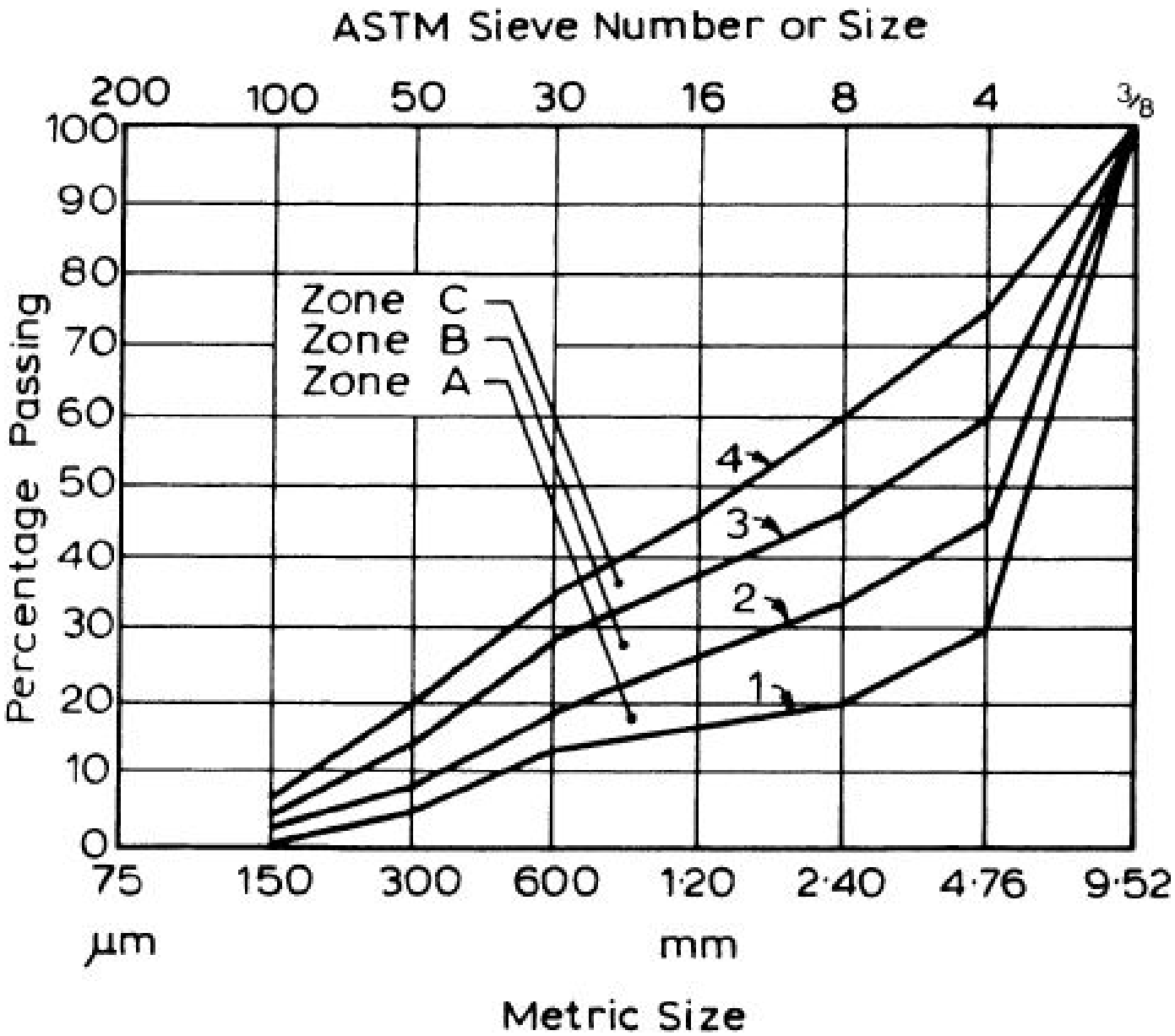


Fig. 3.16. Mclntosh and Erntroy's type grading curves for 9.52 mm ($\frac{3}{8}$ in.) aggregate[3.24]

Four curves are shown for each maximum size of aggregate but, due to the presence of over- and under-size aggregate and also because of variation *within* any fraction size, practical gradings

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are more likely to lie in the vicinity of these curves than to follow them exactly. It is therefore preferable to consider grading zones, and these are marked on all the diagrams.

Curve No. 1 represents the coarsest grading in each of the Figs 3.14 to 3.16. Such a grading is comparatively workable and can, therefore, be used for mixes with a low water/cement ratio or for rich mixes; it is, however, necessary to make sure that segregation does not take place. At the other extreme, curve No. 4 represents a fine grading: it will be cohesive but not very workable. In particular, an excess of material between 1.20 and 4.76 mm (No. 16 and $\frac{3}{16}$ in.) sieves will produce a harsh concrete, which may be suitable for compaction by vibration, but is difficult to place by hand. If the same workability is to be obtained using aggregates with grading curves Nos 1 and 4, the latter would require a considerably higher water content: this would mean a lower strength if both concretes are to have the same aggregate/cement ratio or, if the same strength is required,

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the concrete made with the finer aggregate would have to be considerably richer, i.e. each cubic metre would contain more cement than when the coarser grading is used.

The change between the extreme gradings is progressive. In the case of gradings lying partly in one zone, partly in another, there is, however, a danger of segregation when too many intermediate sizes are missing (cf. gap grading). If, on the other hand, there is an excess of middle-sized aggregate, the mix will be harsh and difficult to compact by hand and possibly even by vibration. For this reason, it is preferable to use aggregate with gradings similar to type, rather than totally dissimilar ones.

Figures 3.17 and 3.18 show the range of gradings used with 152.4 mm (6 in.) and 76.2 mm (3 in.) maximum aggregate size, respectively, as given by McIntosh.[3.25] The actual gradings, as usual, run parallel with the limit curves rather than crossing over from one to the other.

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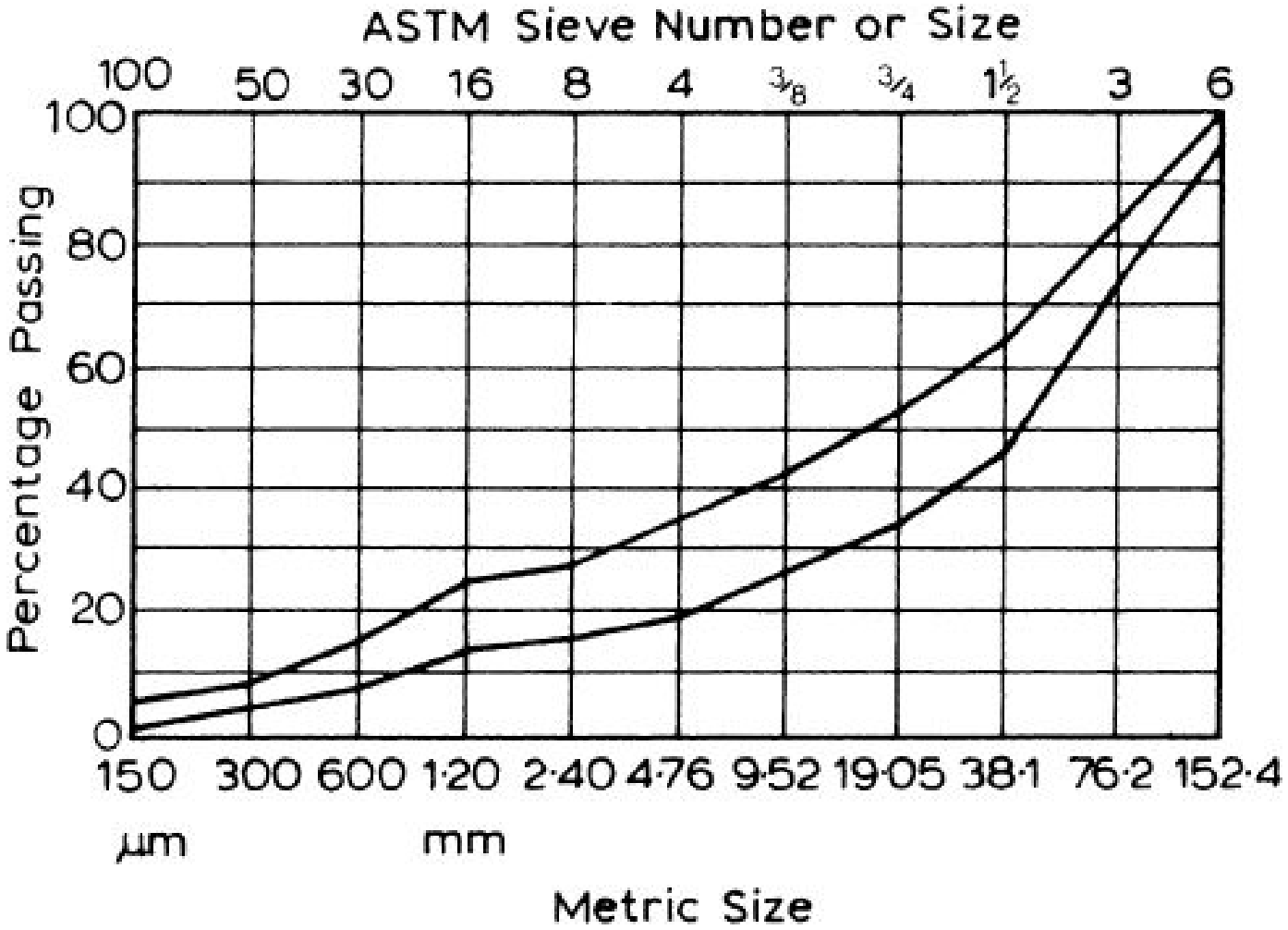


Fig. 3.17. Range of gradings used with 152.4 mm (6 in.) aggregate[3.25]

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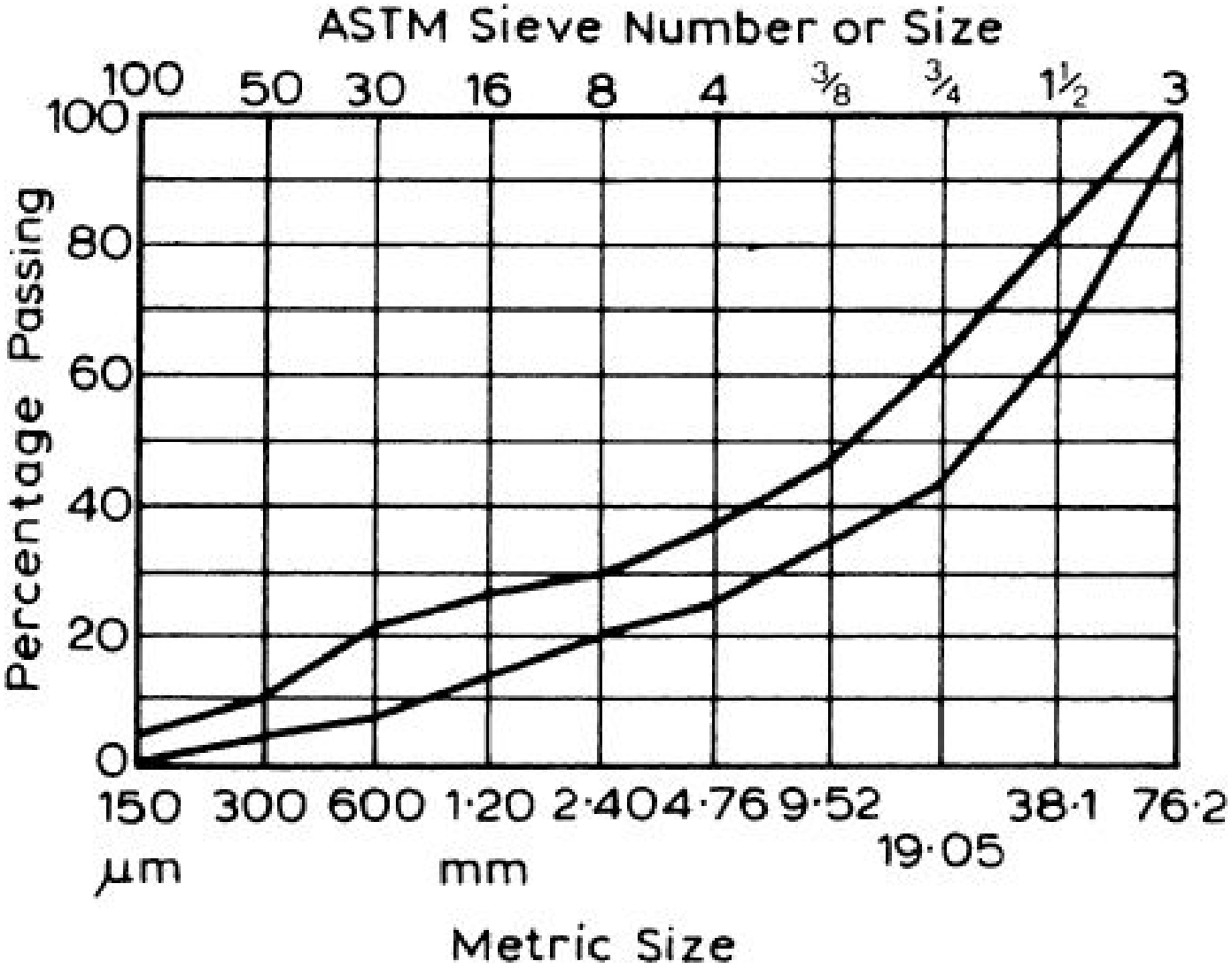


Fig. 3.18. Range of gradings used with 76.2 mm (3 in.) aggregate[3.25]

In practice, the use of separate fine and coarse aggregate means that a grading can be made up to conform exactly with a type grading at one intermediate point, generally the 5 mm ($\frac{3}{16}$ in.) size. Good agreement can usually also be obtained at the ends of the curve (150 μm (No. 100) sieve and

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the maximum size used). If coarse aggregate is delivered in single-size fractions, as is usually the case, agreement at additional points above 5 mm ($\frac{3}{16}$ in.) can be obtained, but for sizes below 5 mm ($\frac{3}{16}$ in.) blending of two or more fine aggregates is necessary.

Grading of fine and coarse aggregates

Given that, for any but unimportant work, fine and coarse aggregates are batched separately, the grading of each part of the aggregate should be known and controlled.

Over the years, there have been several approaches to specifying the grading requirements for fine aggregate. First, type grading curves were given as representing 'good' grading.[3.23] In the 1973 edition of BS 882, four grading zones were introduced. The division into zones was based primarily on the percentage passing the 600 μm (No. 30 ASTM) sieve. The main reason for this was that a large number of natural sands

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divide themselves at just that size, the gradings above and below being approximately uniform. Furthermore, the content of particles finer than the 600 μm (No. 30 ASTM) sieve has a considerable influence on the workability of the mix and provides a fairly reliable index of the overall specific surface of the sand.

Thus, the grading zones largely reflected the grading of natural sands available in the United Kingdom. Little of these sands is now available for concrete-making, and a much less restrictive approach to grading is reflected in the requirements of BS 882 : 1992. This does not mean that 'any grading will do'; rather, given that grading is but one feature of aggregate, a wide range of gradings may be acceptable but a trial-and-error approach is required.

Specifically, BS 882 : 1992 (withdrawn in 2004) requires any fine aggregate to satisfy the overall grading limits of Table 3.22 and also one of the three additional grading limits of the same table, but one in ten consecutive samples is al-

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lowed to fall outside the additional limits. The additional limits are, in effect, a coarse, a medium, and a fine grading. The current standard is BS EN 12620 : 2002.

Table 3.22. BS and ASTM Grading Requirements for Fine Aggregate

Sieve size		*Percentage by mass passing sieves*				
BS	*ASTM No.*	*BS 882 : 1992*				*ASTM C 33-08*
		Overall grading	*Coarse grading*	*Medium grading*	*Fine grading*	
10.0 mm	$\frac{3}{18}$ in.	100				100
5.0 mm	$\frac{3}{16}$ in.	89–100				95–100
2.36 mm	8	60–100	60–100	65–100	80–100	80–100
1.18 mm	16	30–100	30–90	45–100	70–100	50–85
600 μm	30	15–100	15–54	25–80	55–100	25–60
300 μm	50	5–70	5–40	5–48	5–70	5–30
150 μm	100	0–15*				0–10

*For crushed stone fine aggregate, the permissible limit is increased to 20 per cent except for heavy duty floors.

The requirements of BS 882 : 1992 may be inappropriate for some precast concrete and should not be applied in such cases.

For comparison, the requirements of ASTM C 33-08, are in part, included in Table 3.22. ASTM C 33-08 also requires the fine aggregate to have a

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fineness modulus of between 2.3 and 3.1. The requirements of the U.S. Bureau of Reclamation[3.74] are given in Table 3.23. It may be noted that, in the case of air-entrained concrete, lower quantities of the finest particles are acceptable, the entrained air acting effectively as very fine aggregate. ASTM C 33-08 also allows reduced percentages passing sieves 300 and 150 μm (Nos 50 and 100 ASTM) when the cement content is above 297 kg/m^3 (500 lb/yd^3) or if air entrainment is used with at least 237 kg of cement per cubic metre of concrete (400 lb/yd^3).

Table 3.23. U.S. Bureau of Reclamation Grading Requirements for Fine Aggregate[3.74]

Sieve size		
BS	*ASTM No.*	*Individual percentage by mass retained*
4.75 mm	4	0–5
2.36 mm	8	5–15 } or { 5–20
1.18 mm	16	10–25 } or { 10–20
600 μm	30	10–30
300 μm	50	15–35
150 μm	100	12–20
<150 μm	<100	3–7

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Fine aggregate satisfying any of the additional grading limits of BS 882 : 1992 can generally be used in concrete, although under some circumstances the suitability of a given fine aggregate may depend on the grading and shape of the coarse aggregate.

Crushed fine aggregate tends to have different grading from most natural sands. Specifically, there is less material between 600 and 300 μm (Nos 30 and 50) sieve sizes, coupled with more material larger than 1.18 mm (No. 16) sieve size and also more very fine material, smaller than 150 or 75 μm (No. 100 or No. 200) sieve size. Most specifications recognize the last feature and allow a higher content of very fine particles in crushed fine aggregate. It is important to ensure that this very fine material does not include clay or silt.

It has been shown[3.71] that increasing the content of particles smaller than 150 μm (No. 100) in crushed rock fine aggregate from 10 to 25 per cent results in only a small decrease in the com-

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pressive strength of concrete, typically by 10 per cent.

In considering the effects of a large amount of very fine material in the aggregate, it is useful to note that, when the material is well-rounded and smooth, workability is improved, and this is advantageous in terms of reduced water demand. Fine dune sands have such characteristics.[3.38]

In general terms, the ratio of coarse to fine aggregate should be higher the finer the grading of the fine aggregate. When crushed rock coarse aggregate is used, a slightly higher proportion of fine aggregate is required than with gravel aggregate in order to compensate for the lowering of workability by the sharp, angular shape of the crushed particles.

The requirements of BS 882 : 1992 for the grading of coarse aggregate are reproduced in Table 3.24: values are given both for graded aggregate and for nominal one-size fractions. The current British Standard BS EN 12620 : 2000 is augmented by PD 6682-1 : 2009. For comparis-

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on, some of the limits of ASTM C 33-08 are given in Table 3.25.

Table 3.24. Grading Requirements for Coarse Aggregate According to BS 882 : 1992

Sieve size		Percentage by mass passing BS sieves						
mm	in.	Nominal size of graded aggregate			Nominal size of single-sized aggregate			
		40 to 5 mm 1½ in. to ⅜ in.	20 to 5 mm ¾ in. to ⅜ in.	14 to 5 mm ½ in. to ⅜ in.	40 mm 1½ in.	20 mm ¾ in.	14 mm ½ in.	10 mm ⅜ in.
50.0	2	100	—	—	100	—	—	—
37.5	1½	90–100	100	—	85–100	100	—	—
20.0	¾	35–70	90–100	100	0–25	85–100	100	—
14.0	½	25–55	40–80	90–100	—	0–70	85–100	100
10.0	⅜	10–40	30–60	50–85	0–5	0–25	0–50	85–100
5.0	No. 4	0–5	0–10	0–10	—	0–5	0–10	0–25
2.36	No. 8	—	—	—	—	—	—	0–5

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Table 3.25. Grading Requirements for Coarse Aggregate According to ASTM C 33-08

Sieve size		Percentage by mass passing sieves				
mm	in.	Nominal size of graded aggregate			Nominal size of single-sized aggregate	
		37.5 to 4.75 mm $1\frac{1}{2}$ in. to $\frac{3}{16}$ in.	19.0 to 4.75 mm $\frac{3}{4}$ in. to $\frac{3}{16}$ in.	12.5 to 4.75 mm $\frac{1}{2}$ in. to $\frac{3}{16}$ in.	63 mm $2\frac{1}{2}$ in.	37.5 mm $1\frac{1}{2}$ in.
75	3	—	—	—	100	—
63.0	$2\frac{1}{2}$	—	—	—	90–100	—
50.0	2	100	—	—	35–70	100
38.1	$1\frac{1}{2}$	95–100	—	—	0–15	90–100
25.0	1	—	100	—	—	20–55
19.0	$\frac{3}{4}$	35–70	90–100	100	0–5	0–15
12.5	$\frac{1}{2}$	—	—	90–100	—	—
9.5	$\frac{3}{8}$	10–30	20–55	40–70	—	0–5
4.75	$\frac{3}{16}$	0–5	0–10	0–15	—	—
2.36	No. 8	—	0–5	0–5	—	—

The actual grading requirements depend, to some extent, on the shape and surface characteristics of the particles. For instance, sharp, angular particles with rough surfaces should have a slightly finer grading in order to reduce the possibility of interlocking and to compensate for the high friction between the particles. The actual grading of crushed aggregate is affected primarily by the type of crushing plant employed. A roll granulator usually produces fewer fines than oth-

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er types of crushers, but the grading depends also on the amount of material fed into the crusher.

The grading limits for all-in aggregate prescribed by BS 882 : 1992 (substantially retained in PD 6682-1 : 2009) are reproduced in Table 3.26. It should be remembered that this type of aggregate is not used except for small and unimportant jobs, mainly because it is difficult to avoid segregation in stockpiling.

Table 3.26. Grading Requirements for All-in Aggregate According to BS 882 : 1992

Sieve size		*Percentage by mass passing sieves*		
		40 mm (1½ in.) nominal size	*20 mm (¾ in.) nominal size*	*10 mm (⅜ in.) nominal size*
50.0 mm	2 in.	100	—	—
37.5 mm	1½ in.	95–100	100	—
20.0 mm	¾ in.	45–80	95–100	—
14.0 mm	½ in.	—	—	100
10.0 mm	⅜ in.	—	—	95–100
5.0 mm	3/16 in.	25–50	35–55	30–65
2.36 mm	No. 8	—	—	20–50
1.18 mm	No. 16	—	—	15–40
600 μm	No. 30	8–30	10–35	10–30
300 μm	No. 50	—	—	5–15
150 μm	No. 100	0–8*	0–8*	0–8*

*Increased to 10 per cent for crushed rock fine aggregate.

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Oversize and undersize

Strict adherence to size limits of aggregate is not possible: breakage during handling will produce some undersize material, and wear of screens in the quarry or at the crusher will result in oversize particles being present.

In the United States, it is usual to specify over- and undersize screen sizes as $\frac{7}{6}$ and $\frac{5}{6}$, respectively, of the nominal sieve size;[3.74] actual values are given in Table 3.27. The quantity of aggregate smaller than the undersize or larger than the oversize is generally severely limited.

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Table 3.27. Sizes of Over- and Under-Size Screens of U.S. Bureau of Reclamation[3.74]

Nominal size fraction		Test screen for:			
		Undersize		Oversize	
mm	in.	mm	in.	mm	in.
4.76–9.52	$\frac{3}{16}$–$\frac{3}{8}$	4.00	No. 5*	11.2	$\frac{7}{16}$
9.52–19.0	$\frac{3}{8}$–$\frac{3}{4}$	8.0	$\frac{5}{16}$	22.4	$\frac{7}{8}$
19.0–38.1	$\frac{3}{4}$–$1\frac{1}{2}$	16.0	$\frac{5}{8}$	45	$1\frac{3}{4}$
38.1–76.2	$1\frac{1}{2}$–3	31.5	$1\frac{1}{4}$	90	$3\frac{1}{2}$
76.2–152.4	3–6	63	$2\frac{1}{2}$	178	7

*ASTM size.

The grading requirements of BS 882 : 1992 allow some under- and oversize for coarse aggregate. The values, given in Table 3.24, show that between 5 and 10 per cent oversize is permitted. However, no aggregate must be retained on a sieve one size larger (in the standard series) than the nominal maximum size. In the case of single-size aggregate, some undersize is also allowed, and the amount passing the sieve next smaller than the nominal size is also prescribed. It is important that this fine fraction of coarse aggregate be not neglected in the calculation of the actual grading.

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For fine aggregate, BS 882 : 1992 allows 11 per cent oversize (see Table 3.22). General grading requirements for coarse and fine aggregate are given in BS EN 12620 : 2002 in terms of upper sieve size D and lower sieve size d with $D/d \geq 1.4$.

Gap-graded aggregate

As mentioned earlier, aggregate particles of a given size pack so as to form voids that can be penetrated only if the next smaller size of particles is sufficiently small, that is, there is no particle interference. This means that there must be a minimum difference between the sizes of any two adjacent particle fractions. In other words, sizes differing but little cannot be used side by side, and this has led to advocacy of gap-graded aggregate.

Gap grading can then be defined as a grading in which one or more intermediate size fractions are omitted. The term *continuously graded* is used to describe conventional grading when it is necessary to distinguish it from gap grading. On

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a grading curve, gap grading is represented by a horizontal line over the range of sizes omitted. For instance, the top grading curve of Fig. 3.19 shows that no particles of size between 10.0 and 2.36 mm ($\frac{3}{8}$ in. and No. 8 ASTM) sieve are present. In some cases, a gap between 10.0 and 1.18 mm ($\frac{3}{8}$ in. and No. 16 ASTM) sieves is considered suitable. Omission of these sizes would reduce the number of stockpiles of aggregate required and lead to economy. In the case of aggregate of 20.0 mm ($\frac{3}{4}$ in.) maximum size, there would be two stockpiles only: 20.0 to 10.0 mm ($\frac{3}{4}$ to $\frac{3}{8}$ in.), and fine aggregate screened through a 1.18 mm (No. 16 ASTM) screen. The particles smaller than 1.18 mm (No. 16 ASTM) sieve size could easily enter the voids in the coarse aggregate so that the workability of the mix would be higher than that of a continuously graded mix with the same fine aggregate content.

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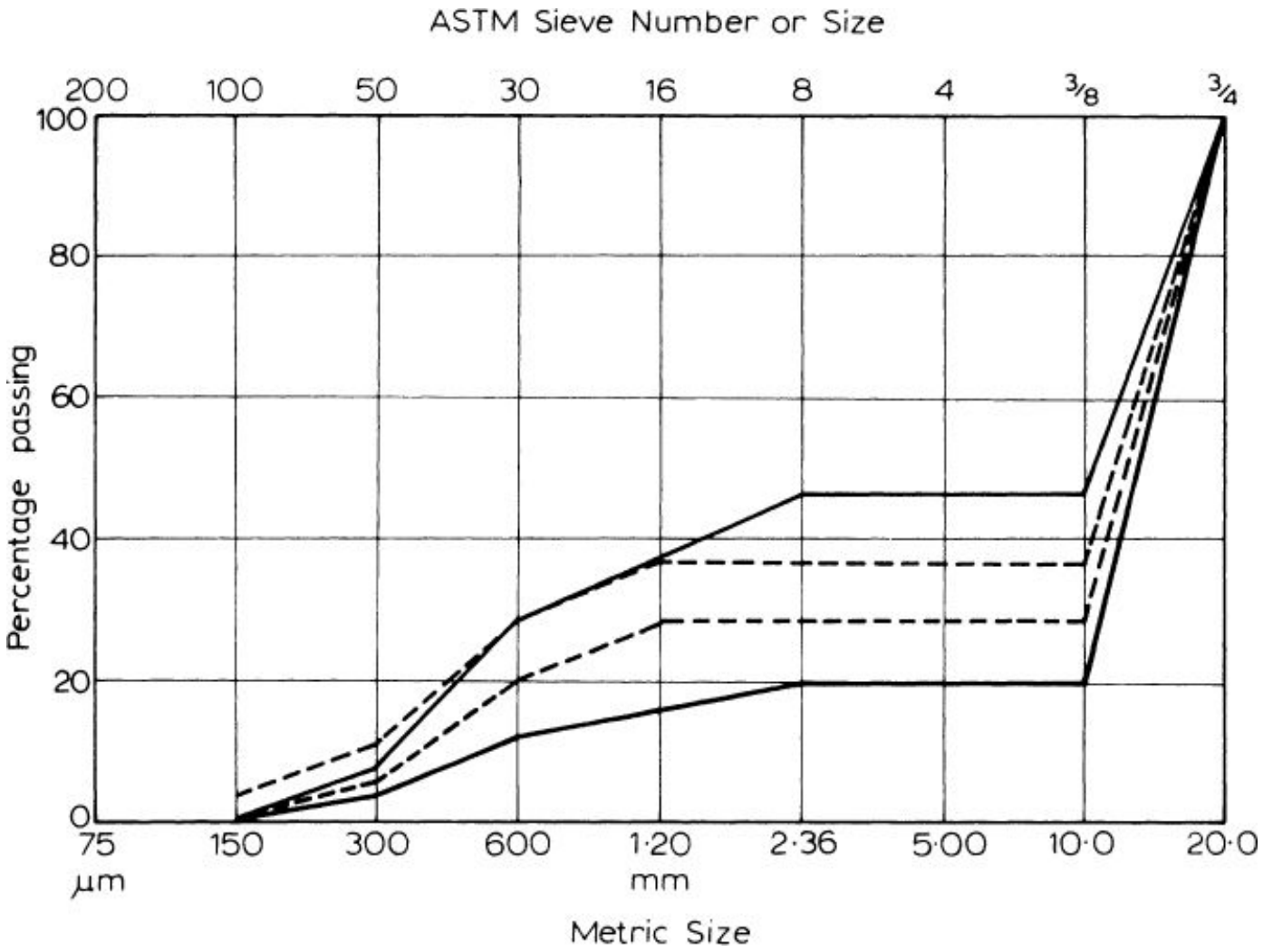


Fig. 3.19. Typical gap gradings

Tests by Shacklock[3.26] have shown that, for a given aggregate/cement ratio and water/cement ratio, a higher workability is obtained with a lower fine aggregate content in the case of gap-graded aggregate than when continuously graded aggregate is used. However, in the more workable range of mixes, gap-graded aggregate showed a greater proneness to segregation. For

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this reason, gap grading is recommended mainly for mixes of relatively low workability: such mixes respond well to vibration. Good control and, above all, care in handling, so as to avoid segregation, are essential.

It may be observed that, even when some 'ordinary' aggregates are used, gap grading exists; for instance, the use of very fine sand, as found in many countries, means that there is a deficiency of particles between the 5.00 mm and 2.36 or 1.18 mm ($\frac{3}{16}$ in. and No. 8 or No. 16 ASTM) sieve sizes. Thus, whenever we use such a sand without blending it with a coarser sand, we are, in fact, using a gap-graded aggregate.

Gap-graded aggregate concrete is difficult to pump because of the danger of segregation, and is not suited to slip-form paving. Otherwise, gap-graded aggregate can be used in any concrete, but there are two cases of interest: preplaced aggregate concrete (see p. 228) and exposed aggregate concrete; in the latter, a pleasing finish is obtained because a large quantity of only one size

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of coarse aggregate becomes exposed after treatment.

From time to time, various claims of superior properties have been made for concrete made with gap-graded aggregate, but these do not seem to have been substantiated. Strength, both compressive and tensile, does not appear to be affected. Likewise, Fig. 3.20, showing McIntosh's[3.27] results, confirms that, using given materials with a fixed aggregate/cement ratio (but adjusting the fine aggregate content), approximately the same workability and strength are obtained with gap and continuous gradings; Brodda and Weber[3.72] reported a slight negative influence of gap grading on strength.

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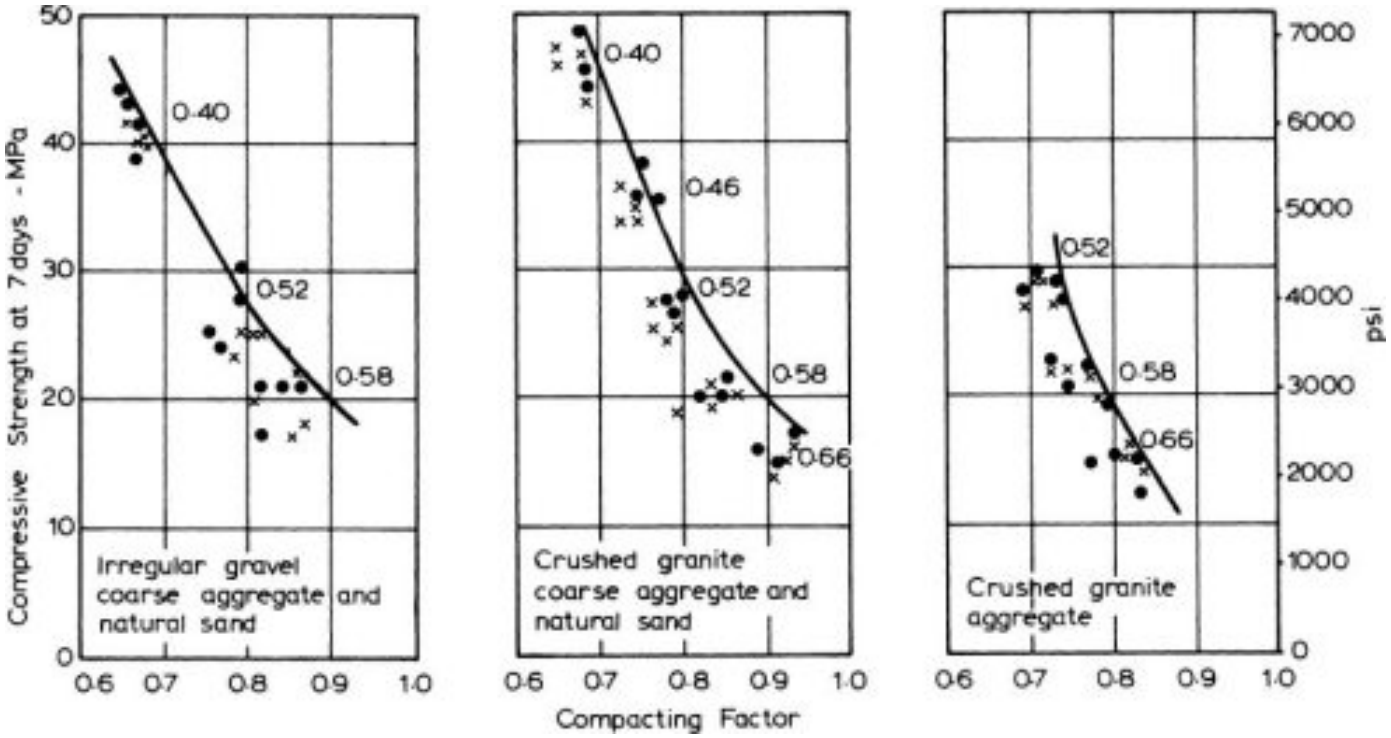


Fig. 3.20. Workability and strength of 1 : 6 concretes made with gap- and continuously graded aggregates.[3.27] Cross denotes gap-graded and circle continuously graded mixes. Each group of points represents mixes with the water/cement ratio indicated but with different sand contents

Similarly, there is no difference in shrinkage of the concretes made with aggregate of either type of grading,[3.26] although it might be expected that a framework of coarse particles almost touching one another would result in a lower total

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change in dimensions on drying. The resistance of concrete to freezing and thawing is lower when gap-graded aggregate is used.[3.26]

It seems, therefore, that the rather extravagant claims made by advocates of gap grading are not borne out. The explanation lies probably in the fact that, while gap grading makes it possible for maximum packing of particles to occur, there is no way of ensuring that it *will* occur. Both gap-graded and continuously graded aggregate can be used to make good concrete, but in each case the right percentage of fine aggregate has to be chosen. Thus, once again, it can be seen that we should not aim at any ideal grading but find the best combination of the available aggregates.

Maximum aggregate size

It has been mentioned before that the larger the aggregate particle the smaller the surface area to be wetted per unit mass. Thus, extending the grading of aggregate to a larger maximum size lowers the water requirement of the mix, so that,

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for a specified workability and cement content, the water/cement ratio can be lowered with a consequent increase in strength.

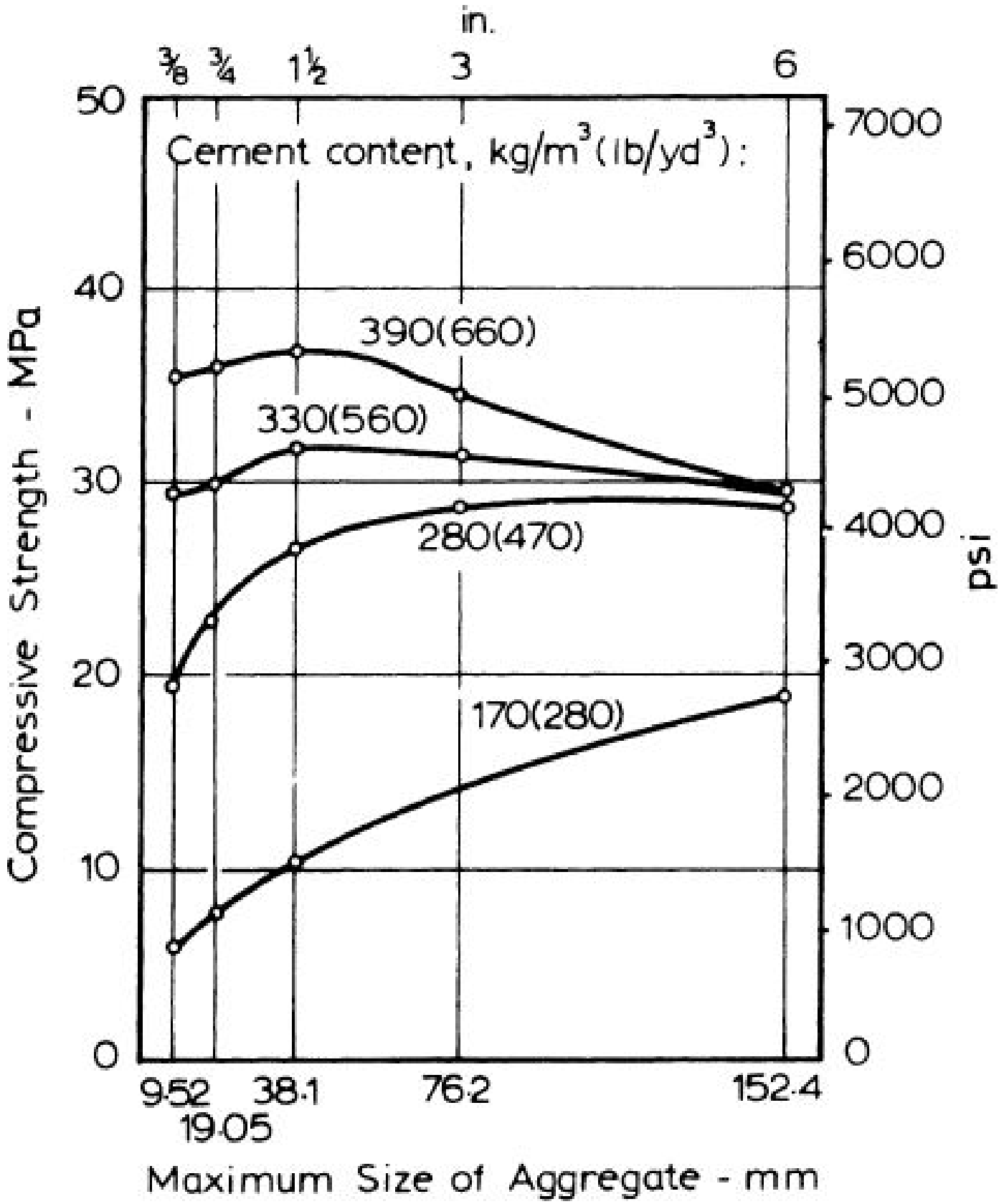
This behaviour has been verified by tests with aggregates up to 38.1 mm ($1\frac{1}{2}$ in.) maximum size,[3.28] and is usually assumed to extend to larger sizes as well. Experimental results show, however, that above the 38.1 mm ($1\frac{1}{2}$ in.) maximum size the gain in strength due to the reduced water requirement is offset by the detrimental effects of lower bond area (so that volume changes in the paste cause larger stresses at interfaces) and of discontinuities introduced by the very large particles, particularly in rich mixes. Concrete becomes grossly heterogeneous and the resultant lowering of strength may possibly be similar to that caused by a rise in the crystal size and coarseness of texture in rocks.

This adverse effect of increase in the size of the largest aggregate particles in the mix exists, in fact, throughout the range of sizes, but below

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38.1 mm ($1\frac{1}{2}$ in.) the effect of size on the decrease in the water requirement is dominant. For larger sizes, the balance of the two effects depends on the richness of the mix,[3.42,3.51] as shown in Fig. 3.21. Nichols[3.89] confirmed that, for any given strength of concrete, that is, for a given water/cement ratio, there is an optimum maximum size of aggregate.

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Fig. 3.21. Influence of maximum size of aggregate on the 28-day compressive strength of concretes of different richness[3.51]

Thus, the best maximum size of aggregate from the standpoint of strength is a function of the richness of the mix. Specifically, in lean concrete (165 kg of cement per cubic metre (280 lb/yd^3)), the use of 150 mm (or 6 in.) aggregate is advantageous. However, in structural concrete of usual proportions, from the point of view of strength, there is no advantage in using aggregate with a maximum size greater than about 25 or 40 mm (1 or $1\frac{1}{2}$ in.). Moreover, the use of larger aggregate would require the handling of a separate stockpile and might increase the risk of segregation, especially when the maximum size is 150 mm (6 in.). However, a practical decision would also be influenced by the availability and cost of different size fractions. The choice of the maximum size of aggregate in high performance concrete is discussed on p. 678.

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There are, of course, structural limitations too: the maximum size of aggregate should be no more than $\frac{1}{5}$ to $\frac{1}{4}$ of the thickness of the concrete section and is related also to the spacing of reinforcement. The governing values are prescribed in codes of practice.

Use of 'plums'

The original idea of the use of aggregate as an inert filler can be extended to the inclusion of large stones in a normal concrete: thus the apparent yield of concrete for a given amount of cement is increased. The resulting concrete is sometimes called cyclopean concrete.

These large stones are called 'plums' and, used in a large concrete mass, they can be as big as a 300 mm (1 ft) cube but should not be greater than one-third of the least dimension to be concreted. The volume of plums should not exceed 20 to 30 per cent of the total volume of the finished concrete, and they have to be well dispersed throughout the mass. This is achieved by

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placing a layer of normal concrete, then spreading the plums, followed by another layer of concrete, and so on. Each layer should be of such thickness as to ensure at least 100 mm (4 in.) of concrete around each plum. Care must be taken to ensure that no air is trapped underneath the stones and that the concrete does not work away from their underside. The plums must have no adhering coating. Otherwise, discontinuities between the plums and the concrete may induce cracking and adversely affect permeability.

The placing of plums requires a large amount of labour and also breaks the continuity of concreting. It is, therefore, not surprising that, with the current high ratio of the cost of labour to the cost of cement, the use of plums is not economical except under special circumstances.

Handling of aggregate

Handling and stockpiling of coarse aggregate can easily lead to segregation. This is particularly so when discharging and tipping permits the aggreg-

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ate to roll down a slope. A natural case of such segregation is a scree (talus): the size of particles is uniformly graded from largest at the bottom to smallest at the top.

A description of the precautions necessary in handling operations is outside the scope of this book, but one vital recommendation should be mentioned: coarse aggregate should be split into size fractions approximately 5 to 10, 10 to 20, 20 to 40 mm (or $\frac{3}{16}$ to $\frac{3}{8}$, $\frac{3}{8}$ to $\frac{3}{4}$, $\frac{3}{4}$ to $1\frac{1}{2}$ in.), etc. These fractions should be handled and stockpiled separately and remixed only when being fed into the concrete mixer in the desired proportions. Thus, segregation can occur only within the narrow size range of each fraction, and even this can be reduced by careful handling procedures.

Care is necessary to avoid breakage of the aggregate: particles greater than 40 mm (or $1\frac{1}{2}$ in.) should be lowered into bins by means of rock ladders and not dropped from a height.

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On large and important jobs, the results of segregation and breakage in handling (i.e. excess of undersize particles) are eliminated by 'finish re-screening' immediately prior to feeding into the batching bins over the mixer. The proportions of different sizes are thus controlled much more effectively but the complexity and cost of the operations are correspondingly increased. This is, however, repaid by easier placing of uniformly workable concrete and by a possible saving in cement due to the uniformity of the concrete.

Improper handling of aggregates can result in contamination by other aggregate or by deleterious material: it was observed on one occasion that aggregate was being transported in sacks which had previously contained sugar (see p. 253).

Special aggregates

This chapter has been concerned solely with natural aggregate of normal weight; lightweight aggregates are discussed in Chapter 13. There exist,

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however, also other aggregates of normal weight, or nearly so, which are artificial in origin. The reasons for their advent on the concrete scene are as follows.

Environmental considerations are increasingly affecting the supply of aggregate. There are strong objections to opening of pits as well as to quarrying. At the same time, there are problems with the disposal of construction demolition waste and with dumping of domestic waste. Both these types of waste can be processed into aggregate for use in concrete, and this is increasingly being done.

Recycled concrete aggregate

The aggregate obtained by comminution of demolished concrete is known as recycled concrete aggregate (RCA). Until now, RCA has been used predominantly in highway pavements and in non-structural concrete. There is, however, little doubt that structural use of RCA will increase, but care is required. According to ASTM C 294-05, RCA

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is classified as an artificial aggregate. The following specific points should be considered when using old concrete as aggregate for making new concrete. Because RCA consists in part of old mortar, the unit weight (density) of concrete made with RCA is lower than that of concrete made with conventional aggregate. For the same reason, concrete made with RCA has a higher porosity and absorption. The higher absorption of RCA can be exploited if it is saturated before mixing: the absorbed water provides internal curing. In particular, this is so for RCA containing a large amount of brick.

The potential compressive strength of the new concrete is largely controlled by the strength of the old concrete, provided the fine aggregate is crushed rock or natural sand of suitable quality. A substantial reduction in compressive strength may result if conventional fine aggregate is replaced partly or wholly by fine aggregate from the old concrete; anything smaller than 2 mm should be discarded. The use of RCA decreases

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workability of fresh concrete at a given water content, increases water requirement at a given consistency, increases drying shrinkage at a given water content, and reduces the modulus of elasticity at a given water/cement ratio. Those effects are greatest when the old concrete is used as both coarse and fine aggregate. Freezing and thawing resistance of the new concrete depends on the air-void system and strength of the old concrete as well as the corresponding properties of the new concrete.

Chemical, air-entraining and mineral admixtures present in the old concrete will not significantly modify the properties of the new concrete. However, high concentrations of chloride ions in the old concrete may contribute to accelerated corrosion of steel embedments in the new concrete. Prospective sources of old concrete may be unsuitable if they were subjected to aggressive chemical attack or leaching, damage by fire or service at high temperature, etc.

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The significance of contaminants in the old concrete, such as noxious, toxic or radioactive substances, should be analysed in relation to the anticipated service of the new concrete. While the presence of bituminous materials may impair air-entrainment, appreciable concentrations of organic materials could produce excessive air-entrainment. Metallic inclusions may cause rust staining or surface blistering, and glass fragments could lead to alkali-aggregate reaction.

A method for determining the composition of RCA is given in BS 8500-2 : 2002.

The necessary treatment of waste is not simple and the use of aggregate made from waste requires specialist knowledge as none of the materials has become standardized. In particular, building rubble may contain deleterious amounts of brick, glass, gypsum or chlorides.[3.31,3.35,3.36] The processing of demolition waste so as to convert it into satisfactory aggregate free from contaminants is still being developed. A reduction in aggregate interlock in concrete made with recycled

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aggregate has been confirmed by González *et al.*[3.90] The influence of aggregate type on interlock is discussed by Regan.[3.91]

As far as the use of domestic refuse is concerned, the incinerator ash, after removal of ferrous and non-ferrous metals, can be ground to a fine powder, blended with clay, pelletized and fired in a kiln to produce artificial aggregate. The material is capable of producing concrete with compressive strengths as high as 50 MPa (7000 psi) at 28 days. There will, obviously, be problems with variations in the composition of the raw ash, and the long-term durability characteristics of the material have yet to be determined, although results to date look promising.

These topics are outside the scope of this book, but readers should be aware of the new and growing possibilities of using processed waste as aggregate.

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Chapter 4. Fresh concrete

Although fresh concrete is only of transient interest, we should note that the strength of concrete of given mix proportions is very seriously affected by the degree of its compaction. It is vital, therefore, that the consistency of the mix be such that the concrete can be transported, placed, compacted, and finished sufficiently easily and without segregation. This chapter is therefore devoted to the properties of fresh concrete which will contribute to such an objective.

Before considering fresh concrete, we should observe that the first three chapters discussed only two of the three essential ingredients of concrete: cement and aggregate. The third essential ingredient is water, and this will be considered below.

It may be appropriate to add, at this stage, that many, if not most, concrete mixes contain also admixtures: these are the topic of Chapter 5.

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Quality of mixing water

The vital influence of the quantity of water in the mix on the strength of the resulting concrete will be considered in Chapter 6. Apart from that, traditionally, those studying concrete have shown little interest in water in the mix. Admittedly, water is necessary to make the mix adequately workable and it is, of course, necessary, to hydrate the cement or, as was later established, only some of the cement. Consequently, relatively little effort was devoted to the study of the quality of water.

However, water is not just a liquid used to make concrete: it is involved in the whole life of concrete, for good or for evil. Most actions on concrete in service, other than loading, involve water, either pure or carrying salts or solids. The important influences of water, in addition to those on workability and strength, are those on: setting, hydration, bleeding, drying shrinkage, creep, ingress of salts, explosive failure of concrete with a very low water-cement ratio, autogenous healing, staining of the surface, chemical attack of con-

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crete, corrosion of reinforcement, freezing and thawing, carbonation, alkali-silica reaction, thermal properties, electrical resistivity, cavitation and erosion, and quality of drinking water passed through concrete pipes or mortar-lined pipes. This is a pretty exhaustive list.

As some of the influences are for good, and others for bad, one could say that water and concrete are in a love-hate relationship; this indeed is the title of a chapter in my book *Neville on Concrete: an examination of issues in concrete practice*.[4.122] Another chapter in that book is titled "Water: Cinderella Ingredient of Concrete".

For these reasons, the suitability of water for mixing and curing purposes should be considered. Clear distinction must be made between the quality of mixing water and the attack on hardened concrete by aggressive waters. Indeed, some waters which adversely affect hardened concrete may be harmless or even beneficial when used in mixing.[4.15] The quality of curing water is considered on p. 324.

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Mixing water should not contain undesirable organic substances or inorganic constituents in excessive proportions. However, quantitative limits of harmful constituents are not known reliably and also, because unnecessary restrictions could be economically damaging. Some limits are specified in BS EN 1008 : 2002.

In many project specifications, the quality of water is covered by a clause saying that water should be fit for drinking. Such water very rarely contains dissolved inorganic solids in excess of 2000 parts per million (ppm), and as a rule less than 1000 ppm. For a water/cement ratio of 0.5, the latter content corresponds to a quantity of solids representing 0.05 per cent of the mass of cement, and any effect of the common solids would be small.

While the use of potable water as mixing water is generally satisfactory, there are some exceptions; for instance, in some arid areas, local drinking water is saline and may contain an excessive amount of chlorides. Also, some natural mineral

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waters contain undesirable amounts of alkali carbonates and bicarbonates which could contribute to the alkali–silica reaction.

Conversely, some waters not fit for drinking may often be used satisfactorily in making concrete. As a rule, water with pH of 6.0 to 8.0,[4.33] or possibly even 9.0, which does not taste brackish is suitable for use, but dark colour or bad smell do not necessarily mean that deleterious substances are present.[4.16] A simple way of determining the suitability of such water is to compare the setting time of cement and the strength of mortar cubes using the water in question with the corresponding results obtained using known 'good' water or distilled water; there is no appreciable difference between the behaviour of distilled and ordinary drinking water. A tolerance of about 10 per cent is usually permitted to allow for chance variations in strength;[4.15] BS EN 1008-2002 also specifies 10 per cent. Such tests are recommended when water for which no service record is available contains dissolved solids in excess of 2000

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ppm or, in the case of alkali carbonate or bicarbonate, in excess of 1000 ppm. When unusual solids are present a test is also advisable. Limits on chlorides, sulfates and alkali are given in BS EN 1008 : 2002 and ASTM C 1602-06.

Because it is undesirable to introduce large quantities of clay and silt into the concrete, mixing water with a high content of suspended solids should be allowed to stand in a settling basin before use; a turbidity limit of 2000 ppm has been suggested.[4.7] However, water used to wash out truck mixers is satisfactory as mixing water, provided of course that it was satisfactory to begin with. ASTM C 94-94a and BS EN 1008-2002 give the requirements for the use of wash water. Clearly, cements and admixtures different from those originally used should not be involved. The use of wash water is an important topic, but is outside the scope of this book.

Natural waters that are slightly acid are harmless, but water containing humic or other organic acids may adversely affect the hardening of con-

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crete; such water, as well as highly alkaline water, should be tested. The effects of different ions vary, as shown by Steinour.[4.15]

It may be interesting to note that the presence of algae in mixing water results in air entrainment with a consequent loss of strength.[4.13] According to the appendix to BS 3148 : 1980, green or brown slime-forming algae should be regarded with suspicion, and water containing them should be tested.

Brackish water contains chlorides and sulfates. When chloride does not exceed 500 ppm, or SO_3 does not exceed 1000 ppm, the water is harmless, but water with even higher salt contents has been used satisfactorily.[4.35] The appendix to BS 3148 : 1980 recommends limits on chloride and on SO_3 as above, and also recommends that alkali carbonates and bicarbonates should not exceed 1000 ppm. Somewhat less severe limitations are recommended in American literature.[4.33]

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Sea water has a total salinity of about 3.5 per cent (78 per cent of the dissolved solids being NaCl and 15 per cent $MgCl_2$ and $MgSO_4$) (cf. p. 517), and produces a slightly higher early strength but a lower long-term strength; the loss of strength is usually no more than 15 per cent[4.25] and can therefore often be tolerated. Some tests suggest that sea water slightly accelerates the setting time of cement, others[4.27] show a substantial reduction in the initial setting time but not necessarily in the final set. Generally, the effects on setting are unimportant if water is acceptable from strength considerations. BS EN 1008-2002 specifies a tolerance of 25 minutes in the initial setting time and a maximum final setting time of 12 hours.

Water containing large quantities of chlorides (e.g. sea water) tends to cause persistent dampness and surface efflorescence. Such water should, therefore, not be used where appearance of unreinforced concrete is of importance, or where a plaster finish is to be applied.[4.9] Much

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more importantly, the presence of chlorides in concrete containing embedded steel can lead to its corrosion; the limits on the total chloride ion content in concrete are considered on p. 566.

In this connection, but also with respect to all impurities in water, it is important to remember that water discharged into the mixer is not the only source of mix water: aggregate usually contains surface moisture (see p. 132). This water can represent a substantial proportion of the total mixing water. It is, therefore, important that the water brought in by the aggregate is also free from harmful material.

Tests on mixes with a range of waters suitable for use in concrete showed no effect on the structure of the hydrated cement paste.[4.103]

The preceding discussion was concerned with structural concrete, usually reinforced or prestressed. Under particular circumstances, for instance in the construction of unreinforced concrete bulkheads in a mine, highly contaminated

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water can be used. Al-Manaseer *et al.*[4.102] showed that water containing very high percentages of salts of sodium, potassium, calcium and magnesium used in making concrete containing Portland cement blended with fly ash did not adversely affect the strength of concrete. However, no information on long-term behaviour is available. Biologically treated domestic waste water has also been investigated for use as mixing water,[4.40] but much more information about the variability of such water, health hazards and long-term behaviour is required.

On page 183 reference was made to a possible effect of cement in the interior surface of a pipe on water destined for human consumption. As long as water moves through a concrete pipe (or a mortar-lined conduit) at speed, no significant chemical reaction with cement occurs. However, when water is near-stagnant, as during the night in domestic water conduits, leaching of cement may occur. This may raise the pH of the water and increase the content of $CaCO_3$, referred to

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as carbonate alkalinity or water hardness. The increase in $CaCO_3$ is induced by carbon dioxide dissolved in the water and a reaction with calcium hydroxide by water may also increase the content of aluminium, calcium, sodium, and potassium and of corrosion inhibitors in the mix.[4.122]

Curing water should generally satisfy the requirements for mixing water, but it should be free from substances that attack hardened concrete. Also, flowing pure water dissolves $Ca(OH)_2$ and causes surface erosion. Curing very young concrete with seawater may lead to an attack on reinforcement.

Density of fresh concrete

Density, also called unit mass or unit weight in air, can be determined experimentally by using ASTM standard C 138-09 or BS EN 12350-6 : 2009. Theoretically, density is the sum of masses of all the ingredients of a batch of concrete divided by the volume filled by the concrete.

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Alternatively, knowing the density of fresh concrete, the yield per batch can be determined as the mass of all the ingredients in a batch divided by the density.

Definition of workability

A concrete which can be readily compacted is said to be workable, but to say merely that workability determines the ease of placement and the resistance to segregation is too loose a description of this vital property of concrete. Furthermore, the desired workability in any particular case would depend on the means of compaction available; likewise, a workability suitable for mass concrete is not necessarily sufficient for thin, inaccessible, or heavily reinforced sections. For these reasons, workability should be defined as a physical property of concrete alone without reference to the circumstances of a particular type of construction.

To obtain such a definition it is necessary to consider what happens when concrete is being

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compacted. Whether compaction is achieved by ramming or by vibration, the process consists essentially of the elimination of entrapped air from the concrete until it has achieved as close a configuration as is possible for a given mix. Thus, the work done is used to overcome the friction between the individual particles in the concrete and also between the concrete and the surface of the mould or of the reinforcement. These two can be called internal friction and surface friction, respectively. In addition, some of the work done is used in vibrating the mould or in shock and, indeed, in vibrating those parts of the concrete which have already been fully consolidated. Thus the work done consists of a 'wasted' part and 'useful' work, the latter, as mentioned before, comprising work done to overcome the internal friction and the surface friction. Because only the internal friction is an intrinsic property of the mix, workability can be best defined as the amount of useful internal work necessary to produce full compaction. This definition was deve-

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loped by Glanville *et al.*[4.1] who exhaustively examined the field of compaction and workability. The ASTM C 125-09a definition of workability is somewhat more qualitative: "property determining the effort required to manipulate a freshly mixed quantity of concrete with minimum loss of homogeneity". The ACI definition of workability, given in ACI 116R-90,[4.46] is: "that property of freshly mixed concrete or mortar which determines the ease and homogeneity with which it can be mixed, placed, consolidated, and finished".

Another term used to describe the state of fresh concrete is *consistency*. In ordinary English usage, this word refers to the firmness of form of a substance or to the ease with which it will flow. In the case of concrete, consistency is sometimes taken to mean the degree of wetness; within limits, wet concretes are more workable than dry concretes, but concretes of the same consistency may vary in workability. The ACI definition of consistency is: "the relative mobility or ability of

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freshly mixed concrete or mortar to flow";[4.46] this is measured by slump.

Technical literature abounds with variations of the definitions of workability and consistency but they are all qualitative in nature and more reflections of a personal viewpoint rather than of scientific precision. The same applies to the plethora of terms such as: flowability, mobility, and pumpability. There is also a term 'stability' which refers to the cohesion of the mix, that is, its resistance to segregation. These terms do have specific meaning but only under a set of given circumstances; they can rarely be used as an objective and quantifiable description of a concrete mix.

A good review of the attempts to define the various terms is presented by Bartos,[4.56] among others.

The need for sufficient workability

Workability has so far been discussed merely as a property of fresh concrete: it is, however, also

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a vital property as far as the finished product is concerned because concrete must have a workability such that compaction to maximum density is possible with a reasonable amount of work or with the amount that we are prepared to put in under given conditions.

The need for compaction becomes apparent from a study of the relation between the degree of compaction and the resulting strength. It is convenient to express the former as a density ratio, i.e. a ratio of the actual density of the given concrete to the density of the same mix when fully compacted. Likewise, the ratio of the strength of the concrete is actually (partially) compacted to the strength of the same mix when fully compacted can be called the strength ratio. Then the relation between the strength ratio and the density ratio is of the form shown in Fig. 4.1. The presence of voids in concrete greatly reduces its strength: 5 per cent of voids can lower strength by as much as 30 per cent, and even 2 per cent voids can result in a drop of strength of more than 10

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per cent.[4.1] This, of course, is in agreement with Féret's expression relating strength to the sum of the volumes of water and air in the hardened cement paste (see p. 271).

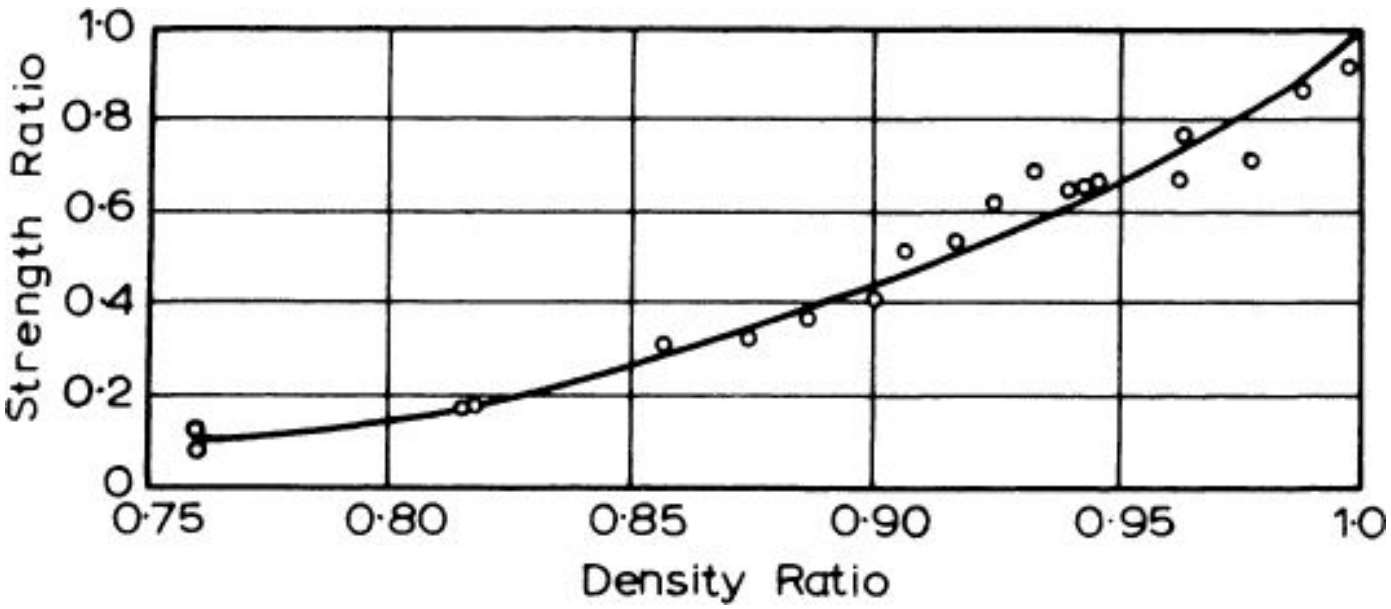


Fig. 4.1. Relation between strength ratio and density ratio.[4.1] (Crown copyright)

Voids in concrete are in fact either bubbles of entrapped air or spaces left after excess water has been removed. The volume of the latter depends primarily on the water/cement ratio of the mix; to a lesser extent, there may be spaces arising from water trapped underneath large particles of aggregate or underneath reinforcement. The air

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bubbles, which represent 'accidental' air, i.e. voids within an originally loose granular material, are governed by the grading of the finest particles in the mix and are more easily expelled from a wetter mix than from a dry one. It follows, therefore, that for any given method of compaction there may be an optimum water content of the mix at which the sum of the volumes of air bubbles and water space will be a minimum. At this optimum water content, the highest density ratio of the concrete would be obtained. It can be seen, however, that the optimum water content may vary for different methods of compaction.

Factors affecting workability

The main factor is the water *content* of the mix, expressed in kilograms (or litres) of water per cubic metre of concrete: it is convenient, though approximate, to assume that, for a given type and grading of aggregate and workability of concrete, the water content is independent of the aggregate/cement ratio or of the cement content of the

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mix. On the basis of this assumption, the mix proportions of concretes of different richness can be estimated, and Table 4.1 gives typical values of water content for different slumps and maximum sizes of aggregate. These values are applicable to non-air-entrained concrete only. When air is entrained, the water content can be reduced in accordance with the data of Fig. 4.2.[4.2] This is indicative only, because the effect of entrained air on workability depends on the mix proportions, as described in detail on p. 562.

Table 4.1. Approximate Water Content for Different Slumps and Maximum Sizes of Aggregate (partially based on the approach of the National Aggregates Association in the United States)

Maximum size of aggregate		Water content of concrete											
		25–50 mm (1–2 in.) slump				75–100 mm (3–4 in.) slump				150–175 mm (6–7 in.) slump			
		Rounded aggregate		Angular aggregate		Rounded aggregate		Angular aggregate		Rounded aggregate		Angular aggregate	
mm	in.	kg/m³	lb/yd³	kg/m³	lb/yd³	kg/m³	lb/yd³	kg/m³	lb/yd³	kg/m³	lb/yd³	kg/m³	lb/yd³
9.5	⅜	185	310	210	350	200	340	225	380	220	375	250	420
12.7	½	175	295	200	335	195	325	215	365	210	355	235	395
19.0	¾	165	280	190	320	185	310	205	345	200	340	220	375
25.4	1	155	265	175	295	175	295	200	325	195	325	210	355
38.1	1½	150	255	165	280	165	280	185	310	185	310	200	340
50.8	2	140	240	160	270	160	270	180	305	170	290	185	315
76.2	3	135	230	155	260	155	260	170	285	165	280	180	305

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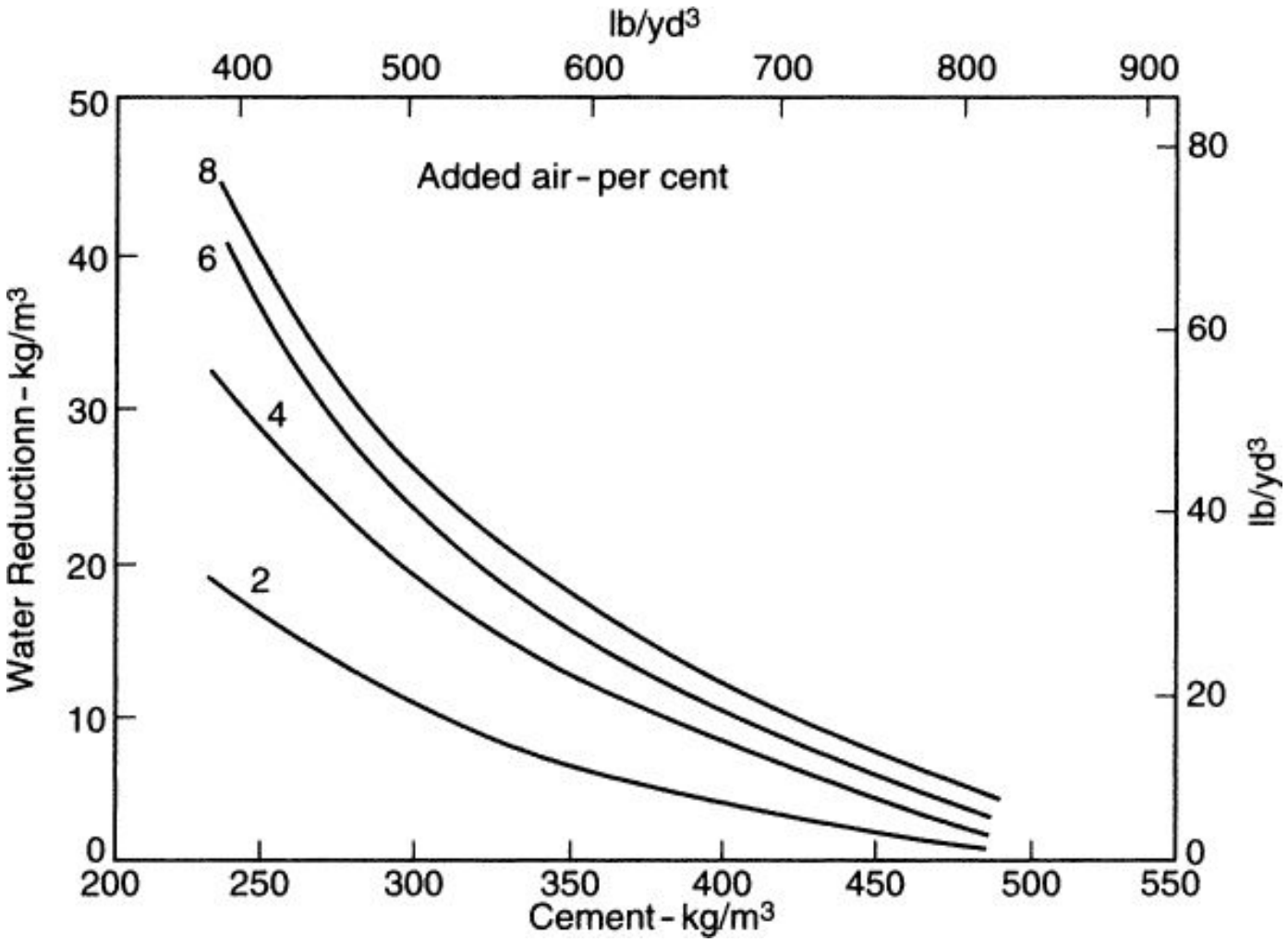


Fig. 4.2. Reduction in mixing water requirement due to addition of air by air entrainment[4.2]

If the water content and the other mix proportions are fixed, workability is governed by the maximum size of aggregate, its grading, shape and texture. The influence of these factors was discussed in Chapter 3. However, the grading and the water/cement ratio have to be considered to-

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gether, as a grading producing the most workable concrete for one particular value of water/cement ratio may not be the best for another value of the ratio. In particular, the higher the water/cement ratio the finer the grading required for the highest workability. In actual fact, for a given value of water/cement ratio, there is one value of the coarse/fine aggregate ratio (using given materials) that gives the highest workability.[4.1] Conversely, for a given workability, there is one value of the coarse/fine aggregate ratio which needs the lowest water content. The influence of these factors was discussed in Chapter 3.

It should be remembered, however, that, although, when discussing gradings of aggregate required for a satisfactory workability, proportions by mass were laid down, these apply only to aggregate of a constant specific gravity. In actual fact, workability is governed by the volumetric proportions of particles of different sizes, so that when aggregates of varying specific gravity are used (e.g. in the case of some lightweight aggreg-

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ates or mixtures of ordinary and lightweight aggregates) the mix proportions should be assessed on the basis of absolute volume of each size fraction. This applies also in the case of air-entrained concrete because the entrained air behaves like weightless fine particles. An example of a calculation on absolute volume basis is given on p. 747. The influence of the properties of aggregate on workability decreases with an increase in the richness of the mix, and possibly disappears altogether when the aggregate/cement ratio is as low as $2\frac{1}{2}$ or 2.

In practice, predicting the influence of mix proportions on workability requires care since, of the three factors, water/cement ratio, aggregate/cement ratio and water content, only two are independent. For instance, if the aggregate/cement ratio is reduced, but the water/cement ratio is kept constant, the water content increases, and consequently the workability also increases. If, on the other hand, the water content is kept constant when the aggregate/cement ratio is reduced, then

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the water/cement ratio decreases but workability is not seriously affected.

The last qualification is necessary because of some secondary effects: a lower aggregate/cement ratio means a higher total surface area of solids (aggregate and cement) so that the same amount of water results in a somewhat decreased workability. This could be offset by the use of a slightly coarser grading of aggregate. There are also other minor factors such as fineness of cement, but the influence of this is still controversial.

Measurement of workability

Unfortunately, there is no acceptable test which will measure directly the workability as given by any of the definitions on p. 187. Numerous attempts have been made, however, to correlate workability with some easily determinable physical measurement, but none of these is fully satisfactory although they may provide useful information within a range of variation in workability.

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Slump test

This is a test used extensively in site work all over the world. The slump test does not measure the workability of concrete, although ACI 116R-90[4.46] describes it as a measure of consistency, but the test is very useful in detecting variations in the uniformity of a mix of given nominal proportions.

The slump test is prescribed by ASTM C 143-10 and BS 1881 : 103 : 1993. The mould for the slump test is a frustum of a cone, 300 mm (12 in.) high. It is placed on a smooth surface with the smaller opening at the top, and filled with concrete in three layers. Each layer is tamped 25 times with a standard 16 mm ($\frac{5}{8}$ in.) diameter steel rod, rounded at the end, and the top surface is struck off by means of a sawing and rolling motion of the tamping rod. The mould must be firmly held against its base during the entire operation; this is facilitated by handles or foot-rests brazed to the mould.

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Immediately after filling, the cone is slowly lifted, and the unsupported concrete will now slump – hence the name of the test. The decrease in the height of the slumped concrete is called *slump*, and is measured to the nearest 5 mm ($\frac{1}{4}$ in.). The decrease is measured to the highest point according to BS EN 12350-2 : 2009, but to the "displaced original center" according to ASTM C 143-10. In order to reduce the influence on slump of the variation in the surface friction, the inside of the mould and its base should be moistened at the beginning of every test, and prior to lifting of the mould the area immediately around the base of the cone should be cleaned of concrete which may have dropped accidentally.

If instead of slumping evenly all round as in a true slump (Fig. 4.3), one half of the cone slides down an inclined plane, a shear slump is said to have taken place, and the test should be repeated. If shear slump persists, as may be the case with harsh mixes, this is an indication of lack of cohesion in the mix.

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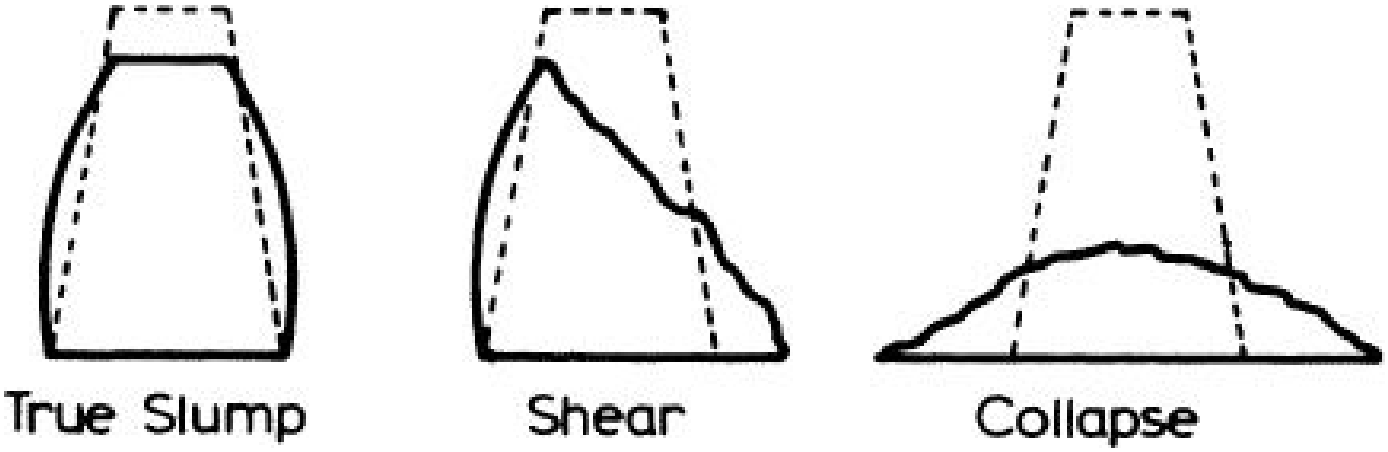


Fig. 4.3. Slump: true, shear, and collapse

Mixes of stiff consistency have a zero slump, so that, in the rather dry range, no variation can be detected between mixes of different workability. Rich mixes behave satisfactorily, their slump being sensitive to variations in workability. However, in a lean mix with a tendency to harshness, a true slump can easily change to the shear type, or even to collapse (Fig. 4.3), and widely different values of slump can be obtained in different samples from the same mix.

The approximate magnitude of slump for different workabilities (in a modified form of Bartos' proposals[4.56]) is given in Table 4.2. Table 4.3 gives the proposed European classification of BS EN 206-1 : 2000. One reason for the differen-

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ce between the two tables is that the European approach is to measure slump to the nearest 10 mm. It should be remembered, however, that with different aggregates, especially a different content of fine aggregate, the same slump can be recorded for different workabilities, as indeed the slump bears no unique relation to the workability as defined earlier. Moreover, slump does not measure the ease of compaction of concrete and, as slump occurs under the self-weight of the test concrete only, it does not reflect behaviour under dynamic conditions such as vibration, finishing, pumping or moving through a tremie. Rather, slump reflects the 'yield' of concrete.[4.110]

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Table 4.2. Description of Workability and Magnitude of Slump

Description of workability	*Slump*	
	mm	*in.*
No slump	0	0
Very low	5–10	$\frac{1}{4}$–$\frac{1}{2}$
Low	15–30	$\frac{3}{4}$–$1\frac{1}{4}$
Medium	35–75	$1\frac{1}{2}$–3
High	80–155	$3\frac{1}{4}$–6
Very high	160 to collapse	$6\frac{1}{4}$ to collapse

Table 4.3. Classification of Workability and Magnitude of Slump According to BS EN 206-1 : 2000

Classification of workability	*Slump mm*
S1	10–40
S2	50–90
S3	100–150
S4	⩾160

Despite these limitations, the slump test is very useful on the site as a check on the batch-to-batch or hour-to-hour variation in the materials

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being fed into the mixer. An increase in slump may mean, for instance, that the moisture content of aggregate has unexpectedly increased; another cause would be a change in the grading of the aggregate, such as a deficiency of sand. Too high or too low a slump gives immediate warning and enables the mixer operator to remedy the situation. This application of the slump test, as well as its simplicity, is responsible for its widespread use.

A mini-slump test was developed for the purpose of assessing the influence of various water-reducing admixtures and superplasticizers on neat cement paste.[4.105] The test may be useful for that specific purpose, but it is important to remember that the workability of concrete is affected also by factors other than the flow properties of the constituent cement paste.

Compacting factor test

There is no generally accepted method of directly measuring the amount of work necessary to achieve full compaction, which is a definition of

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workability.[4.1] Probably the best test yet available uses the inverse approach: the degree of compaction achieved by a standard amount of work is determined. The work applied includes perforce the work done against the surface friction but this is reduced to a minimum, although probably the actual friction varies with the workability of the mix.

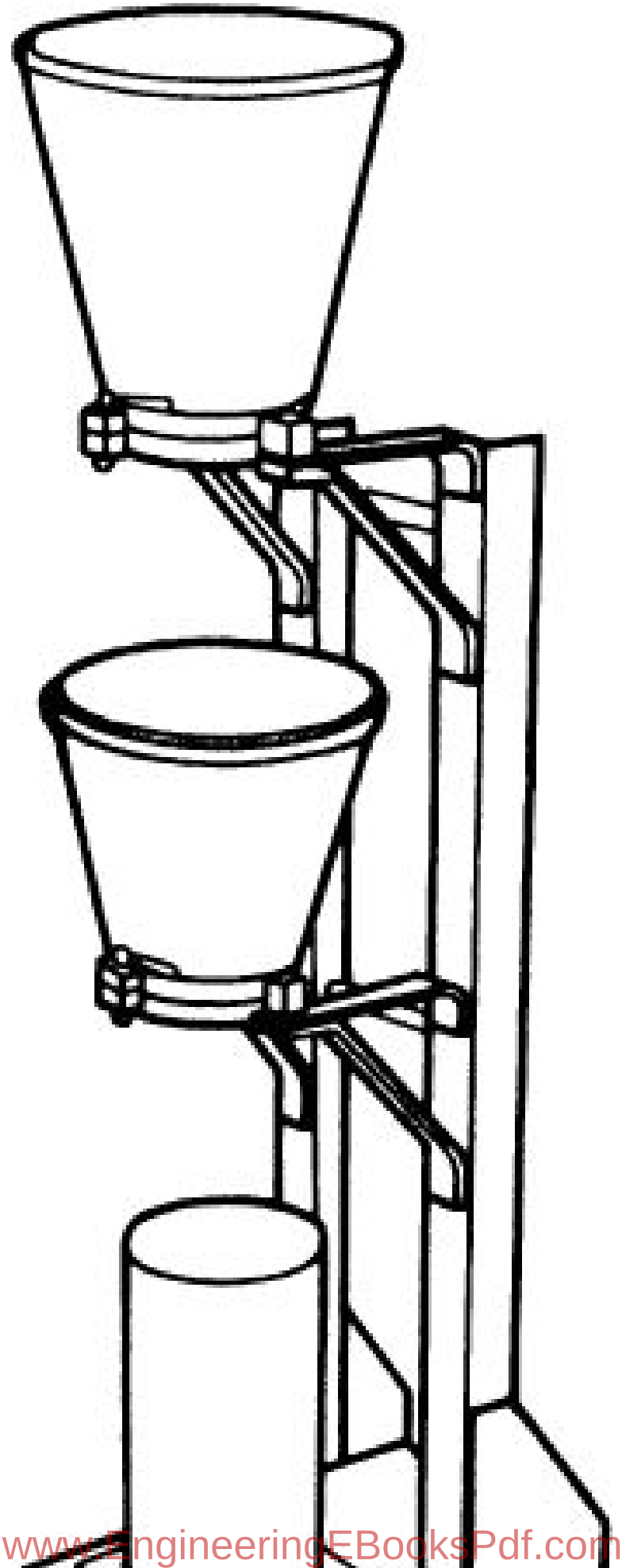
The degree of compaction, called the *compacting factor*, is measured by the density ratio, i.e. the ratio of the density actually achieved in the test to the density of the same concrete fully compacted.

The test, known as the compacting factor test, is described in BS 1881-103 : 1993 and in ACI 211.3-75 (Revised 1987) (Reapproved 1992),[4.70] and is appropriate for concrete with a maximum size of aggregate up to 40 mm (or $1\frac{1}{2}$ in.). The apparatus consists essentially of two hoppers, each in the shape of a frustum of a cone, and one cylinder, the three being above one another. The hoppers have hinged doors at the bottom, as shown

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in Fig. 4.4. All inside surfaces are polished to reduce friction.

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Fig. 4.4. Compacting factor apparatus

The upper hopper is filled with concrete, this being placed gently so that at this stage no work is done on the concrete to produce compaction. The bottom door of the hopper is then released and the concrete falls into the lower hopper. This is smaller than the upper one and is, therefore, filled to overflowing, and thus always contains approximately the same amount of concrete in a standard state; this reduces the influence of the personal factor in filling the top hopper. The bottom door of the lower hopper is then released and the concrete falls into the cylinder. Excess concrete is cut by two floats slid across the top of the mould, and the net mass of concrete in the known volume of the cylinder is determined.

The density of the concrete in the cylinder is now calculated, and this density divided by the density of the fully compacted concrete is defined as the compacting factor. The latter density can be obtained by actually filling the cylinder with concrete in four layers, each tamped or vi-

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brated, or alternatively calculated from the absolute volumes of the mix ingredients. The compacting factor can also be calculated from the reduction in volume that occurs when a defined volume of partially compacted concrete (by passing through the hoppers) is fully compacted.

The compacting factor apparatus shown in Fig. 4.4 is about 1.2 m (4 ft) high and its use is generally limited to pavement construction and precast concrete manufacture.

Table 4.4 lists values of the compacting factor for different workabilities.[4.3] Unlike the slump test, variations in the workability of dry concrete are reflected in a large change in the compacting factor, i.e. the test is more sensitive at the low workability end of the scale than at high workability. However, very dry mixes tend to stick in one or both hoppers and the material has to be eased gently by poking with a steel rod. Moreover, it seems that for concrete of very low workability the actual amount of work required for full compaction depends on the richness of the mix while

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the compacting factor does not: leaner mixes need more work than richer ones.[4.4] This means that the implied assumption that all mixes with the same compacting factor require the same amount of useful work is not always justified. Likewise, the assumption, mentioned earlier, that the wasted work represents a constant proportion of the total work done regardless of the properties of the mix is not quite correct. Nevertheless, the compacting factor test undoubtedly provides a good measure of workability.

Table 4.4. Description of Workability and Compacting Factor[4.3]

Description of workability	*Compacting factor*	*Corresponding slump mm*
Very low	0.78	0–25
Low	0.85	25–50
Medium	0.92	50–100
High	0.95	100–175

ASTM flow test

This laboratory test gives an indication of the consistency of concrete and its proneness to se-

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gregation by measuring the spread of a pile of concrete on a table subjected to jolting. This test also gives a good assessment of consistency of stiff, rich, and rather cohesive mixes. The test was covered by ASTM C 124-39 (Reapproved 1966) which was withdrawn in 1974 because the test was little used, rather than because it was thought to be not appropriate.

Remoulding test

Use of a jolted table is made in another test, in which an assessment of workability is made on the basis of the effort involved in changing the shape of a sample of concrete. This is the re-moulding test, developed by Powers.[4.5]

The apparatus is shown diagrammatically in Fig. 4.5. A standard slump cone is placed in a cylinder 305 mm (12 in.) in diameter and 203 mm (8 in.) high, the cylinder being mounted rigidly on a flow table, adjusted to give a 6.3 mm ($\frac{1}{4}$ in.) drop. Inside the main cylinder, there is an inner ring, 210 mm ($8\frac{1}{4}$ in.) in diameter and 127 mm (5 in.)

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high. The distance between the bottom of the inner ring and the bottom of the main cylinder can be set between 67 and 76 mm ($2\frac{5}{8}$ and 3 in.).

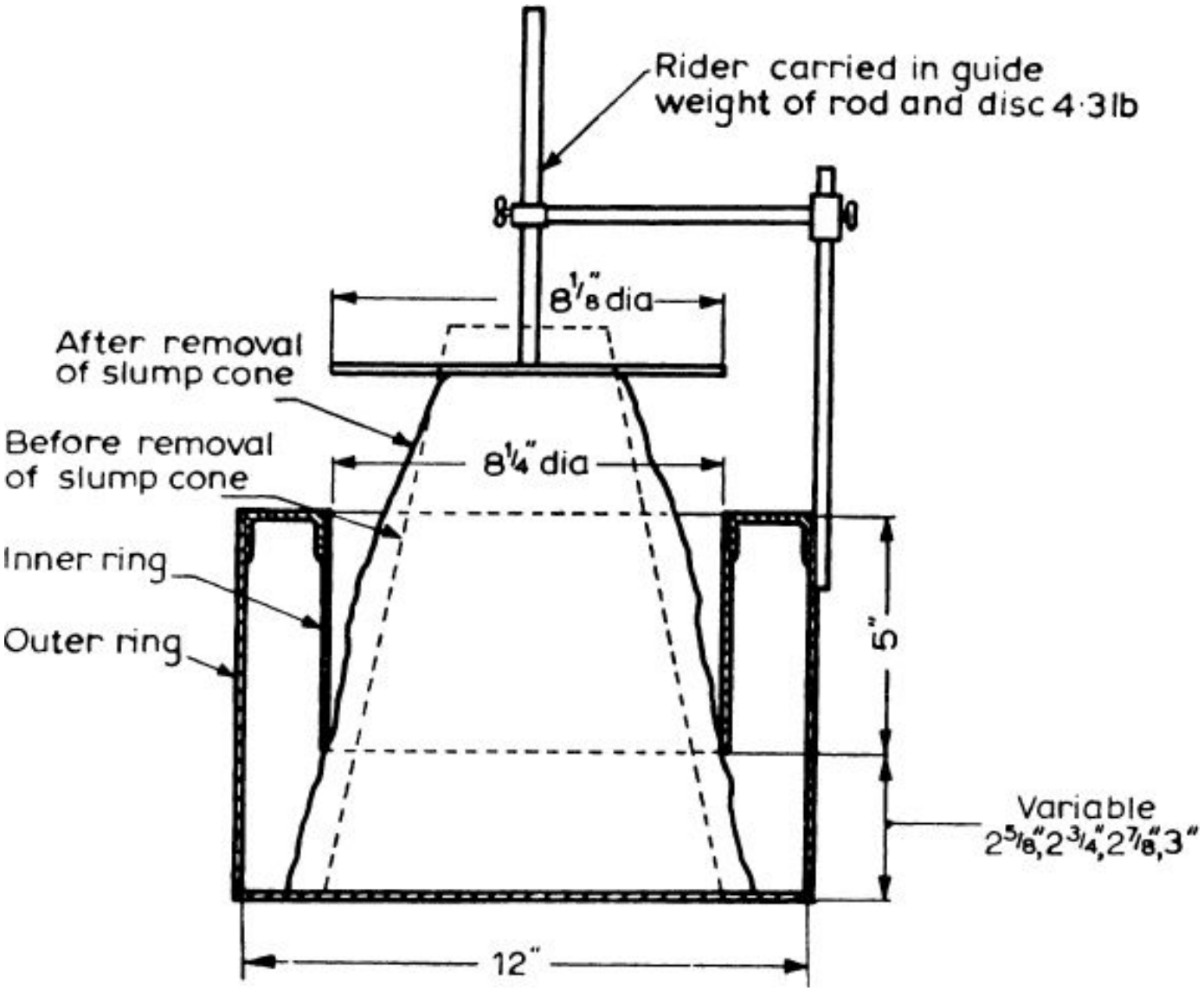


Fig. 4.5. Remoulding test apparatus

The slump cone is filled in the standard manner, removed, and a disc-shaped rider (weighing 1.9 kg (4.3 lb)) is placed on top of the concrete.

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The table is now jolted at the rate of one jolt per second until the bottom of the rider is 81 mm ($3\frac{3}{16}$ in.) above the base plate. At this stage, the shape of the concrete has changed from a frustum of a cone to a cylinder. The effort required to achieve this remoulding is expressed as the number of jolts required. For very dry mixes a considerable effort may be necessary.

The test is purely a laboratory one but is valuable because the remoulding effort appears to be closely related to workability.

Vebe test

This is a development of the remoulding test in which the inner ring of Powers' apparatus is omitted and compaction is achieved by vibration instead of jolting. The apparatus is shown diagrammatically in Fig. 4.6. The name 'Vebe' is derived from the initials of V. Bährner of Sweden who developed the test. The test is covered by BS EN 12350-3 : 2009; it is referred to also in ACI 211.3-75 (Revised 1987).[4.70]

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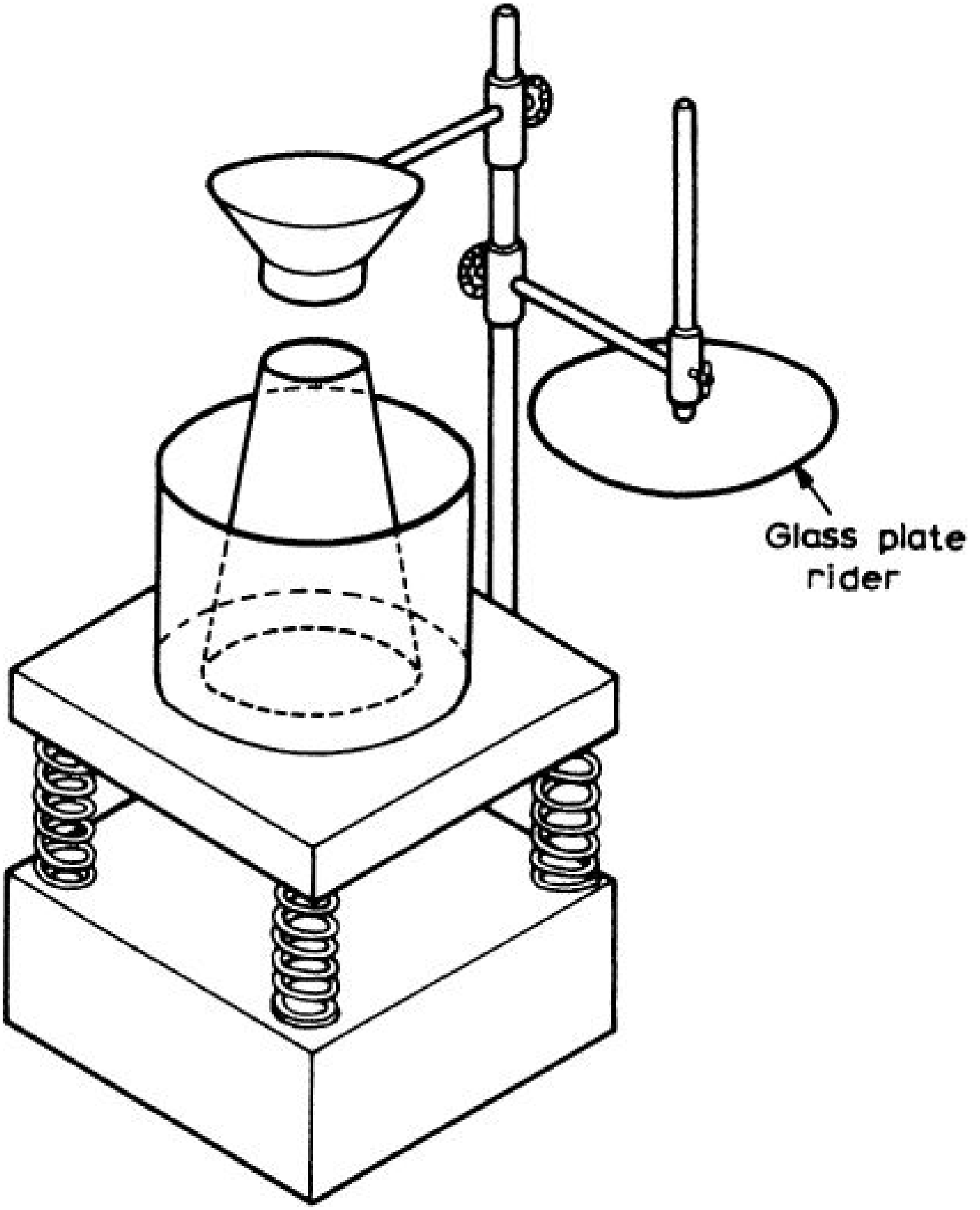


Fig. 4.6. Vebe apparatus

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The remoulding is assumed to be complete when the glass plate rider is completely covered with concrete and all cavities in the surface of the concrete have disappeared. This is judged visually, and the difficulty of establishing the end point of the test may be a source of error. To overcome it, an automatically operated device for recording the movement of the plate against time may be fitted.

Compaction is achieved using a vibrating table with an eccentric mass rotating at 50 to 60 Hz and a maximum acceleration of $3g$ to $4g$. It is assumed that the input of energy required for compaction is a measure of workability of the mix, and this is expressed as the time in seconds, called *Vebe time*, required for the remoulding to be complete. Sometimes, a correction for the change in the volume of concrete from V_2 before, to V_1 after, vibration is applied, the time being multiplied by V_2/V_1. The test is appropriate for mixes with a Vebe time between 3 and 30 seconds.

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Vebe is a good laboratory test, particularly from very dry mixes. This is in contrast to the compacting factor test where error may be introduced by the tendency of some dry mixes to stick in the hoppers. The Vebe test also has the additional advantage that the treatment of concrete during the test is comparatively closely related to the method of placing in practice. Both the Vebe test and the remoulding tests determine the time required to achieve compaction, which is related to the total work done.

Flow table test

This test, which was developed in Germany in 1933, was covered by BS 1881 : 105 : 1984. The test is appropriate for concrete of high and very high workability, including flowing concrete (see p. 259) which would exhibit a collapse slump.

The apparatus consists essentially of a wooden board covered by a steel plate with a total mass of 16 kg. This board is hinged along one side to a base board, each board being 700 mm square.

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The upper board can be lifted up to a stop so that the free edge rises 40 mm. Appropriate markings indicate the location of the concrete to be deposited on the table.

The table top is moistened and a frustum of a cone of concrete, lightly tamped by a wooden tamper in a prescribed manner, is placed using a mould 200 mm high with a bottom diameter of 200 mm and a top diameter of 130 mm. Excess concrete is removed, the surrounding table top is cleaned and, after an interval of 30 seconds, the table top is lifted 15 times in a period of 45 to 75 seconds, this motion avoiding a significant force against the stop. In consequence, the concrete spreads and the maximum spread parallel to the two edges of the table is measured. The average of these two values, given to the nearest millimetre, represents the *flow*. The test is appropriate for mixes having a flow of 340 to 600 mm. If the concrete at this stage does not appear uniform and cohesive, this is an indication of a lack of co-

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hesiveness of the mix. The current standard is BS EN 12350-5 : 2010.

A laboratory investigation[4.39] has shown a linear relation between flow and slump, but the tests were limited in scope in that they involved only one aggregate type and only one aggregate grading. Also, the effect of site conditions was not included. In consequence, no generalization can be inferred from the data which were published, and it would be unwise to view the slump test and the flow test as generally interchangeable. In essence, the two tests do not measure the same physical phenomena so that there is no reason to expect a single relationship between the two when grading or aggregate shape or content of fine material in the mix vary. For practical purposes, an appropriate test should be adopted. Such a test makes it possible to recognize a departure from the specified mix proportions, and this is what matters on site.

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Ball penetration test and compactability test

This is a simple field test consisting of the determination of the depth to which a 152 mm (6 in.) diameter metal hemisphere, weighing 13.6 kg (30 lb), will sink under its own weight into fresh concrete. A sketch of the apparatus, devised by J. W. Kelly and known as the Kelly ball, is shown in Fig. 4.7.

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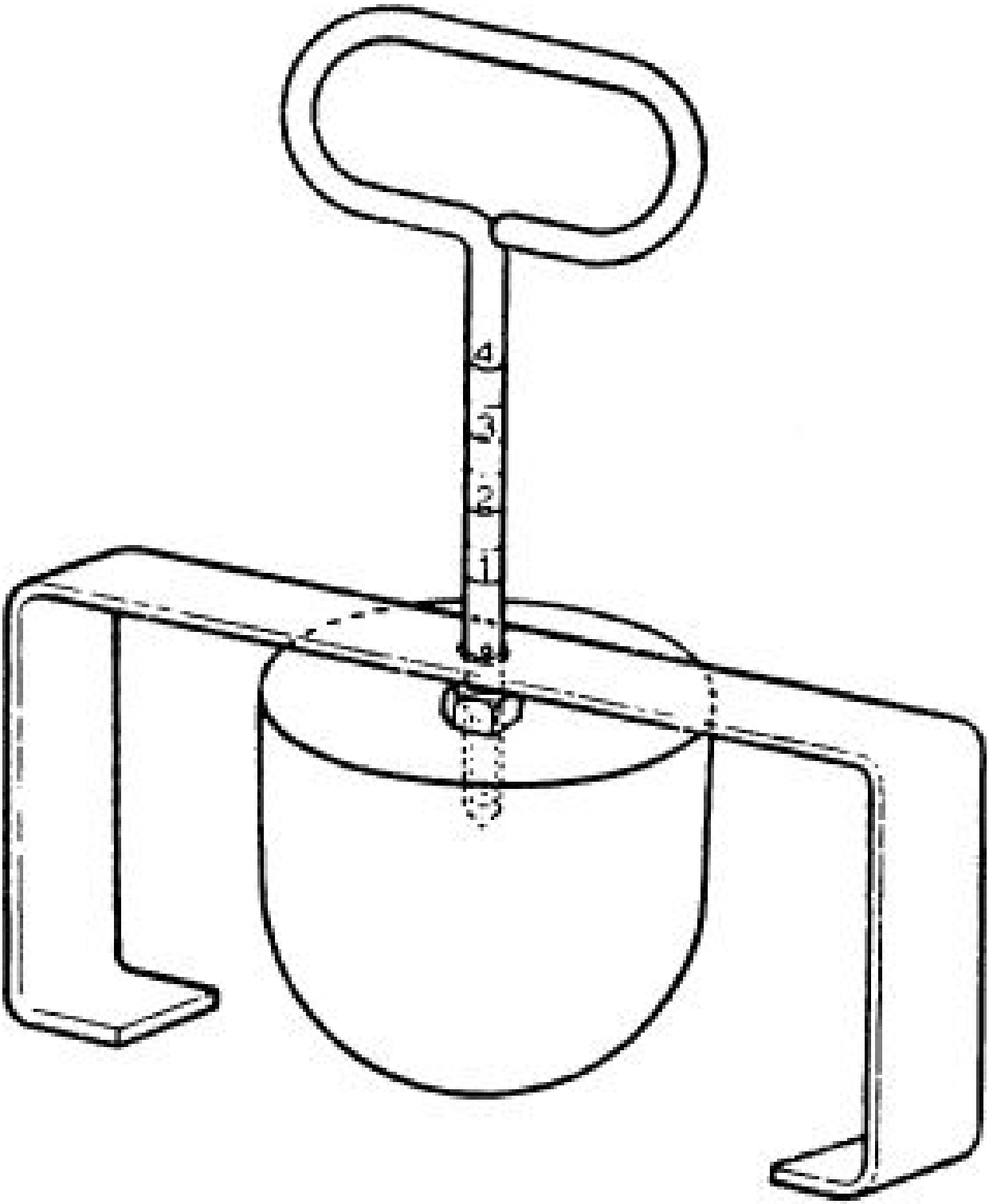


Fig. 4.7. Kelly ball

The use of this test is similar to that of the slump test, that is, routine checking of consist-

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ency for control purposes. The test is essentially an American one, and is rarely used elsewhere. It is, however, worth considering the Kelly ball test as an alternative to the slump test, over which it has some advantages. In particular, the ball test is simpler and quicker to perform and, what is more important, it can be applied to concrete in a buggy or actually in the form. In order to avoid the effects of a boundary, the depth of the concrete being tested should be not less than 200 mm (8 in.), and the least lateral dimension should be 460 mm (18 in.).

As would be expected, there is no simple correlation between penetration and slump, since neither test measured any basic property of concrete but only the response to specific conditions. On a site, when a particular mix is used, correlation can be found, as shown for instance in Fig. 4.8.[4.6] In practice, the ball test is essentially used to measure variations in the mix, such as those due to a variation in the moisture content of the aggregate.

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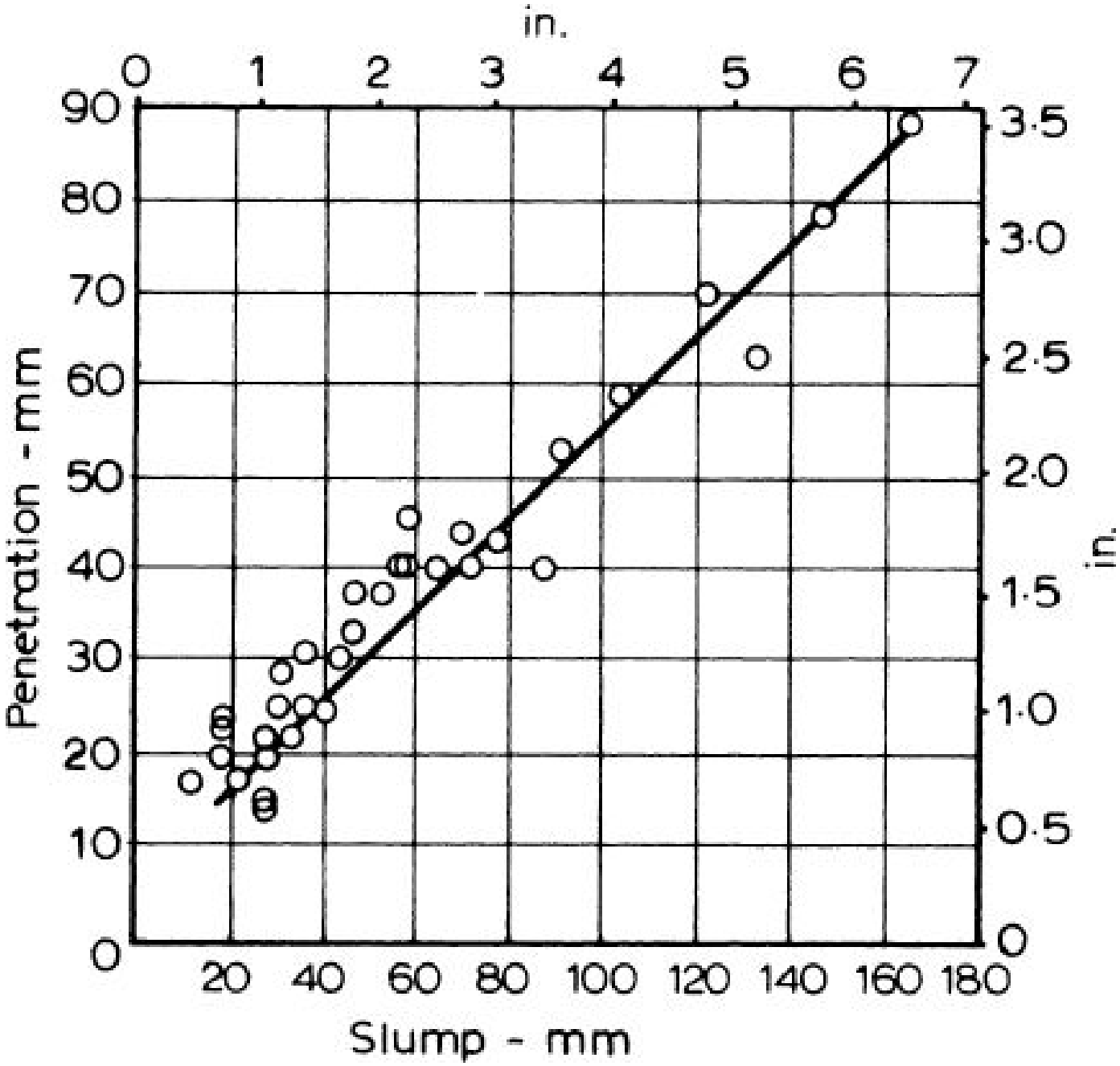


Fig. 4.8. Relation between Kelly ball penetration and slump[4.6]

A compactability test was introduced by BS EN 12350-4 : 2009, which determines the reduction in volume of a loosely-packed concrete after

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vibration in a cylinder. The *degree of compactability* is the ratio of the height of the cylinder to the height of the compacted concrete. The compaction is effected by a vibrating table or by an internal vibrator.

Nasser's K-tester

Among the various attempts to devise a simple workability test, the probe test of Nasser[4.41] deserves mention. This test uses a hollow probe 19 mm ($\frac{3}{4}$ in.) in diameter with openings through which mortar can enter the tube. The probe is inserted vertically into fresh concrete in situ (and thus avoids using a sample). The height of the mortar in the tube after 1 minute and also the residual height following withdrawal of the probe are measured.

It is claimed,[4.42,4.106] that these readings give an indication of consistency and workability of the concrete because the probe readings are affected by cohesive, adhesive, and friction forces within the mix. Thus, an over-wet mix, which ex-

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hibits a high slump, would lead to a relatively low level of mortar retained in the probe, this being the result of segregation. The residual height of mortar in the probe appears to be related to slump, providing this does not exceed 80 mm (or 3 in.).[4.41] However, the K-tester can be used even for flowing concrete.[4.106] The K-tester has not been standardized and is not widely used.

Two-point test

Tattersall[4.43] has repeatedly criticized all the existing workability tests on the grounds that they measure only one parameter. His argument is that the flow of fresh concrete should be described by the Bingham model, i.e. by the equation

$$\tau = \tau_0 + \mu\dot{\gamma}$$

where τ = shear stress at rate of shear $\dot{\gamma}$

τ_0 = yield stress and

μ = plastic viscosity.

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Because there are two unknowns, measurements at two rates of shear are required; hence, the name 'two-point test'. The yield stress represents the threshold value for flow to begin and is closely related to slump.[4.107] The plastic viscosity reflects the increase in shear stress with an increase in the rate of shear.

Tattersall[4.43] developed techniques of torque measurement using a modified food mixer. Hence, he deduced experimentally data related to the shear stress at a given rate of shear and to constants representing the yield stress, τ_0, and plastic viscosity, μ, of the mix. It is the latter two that, in his view, provide a measure of the fundamental rheological properties of concrete. Their determination requires the measurement of torque to rotate the mixer at two speeds. This apparatus was modified both by Tattersall[4.43] and by Wallevik and Gjørv[4.104] who claim that their apparatus is more reliable and, in addition, gives a quantified measure of the susceptibility of the mix to segregation.

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Problems in use are that the apparatus is cumbersome, complicated, and requires skill in interpretation of the test readings, which are not directly usable, unlike slump. For these reasons, the two-point test is inappropriate for site operation as a means of control, but may be of value in the laboratory.

With respect to two-point description of workability, it is worth noting that, for robot-placed concrete, it is important to establish the value of plastic viscosity and yield stress of concrete, and the variation in these two parameters with temperature and time since mixing. Equations predicting viscosity on the basis of the viscosity equation for high-concentration suspensions, taking into account aggregate properties and using experimental constants, were developed by Murata and Kikukawa.[4.107] They also developed an equation for the yield value of concrete based on slump. The validity of this approach is yet to be proven.

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Comparison of tests

It should be said at the outset that no comparison is really possible as each test measures the behaviour of concrete under different conditions. The particular uses of each test have been mentioned but it is worth adding that BS 1881 : 1983 (withdrawn but useful) lists the test methods appropriate to mixes of different workability as shown in Table 4.5.

Table 4.5. Test Methods Apropriate to Mixes of Different Workability According to BS 1881 : 1983

Workability	*Method*
Very low	Vebe time
Low	Vebe time, compacting factor
Medium	Compacting factor, slump
High	Compacting factor, slump, flow
Very high	Flow

The compacting factor test is closely related to the reciprocal of workability, whereas the re-

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moulding, flow, and Vebe tests are direct functions of workability. The Vebe test measures the properties of concrete under vibration as compared with the free-fall conditions of the compacting factor test and the jolting in the remoulding and flow tests. All four tests are satisfactory in the laboratory, but the compacting factor apparatus is also suitable for site use.

An indication of the relation between the compacting factor and the Vebe time is given by Fig. 4.9, but this applies only to the mixes used, and the relation must not be assumed to be generally applicable because it depends on factors such as the shape and texture of the aggregate or presence of entrained air, as well as on mix proportions. For specific mixes, the relation between compacting factor and slump has been obtained, but such a relation is also a function of the properties of the mix. The relation between the number of jolts in Powers' remoulding test and slump (Fig. 4.10) is also only broadly defined.[4.58] A general indication of the pattern of the relation between the

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compacting factor, Vebe time and slump is shown in Fig. 4.11.[4.14] The influence of the richness of the mix in two of these relations is clear. The absence of influence in the case of the relation between slump and Vebe time is illusory because slump is insensitive at one end of the scale (low workability) and Vebe time is insensitive at the other end; thus two asymptotic lines with a small connecting part are present.

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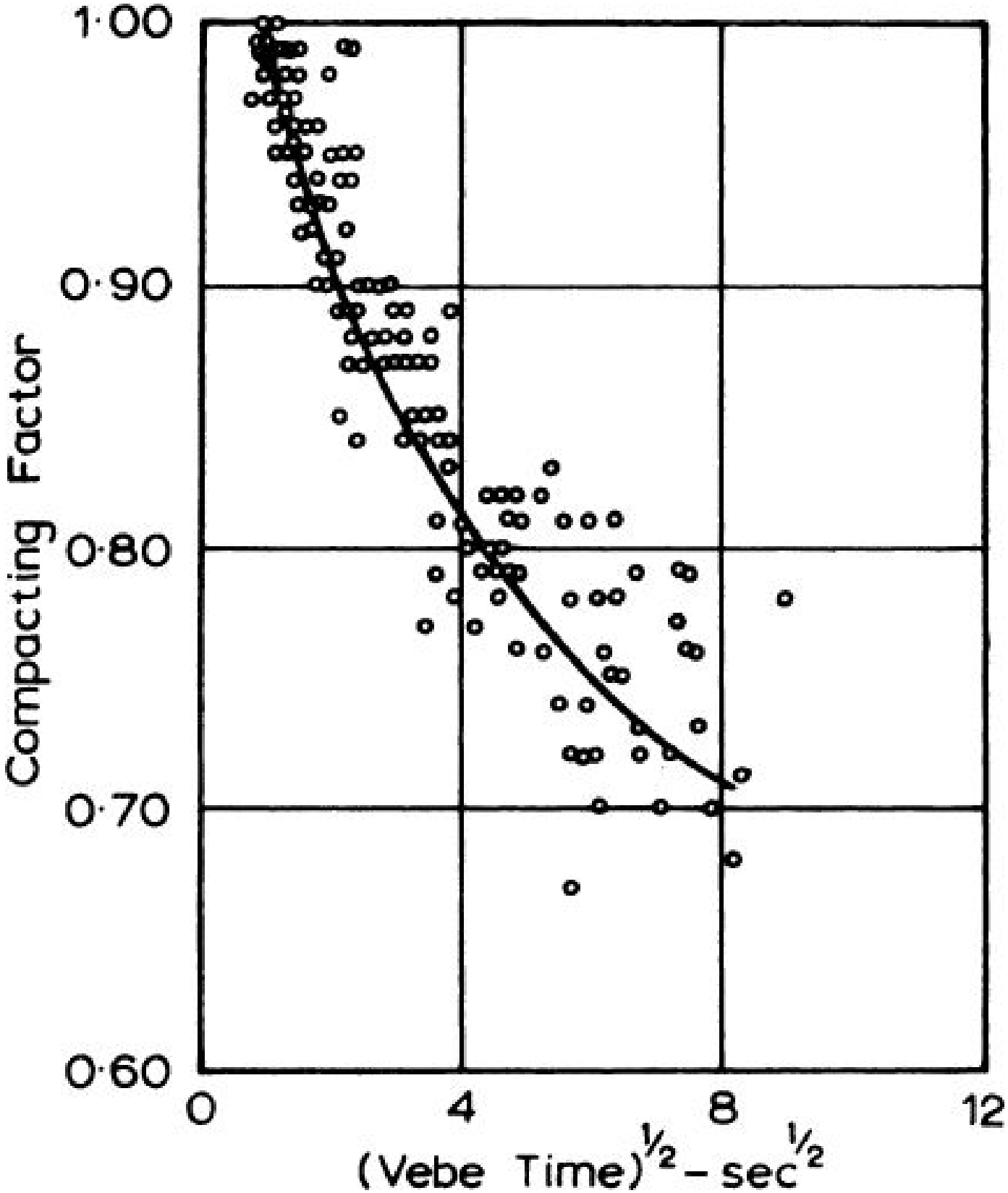


Fig. 4.9. Relation between compacting factor and Vebe time[4.4]

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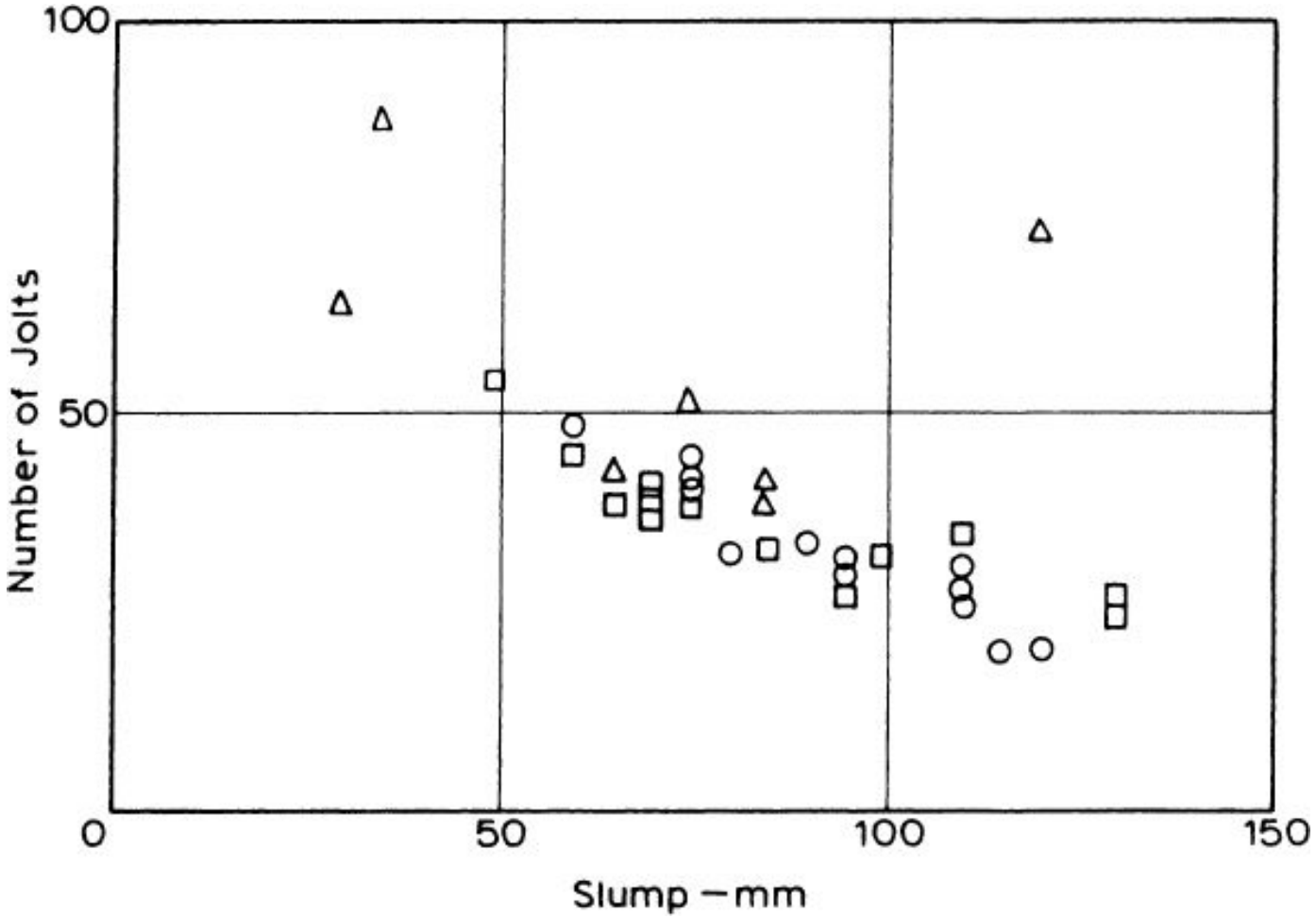
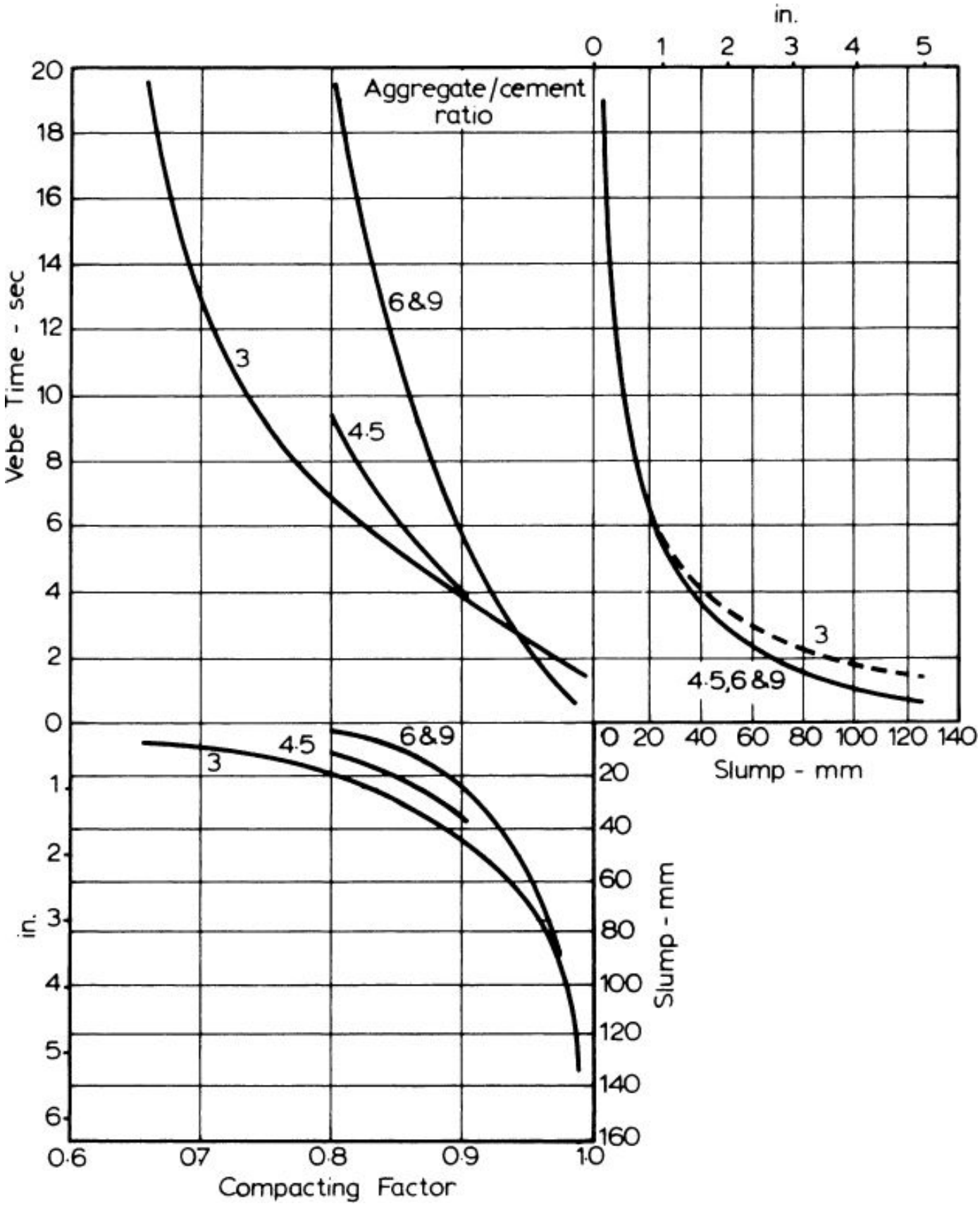


Fig. 4.10. Relation between the number of jolts using Powers' remoulding test apparatus and slump for mixes with fine aggregates of different fineness[4.58]

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Fig. 4.11. General pattern of relations between workability tests for mixes of varying aggregate/cement ratios[4.14]

The flow test is valuable in assessing the cohesiveness and workability of very high workability concrete or flowing concrete.

The slump and penetration tests are purely comparative and, in that capacity, both are very useful except that the slump test is unreliable with lean mixes, for which good control is often of considerable importance. The slump test is periodically attacked as useless and as a poor indicator of the strength of concrete.[4.52,4.111] Such criticism may well be misplaced because the slump test does not purport to measure the potential strength of concrete: the purpose of the slump test is to verify the uniformity of the slump from batch to batch; and no more. Such a verification is useful in that it ensures that the concrete, as placed, has the desired workability. Moreover, the mere knowledge that testing is under way

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concentrates the mind at the batching plant, and the psychological effect of this knowledge is to prevent a lapse into the ‘anything-will-do’ attitude.

It has to be admitted that the slump test, which represents a single rate of shear situation, cannot fully characterize the workability of concrete. The test can, however, give a comparative value of workability if the only variable is the water content of the mix because, under such circumstances, the straight lines representing the Bingham equations do not cross one another.[4.43] A perfect, practical test for workability has yet to be devised. Although this seems primitive, there is value in visual assessment of workability by patting concrete with a trowel in order to see the ease of finishing. Experience is clearly necessary but, once it has been acquired, the ‘by eye’ test, particularly for the purpose of checking uniformity, is both rapid and reliable.

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Stiffening time of concrete

It is possible to determine whether concrete has stiffened to a given degree by testing mortar sieved out of the concrete, using a 5 mm (No. 4 ASTM) sieve. A spring reaction-type probe, known as Proctor probe, is used to determine the times when the resistance to penetration is 3.5 MPa (500 psi) and 27.6 MPa (4000 psi). The former is referred to as initial setting time and indicates that the concrete has become too stiff to be made mobile by vibration. The time when the resistance to penetration has reached 27.6 MPa (4000 psi) is the final setting time; the compressive strength of concrete measured on a standard cylinder is then about 0.7 MPa (100 psi). These setting times are distinct from the setting times of cement.

The test method is prescribed by ASTM C 403-08 and can be used for comparative purposes. It cannot be an absolute measure because the test is performed on mortar and not on the parent concrete. British Standard BS 5075-1 :

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1982 (superseded by BS EN 480 and 934) also prescribes a stiffening time test.

Effect of time and temperature on workability

Freshly mixed concrete stiffens with time. This should not be confused with setting of cement. It is simply that some water from the mix is absorbed by the aggregate if not saturated, some is lost by evaporation, particularly if the concrete is exposed to sun or wind, and some is removed by the initial chemical reactions. The compacting factor decreases by up to about 0.1 during a period of one hour from mixing.

The exact value of the loss in workability depends on several factors. First, the higher the initial workability the greater the slump loss. Second, the rate of loss of slump is higher in rich mixes. Furthermore, the rate of loss depends on the properties of the cement used: the rate is higher when the alkali content is high[4.108] and when the sulfate content is too low.[4.62] An example of

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the slump-time relation for concrete made with a water/cement ratio of 0.4 and cement having an alkali content of 0.58 is shown in Fig. 4.12.[4.60]

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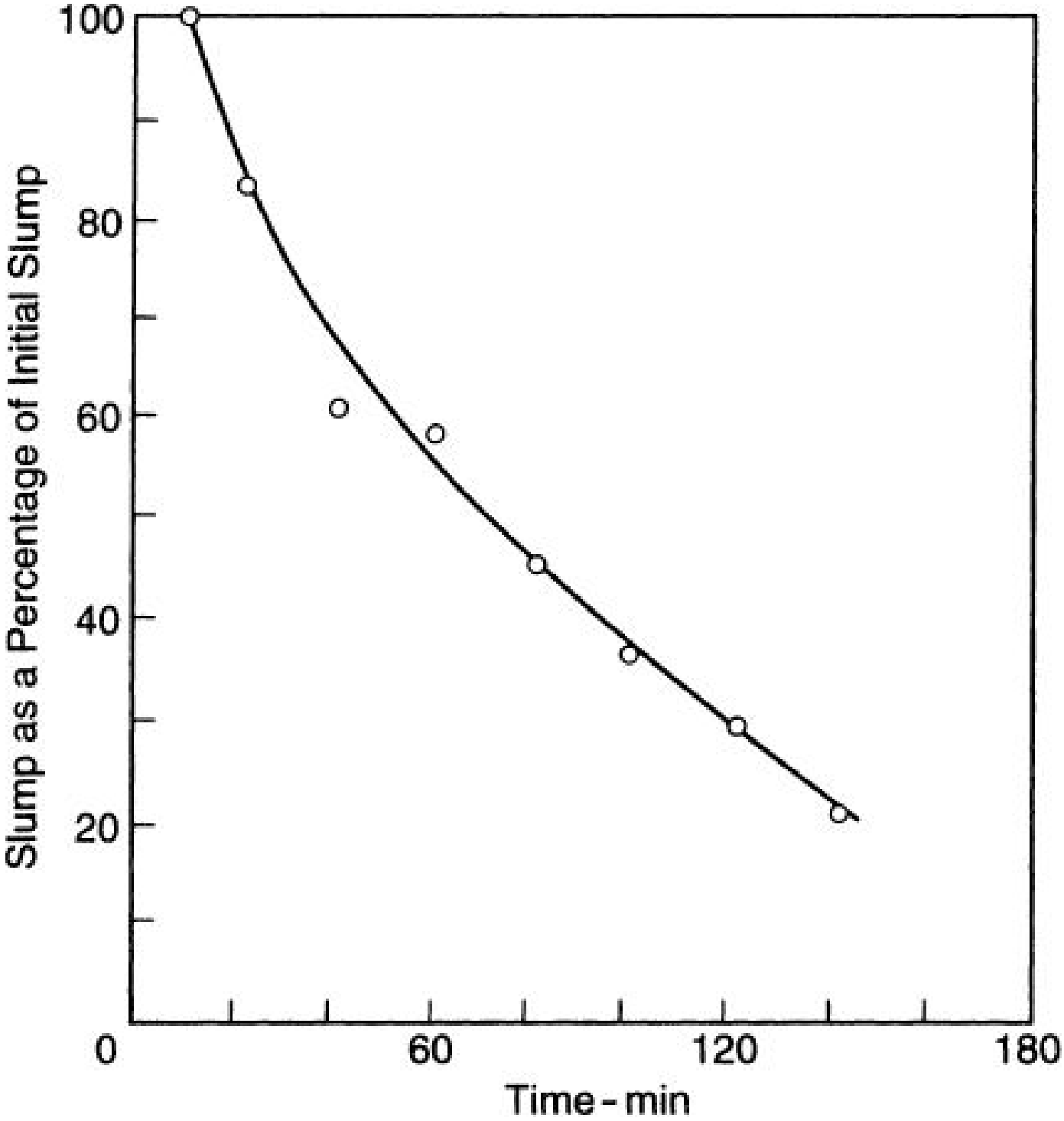


Fig. 4.12. Loss of slump with time since mixing (based on ref. 4.60)

The change in workability with time depends also on the moisture condition of aggregate (at

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a given total water content): the loss is greater with dry aggregate due to the absorption of water by aggregate, as of course would be expected. Water-reducing admixtures, although they delay the initial stiffening of concrete, often lead to a somewhat increased rate of loss of slump with time.

The workability of a mix is also affected by the ambient temperature, although, strictly speaking, we are concerned with the temperature of the concrete itself. Figure 4.13 gives an example of the effect of temperature on slump of laboratory-mixed concrete:[4.7] it is apparent that on a hot day the water content of the mix would have to be increased for a constant early workability to be maintained. The loss of slump in stiff mixes is less influenced by temperature because such mixes are less affected by changes in water content. Figure 4.14 shows that as the concrete temperature increases the percentage increase in water required to effect a 25 mm (1 in.) change in slump also increases.[4.8] The loss of slump with

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time is also affected by the temperature, as shown in Fig. 4.15.

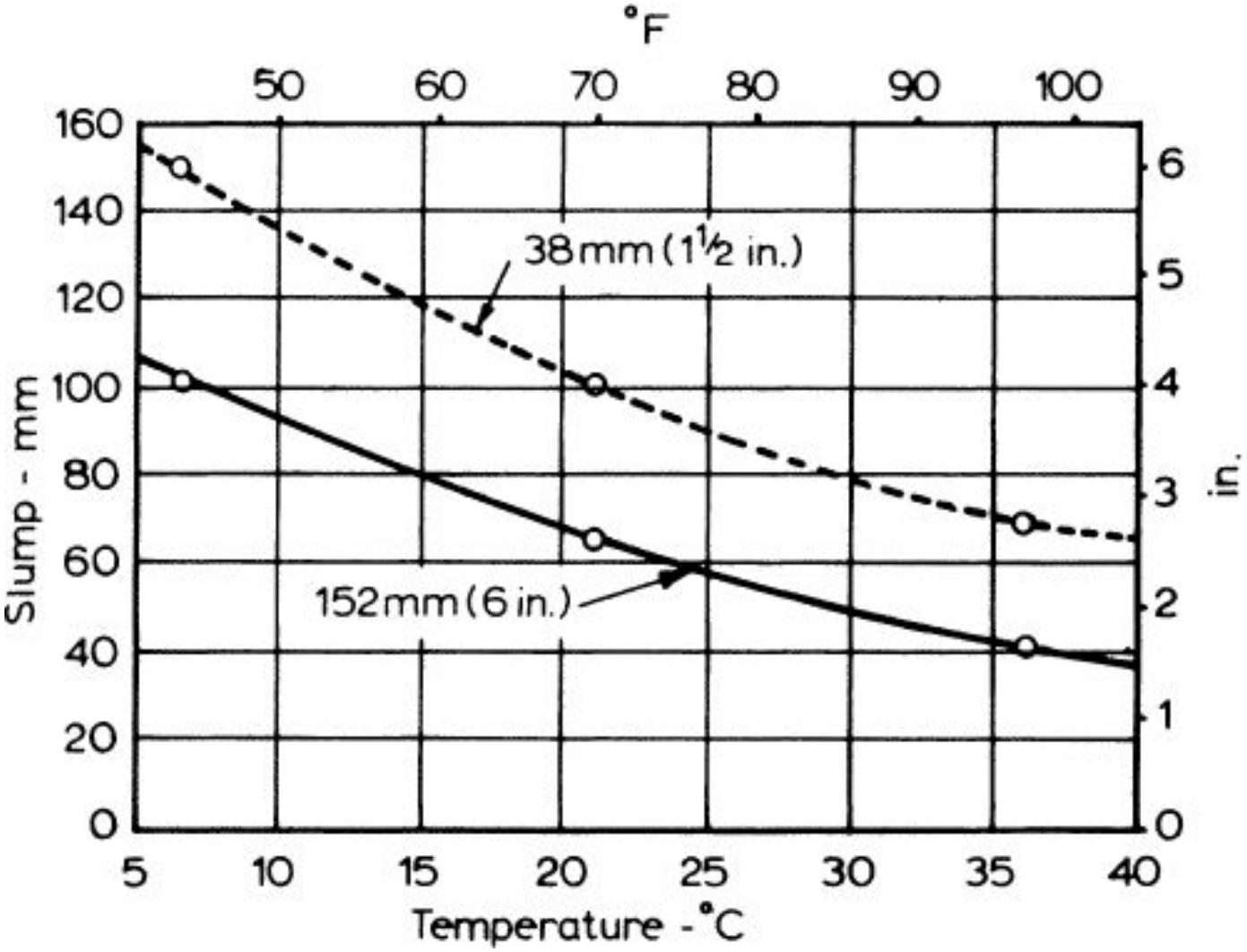


Fig. 4.13. Influence of temperature on slump of concretes with different maximum aggregate size[4.7]

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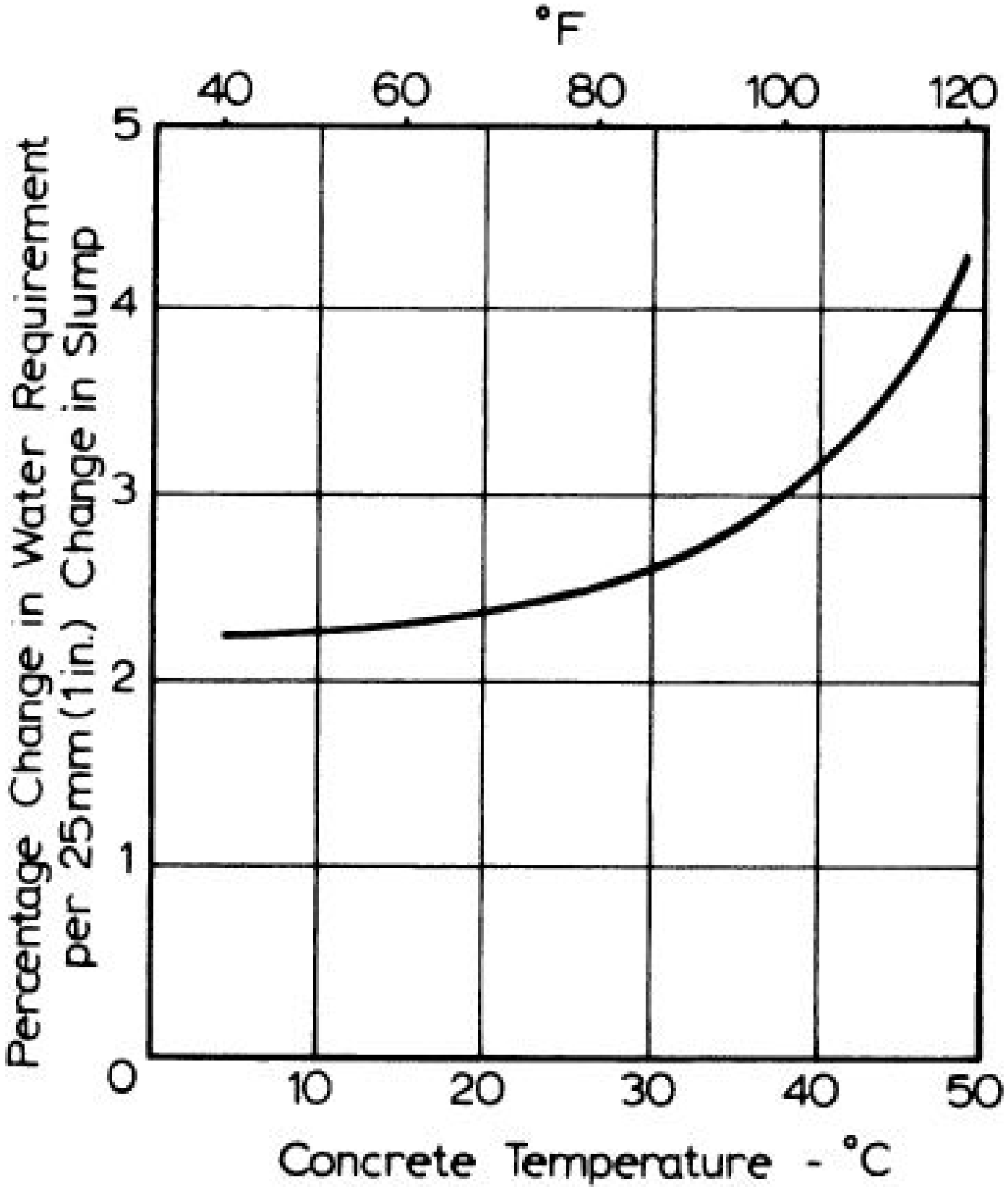
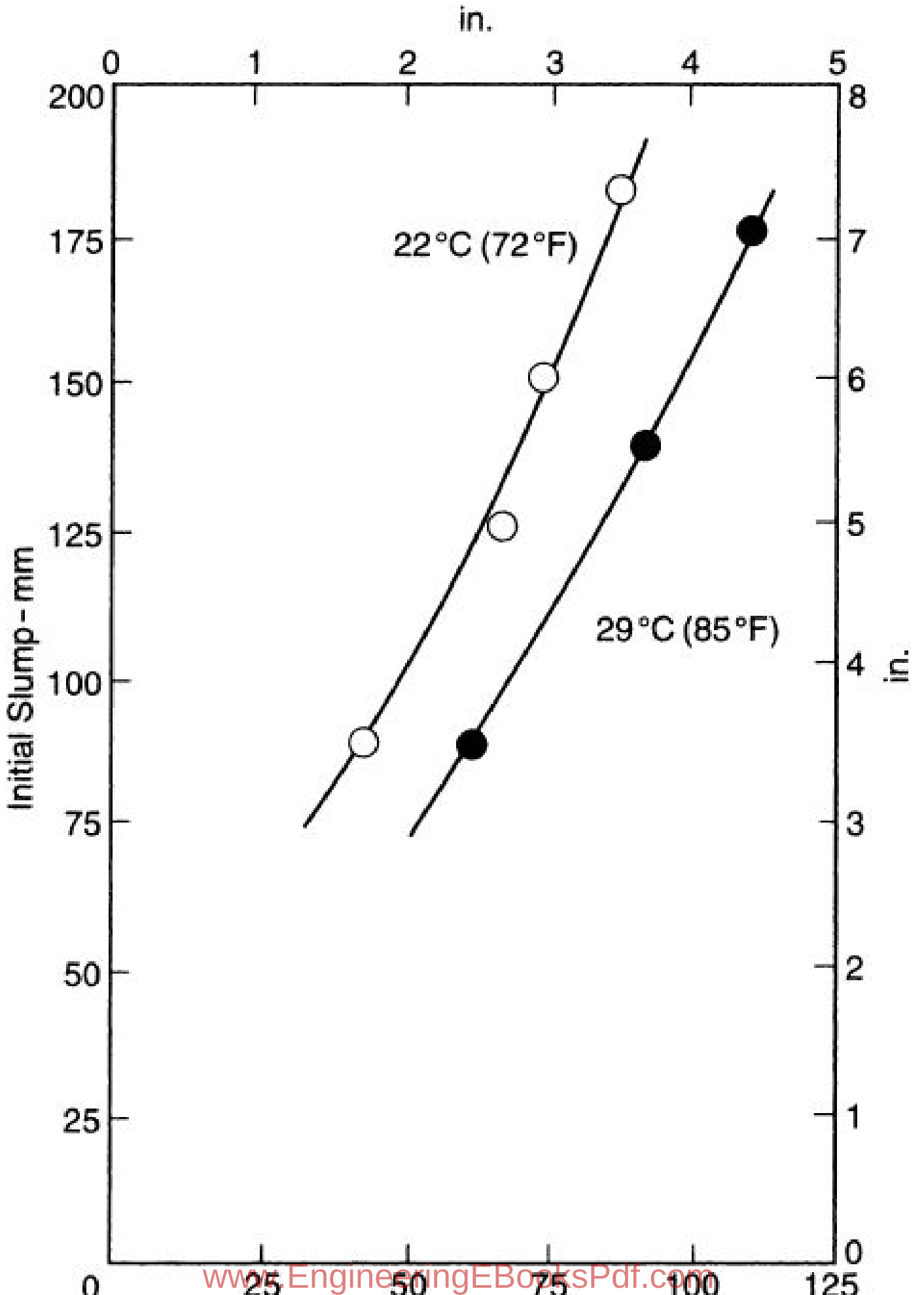


Fig. 4.14. Influence of temperature on the amount of water required to change slump[4.8]

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Fig. 4.15. Influence of temperature on loss of slump after 90 minutes for concrete with a cement content of 306 kg/m^3 (517 lb/yd^3) (based on ref. 4.61)

The effects of temperature on concrete are discussed in Chapter 8.

Because workability decreases with time, it is important to measure, say, slump after a predetermined time lapse since mixing. There is value in determining slump immediately after the discharge of the concrete from the mixer for the purpose of control of batching. There is also value in determining slump at the time of placing the concrete in the formwork for the purpose of ensuring that the workability is appropriate for the means of compaction to be used.

Segregation

In discussing workable concrete in general terms, it was stated that such concrete should not easily segregate, i.e. it ought to be cohesive. However,

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strictly speaking, the absence of a tendency to segregate is not included in the definition of a workable mix. Nevertheless, the absence of appreciable segregation is essential as full compaction of a segregated mix is impossible.

Segregation can be defined as separation of the constituents of a heterogeneous mixture so that their distribution is no longer uniform. In the case of concrete, it is the differences in the size of particles and in the specific gravity of the mix constituents that are the primary causes of segregation, but its extent can be controlled by the choice of suitable grading and by care in handling.

It is worth noting that a higher viscosity of the fresh cement paste component militates against the downward movement of the heavier aggregate particles; consequently, mixes with low water/cement ratios are less prone to segregation.[4.48]

There are two forms of segregation. In the first, the coarser particles tend to separate out because they tend to travel further along a slope

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or to settle more than finer particles. The second form of segregation, occurring particularly in wet mixes, is manifested by the separation of grout (cement plus water) from the mix. With some gradings, when a lean mix is used, the first type of segregation may occur if the mix is too dry; addition of water would improve the cohesion of the mix, but when the mix becomes too wet the second type of segregation would take place.

The influence of grading on segregation was discussed in detail in Chapter 3, but the actual extent of segregation depends on the method of handling and placing of concrete. If the concrete does not have far to travel and is transferred directly from the skip or bucket to the final position in the form, the danger of segregation is small. On the other hand, dropping concrete from a considerable height, passing along a chute, particularly with changes of direction, and discharging against an obstacle – all these encourage segregation so that under such circumstances a particularly cohesive mix should be used. With a correct

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method of handling, transporting and placing, the likelihood of segregation can be greatly reduced: there are many practical rules, which are presented in ACI 304R-85.[4.79]

It should be stressed, however, that concrete should always be placed direct in the position in which it is to remain and must not be allowed to flow or be worked along the form. This prohibition includes the use of a vibrator to spread a heap of concrete over a larger area. Vibration provides a most valuable means of compacting concrete but, because a large amount of work is being done on the concrete, the danger of segregation (in placing as distinct from handling) due to an improper use of a vibrator is increased. This is particularly so when vibration is allowed to continue too long: with many mixes, separation of coarse aggregate toward the bottom of the form and of the cement paste towards the top may result. Such concrete would obviously be weak, and the laitance (scum) on its surface would be too rich and too wet so that a crazed surface with a tendency

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to dusting might result. Laitance should be distinguished from bleed water, which is considered in the next section.

It may be noted that entrained air reduces the danger of segregation. On the other hand, the use of coarse aggregate whose specific gravity differs appreciably from that of the fine aggregate would lead to increased segregation.

Segregation is difficult to measure quantitatively, but is easily detected when concrete is handled on a site in any of the ways listed earlier as undesirable. A good picture of cohesion of the mix is obtained by the flow test. The jolting applied during the test encourages segregation, and if the mix is not cohesive the larger particles of aggregate will separate out and move toward the edge of the table. Another form of segregation is possible: in a sloppy mix the cement paste tends to run away from the centre of the table leaving the coarser material behind.

As far as proneness to segregation on over-vibration is concerned, a good test is to vibrate a

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concrete cylinder or cube for about 10 minutes and then to strip it and observe the distribution of coarse aggregate: any segregation will be easily seen.

Bleeding

Bleeding, known also as *water gain*, is a form of segregation in which some of the water in the mix tends to rise to the surface of freshly placed concrete. This is caused by the inability of the solid constituents of the mix to hold all of the mixing water when they settle downwards, water having the lowest specific gravity of all the mix constituents. We are thus dealing with subsidence, and Powers[4.10] treats bleeding as a special case of sedimentation. Bleeding can be expressed quantitatively as the total settlement per unit height of concrete or as a percentage of the mixing water; in extreme cases, this may reach 20 per cent.[4.112] ASTM C 232-09 prescribes two methods of determination of total bleeding. The rate of bleeding can also be determined experimentally.

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The initial bleeding proceeds at a constant rate, but subsequently the rate of bleeding decreases steadily. Bleeding of concrete continues until the cement paste has stiffened sufficiently to put an end to the process of sedimentation.

If the bleeding water is remixed during finishing of the top surface, a weak wearing surface, consisting of laitance, will be formed. This can be avoided by delaying the finishing operations until the bleed water has evaporated, and also by the use of wood floats and avoidance of overworking the surface. On the other hand, if evaporation of water from the surface of the concrete is faster than the bleeding rate, plastic shrinkage cracking may result (see p. 424).

Some of the rising water becomes trapped on the underside of coarse aggregate particles or of reinforcement, thus creating zones of poor bond. This water leaves behind air pockets or lenses, and because all the voids are oriented in the same direction, the permeability of the concrete in a horizontal plane may be increased. Hence, in-

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gress of an attacking medium into concrete is facilitated. A horizontal zone of weakness may also be created. The formation of such zones was confirmed by means of tensile tests in the direction of casting and at right angles to it.[4.65] Trapping an appreciable amount of bleed water must be avoided also because of the danger of frost damage, especially in road slabs.

Some bleeding is unavoidable. However, in high elements, such as columns or walls, as bleed water moves upwards, the water/cement ratio in the lower part of the element is reduced, but the water trapped in the upper part of the now stiffer concrete results in an increased water/cement ratio there, and hence in a reduced strength (see p. 272).

The bleed water can also travel upwards along the surface of the form; if a channel is formed due to some imperfection in the form surface, a preferred drainage path is created with resulting surface streaking. Vertical bleed channels can also form in the interior of the concrete.

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Bleeding need not necessarily be harmful. If the surface of the concrete is to be vacuum-dewatered (see p. 234) the removal of water is facilitated. If bleeding is undisturbed and the water evaporates, the effective water/cement ratio may be lowered with a resulting increase in strength. On the other hand, if the rising water carries with it a considerable amount of the finer cement particles, a layer of laitance will be formed. If this is at the top of a slab, a porous and weak surface layer will result, with a permanently 'dusty' surface. At the top of a lift, a plane of weakness would form and the bond with the next lift would be inadequate. For this reason, laitance should always be removed by brushing and washing.

The tendency to bleeding depends largely on the properties of cement. Bleeding is decreased by increasing the fineness of cement, possibly because finer particles hydrate earlier and also because their rate of sedimentation is lower. Other properties of cement also affect bleeding: there is less bleeding when the cement has a high alkali

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content, a high C_3A content, or when calcium chloride is added;[4.11] for limitations on the use of calcium chloride see p. 566. The test methods for bleeding of cement pastes and mortar were prescribed by ASTM C 243-85 (withdrawn).

The properties of cement, however, are not the sole factor influencing the bleeding of concrete[4.120] so that other factors must also be considered. Specifically, the presence of an adequate proportion of very fine aggregate particles (especially smaller than 150 μm (No. 100 sieve)) significantly reduces bleeding.[4.12] The use of crushed fine aggregate does not necessarily lead to more bleeding than rounded sand. In fact, when the crushed fine aggregate contains excess very fine material (up to about 15 per cent passing the 150 μm (No. 100) sieve), bleeding is reduced,[4.37] but the very fine material must consist of crusher dust only, and not of clay.

Rich mixes are less prone to bleeding than lean ones. Reduction in bleeding is obtained by

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the addition of pozzolanas or other fine material or aluminium powder. Schiessl and Schmidt[4.66] found that addition to mortar of fly ash or silica fume significantly decreased bleeding. This may not necessarily be so in the case of concrete, much depending on the basis of comparison, e.g. whether the cementitious materials are additional to Portland cement or whether they replace some of it. Air entrainment effectively reduces bleeding so that finishing can follow casting without delay.

A higher temperature, within the normal range, increases the rate of bleeding, but the total bleeding capacity is probably unaffected. Very low temperature, however, may increase the bleeding capacity, probably because there is more time prior to stiffening for bleeding to occur.[4.68]

The influence of admixtures is not straightforward. Superplasticizers generally decrease bleeding except at a very high slump.[4.67] However, if they are used with a retarder, increased bleed-

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ing may occur,[4.68] possibly because retardation allows more time for bleeding to occur. If, at the same time, air entrainment is used, its effect in reducing bleeding may be dominant.

The mixing of concrete

It is essential that the mix ingredients, whose properties were discussed in Chapters 1 to 3, are properly mixed so as to produce fresh concrete in which the surface of all aggregate particles is coated with cement paste and which is homogeneous on the macro-scale and therefore possessing uniform properties. Almost invariably, mixing is effected by mechanical mixers.

Concrete mixers

Concrete mixers must not only achieve the uniformity of the mix, just referred to, but they must also discharge the mix without disturbing that uniformity. In fact, the method of discharging is one of the bases of classification of concrete mixers. Several types exist. In the *tilting mixer*, the

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mixing chamber, known as the drum, is tilted for discharging. In the *non-tilting mixer*, the axis of the mixer is always horizontal, and discharge is obtained either by inserting a chute into the drum or by reversing the direction of rotation of the drum (when the mixer is known as a *reversing drum mixer*), or rarely by splitting of the drum. There are also *pan-type mixers*, rather similar in operation to an electric cake-mixer; these are called *forced action mixers*, as distinct from the tilting and non-tilting mixers which rely on the free fall of concrete in the drum.

Tilting mixers usually have a conical or bowl-shaped drum with vanes inside. The efficiency of the mixing operation depends on the details of design, but the discharge action is always good as all the concrete can be tipped out rapidly and in an unsegregated mass as soon as the drum is tilted. For this reason, tilting-drum mixers are preferable for mixes of low workability and for those containing large-size aggregate.

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On the other hand, because of a rather slow rate of discharge from a non-tilting drum mixer, concrete is sometimes susceptible to segregation. In particular, the largest size of aggregate may tend to stay in the mixer so that the discharge sometimes starts as mortar and ends as a collection of coated coarse aggregate particles. Non-tilting mixers are less frequently used than in the past.

Non-tilting mixers are always charged by means of a loading skip, which is also used with the larger tilting drum mixers. It is important that the whole charge from the skip be transferred into the mixer every time, i.e. no sticking must occur. Sometimes, a shaker mounted on the skip assists in emptying it.

The pan mixer is generally not mobile and is therefore used at a central mixing plant, at a precast concrete plant, or in a small version in the concrete laboratory. The mixer consists essentially of a circular pan rotating about its axis, with one or two stars of paddles rotating about a

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vertical axis not coincident with the axis of the pan. Sometimes, the pan is static and the axis of the star travels along a circular path about the axis of the pan. In either case, the relative movement between the paddles and the concrete is the same, and concrete in every part of the pan is thoroughly mixed. Scraper blades prevent mortar sticking to the sides of the pan, and the height of the paddles can be adjusted so as to prevent a permanent coating of mortar forming on the bottom of the pan.

Pan mixers offer the possibility of observing the concrete in them, and therefore of adjusting the mix in some cases. They are particularly efficient with stiff and cohesive mixes and are, therefore, often used in the manufacture of precast concrete. They are also suitable, because of the scraping arrangements, for mixing very small quantities of concrete – hence their use in the laboratory.

It may be relevant to mention that, in drum-type mixers, no scraping of the sides takes place

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during mixing so that a certain amount of mortar adheres to the sides of the drum and stays there until the mixer has been cleaned. It follows that, at the beginning of concreting, the first mix would leave a large proportion of its mortar behind, and the discharge would consist largely of coated coarse particles. This initial batch should not be routinely used. As an alternative, a certain amount of mortar may be introduced into the mixer prior to the commencement of concreting, a procedure known as 'buttering' or priming the mixer. A convenient and simple way is to charge the mixer with the usual quantities of cement, water and fine aggregate, simply omitting the coarse aggregate. The mix in excess of that stuck in the mixer can be used in construction and may in fact be particularly suitable for placing at a cold joint. The necessity of buttering should not be forgotten in laboratory work.

The nominal size of a mixer is described by the volume of concrete after compaction (BS 1305 : 1974 (obsolescent)), which may be as low

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as one-half of the volume of the unmixed ingredients in a loose state. Mixers are made in a variety of sizes from 0.04 m^3 ($1\frac{1}{2}$ ft^3) for laboratory use up to 13 m^3 (17 yd^3). If the quantity mixed represents less than one-third of the *nominal* capacity of the mixer, the resulting mix may not be uniform, and the operation would, of course, be uneconomical. Overload not exceeding 10 per cent is generally harmless.

All the mixers considered so far are batch mixers, in that one batch of concrete is mixed and discharged before any more materials are added. As opposed to this, a *continuous mixer* discharges mixed concrete steadily without interruption, being fed by a continuous volume- or weigh-batching system. The mixer itself consists of a spiral blade rotated at a relatively high speed in an enclosed, slightly inclined trough. ASTM C 685-10 prescribes the requirements for concrete made by volumetric batching and continuous mixing, and ACI 304.6R-91[4.113] offers a guide for the use of the relevant equipment. Modern continuous

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mixers produce concrete of high uniformity.[4.113] Using a continuous-feed mixer, placing, compaction and finishing can all be achieved within 15 minutes of the introduction of water into the mix.[4.101] Volume-batched mixers are used also with recycled concrete aggregate.[4.123]

Other mixers should be briefly mentioned. These include revolving-drum truck mixers, reference to which is made on p. 217. There have also been developed twin-fin truck mixers with water nozzles distributed within the drum, but no adequate data on their performance are available.

Specialized mixers are used in shotcreting and for mortar for preplaced aggregate concrete. In the 'colloid' mixer used for the latter, cement and water are formed into colloidal grout by passage, at a speed of 2000 rev/min, through a narrow gap, and sand is subsequently added to the grout. The pre-mixing of cement and water allows better subsequent hydration and, when used for concrete, leads to a higher strength at a given water/cement ratio than conventional mixing. For

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instance, at water/cement ratios of 0.45 to 0.50, a gain in strength of 10 per cent has been observed.[4.26] However, a large amount of heat is generated at very low water/cement ratios.[4.64] Moreover, two-stage mixing undoubtedly represents a higher cost and is likely to be justifiable only in special cases.

Uniformity of mixing

In any mixer, it is essential that sufficient interchange of materials between different parts of the chamber takes place, so that uniform concrete is produced. The efficiency of the mixer can be measured by the variability of the mix discharged into a number of receptacles without interrupting the flow of concrete. For instance, a rather rigid test of ASTM C 94-09a (formally applicable only to truck mixers) lays down that samples of concrete should be taken from about $\frac{1}{6}$ to $\frac{5}{6}$ points of a batch, and the differences in the properties of the two samples should not exceed any of the following:

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Density of concrete	16 kg/m^3 (1 lb/ft^3)
Air content	1 per cent
Slump	25 mm (1 in.) when the average is under 100 mm (4 in.), and 40 mm (1.5 in.) when the average is 100 to 150 mm (4 to 6 in.)
Percentage of aggregate retained on a 4.75 mm ($\frac{3}{16}$ in.) sieve	6 per cent
Density of air-free mortar	1.6 per cent
Compressive strength (average 7-day strength of three cylinders)	7.5 per cent.

In the United Kingdom, BS 3963 : 1974 (1980) gives a guide to the assessment of performance of mixers using a specified concrete mix. Tests are made on two samples from each quarter of a batch. Each sample is subjected to wet analysis and the following are determined:

water content as percentage of solids to 0.1 per cent

fine aggregate content as percentage of total aggregate to 0.5 per cent

cement as percentage of total aggregate to 0.01 per cent

water/cement ratio to 0.01.

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The sampling accuracy is assured by a limit on the average range of pairs. If two samplers in a pair differ unduly, i.e. their range is an outlier[*], that pair of results can be discarded.

[*]See, for instance, J. B. Kennedy and A. M. Neville, *Basic Statistical Methods for Engineers and Scientists*, 3rd Edn., 613 pp. (New York and London, Harper and Row, 1986).

The mixer performance is judged by the average value of the difference between the highest and the lowest average of pairs of readings for the four samples in each of three test batches; thus one bad mixing operation does not condemn a mixer. The maximum acceptable variabilities of the percentages listed earlier are prescribed by an obsolescent British Standard BS 1305 : 1974 for different maximum aggregate sizes.

Swedish investigations[4.115] have shown that the uniformity of the cement content is the best measure of uniformity of mixing: this is con-

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sidered to be satisfactory if the coefficient of variation (see p. 642) does not exceed 6 per cent for mixes with a slump of at least 20 mm, and 8 per cent for mixes of lower workability.

A method of the determination of the distribution of water or admixture in the mix by a radioactive tracer has been developed in France.[4.116]

As far as volume-batched continuous mixers are concerned, the uniformity of mixing has to be measured by tolerances on the proportions of the mix ingredients. ASTM C 685-10 prescribes the following percentage values by mass:

Cement	0 to +4
Water	±1
Fine aggregate	±2
Coarse aggregate	±2.

The US Army Corps of Engineers Test Method CRD-C 55-92[4.117] specifies taking samples from each one-third of a stationary mixer. For mass concrete, the conformity requirements are

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given in the Corps of Engineers Guide Specification 03305; these are similar to those of ASTM C 94-09a, but the allowable range of density is 32 kg/m^3 (2 lb/ft^3) and, for compressive strength, 10 per cent. These seemingly higher values are a reflection of the fact that three samples are used, rather than two as in the test of ASTM C 94-09a.

It can be added that tests on the uniformity of mixing measure not only the performance of a mixer, but can also be used to assess the effects of a sequence of charging the mixer.

Mixing time

On a site, there is often a tendency to mix concrete as rapidly as possible, and it is, therefore, important to know what is the minimum mixing time necessary to produce a concrete uniform in composition and, as a result, of satisfactory strength. This time varies with the type of mixer and, strictly speaking, it is not the mixing time but the number of revolutions of the mixer that is the criterion of adequate mixing. Generally, about

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20 revolutions are sufficient. Because there is an optimum speed of rotation recommended by the manufacturer of the mixer, the number of revolutions and the time of mixing are interdependent.

For a given mixer, there exists a relation between mixing time and uniformity of the mix. Typical data are shown in Fig. 4.16, based on tests by Shalon and Reinitz,[4.22] the variability being represented as the range of strengths of specimens made from the given mix after a specified mixing time. Figure 4.17 shows the results of the same tests plotted as the coefficient of variation against mixing time. It is apparent that mixing for less than 1 to $1\frac{1}{4}$ minutes produces an appreciably more variable concrete, but prolonging the mixing time beyond these values results in no significant improvement in uniformity.

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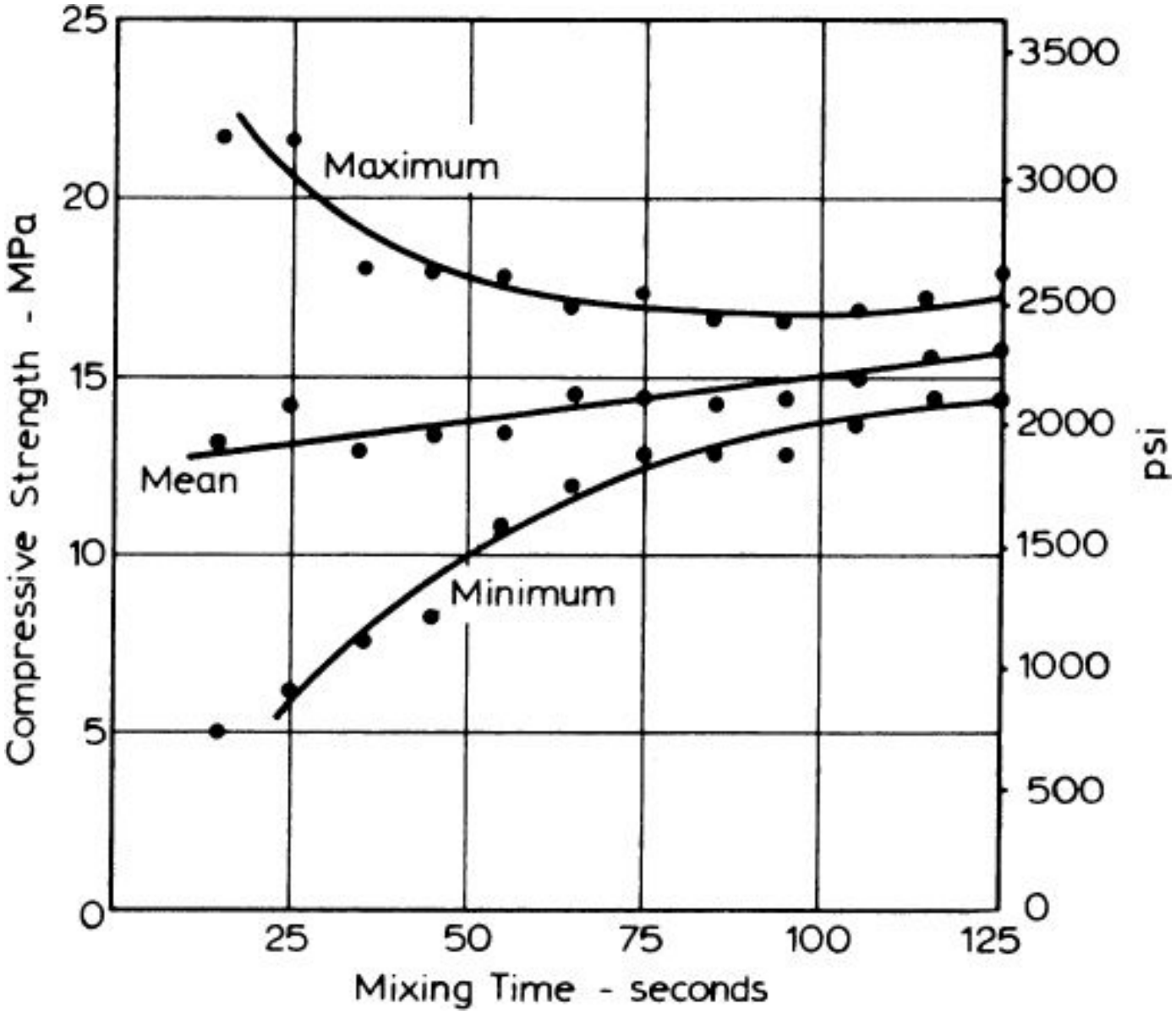


Fig. 4.16. Relation between compressive strength and mixing time[4.22]

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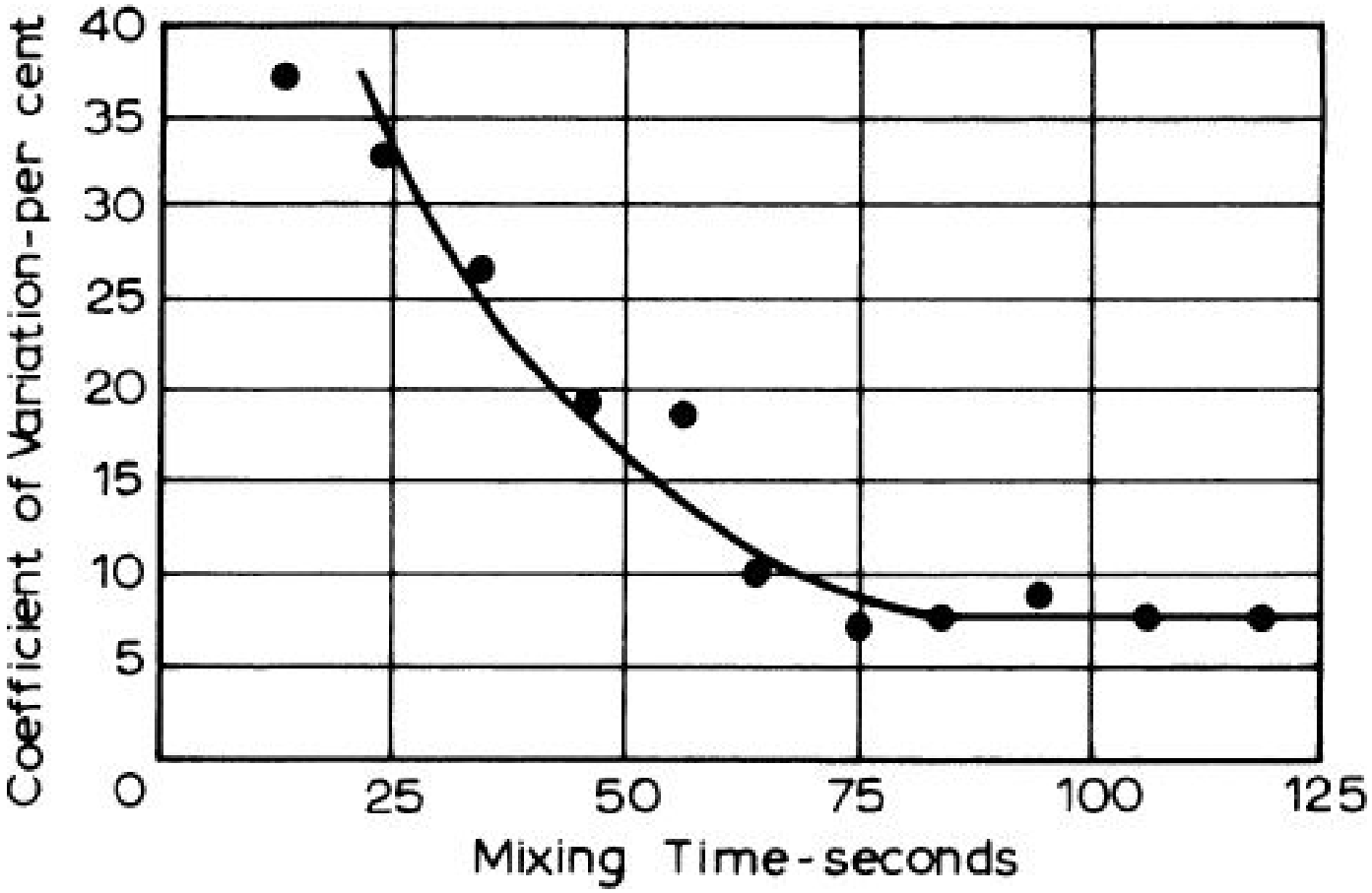


Fig. 4.17. Relation between the coefficient of variation of strength and mixing time[4.22]

The average strength of concrete also increases with an increase in mixing time, as shown for instance by Abrams' tests.[4.23] The rate of increase falls rapidly beyond about one minute and is not significant beyond two minutes; sometimes, even a slight decrease in strength has been observed.[4.44] Within the first minute, however,

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the influence of mixing time on strength is of considerable importance.[4.22]

As mentioned before, the exact value of the minimum mixing time, which is given by the mixer manufacturer, varies with the type of mixer and depends also on its size. What is essential is to ensure uniformity of mixing, which generally can be achieved by a minimum mixing time of 1 minute for a mixer size of 1 yd^3 ($\frac{3}{4}$ m^3) and 15 additional seconds for each additional cubic yard ($\frac{3}{4}$ m^3). This guidance is given both by ASTM C 94-09a and by ACI 304R-89.[4.76] According to ASTM C 94-09a, the mixing time is reckoned from the time when all the solid materials have been put in the mixer, and it is also required that all the water has to be added not later than after one-quarter of the mixing time. ACI 304R-89 reckons the mixing time from the time when all the ingredients have been discharged into the mixer.

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The figures quoted refer to the usual mixers but there are many modern large mixers which perform satisfactorily with a mixing time of 1 to $1\frac{1}{2}$ minutes. In high-speed pan mixers, the mixing time can be as short as 35 seconds. On the other hand, when lightweight aggregate is used, the mixing time should be not less than 5 minutes, sometimes divided into 2 minutes of mixing the aggregate with water, followed by 3 minutes with cement added. In general, the length of mixing time required for sufficient uniformity of the mix depends on the quality of blending of materials during charging of the mixer: simultaneous feed is beneficial.

Let us consider now the other extreme – mixing over a long period. Generally, evaporation of water from the mix takes place, with a consequent decrease in workability and increase in strength. A secondary effect is that of grinding of the aggregate, particularly if soft: the grading of the aggregate thus becomes finer, and the work-

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ability lower. The friction effect also produces an increase in the temperature of the mix.

In the case of air-entrained concrete, prolonged mixing reduces the air content by about $\frac{1}{6}$ per hour (depending on the type of air-entraining agent), while a delay in placing without continuous mixing causes a drop in air content by only about $\frac{1}{10}$ per hour. On the other hand, a decrease in mixing time below 2 or 3 minutes may lead to inadequate entrainment of air.

Intermittent remixing up to about 3 hours, and in some cases up to 6 hours, is harmless as far as strength and durability are concerned, but the workability falls off with time unless loss of moisture from the mixer is prevented. Adding water to restore workability, known as retempering, will lower the strength of the concrete. This is considered on p. 218.

No general rules on the order of feeding the ingredients into the mixer can be given as they depend on the properties of the mix and of the

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mixer. Generally, a small amount of water should be fed first, followed by all the solid materials, preferably fed uniformly and simultaneously into the mixer. If possible, the greater part of the water should also be fed during the same time, the remainder of the water being added after the solids. With some drum mixers, however, when a very dry mix is used, it is necessary to feed first some water and the coarse aggregate, as otherwise its surface does not become sufficiently wetted. Moreover, if coarse aggregate is totally absent to begin with, sand or sand and cement become lodged in the head of the mixer and do not become incorporated in the mix; this is known as head pack. If water or cement are fed too fast, or are too hot, there is a danger of formation of cement balls, sometimes up to 70 mm (or 3 in.) in diameter. With small laboratory pan mixers and very stiff mixes, it has been found convenient to feed first the fine aggregate, a part of the coarse aggregate and cement, then the water, and finally the remainder of the coarse aggregate so as to break up any nodules of mortar.

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Tests on flowing concrete made with a superplasticizer,[4.118] have shown the slump to be highest when cement and fine aggregate are mixed together first, and to be lowest when cement and water are mixed together first. Mixing all the ingredients simultaneously resulted in an intermediate slump. Figure 4.18 shows this situation and shows also that the rate of slump loss was highest when cement and fine aggregate were mixed together first. The slump loss was lowest when all the materials were mixed simultaneously. It seems thus that, to minimize slump loss, the conventional mixing technique is the most beneficial.

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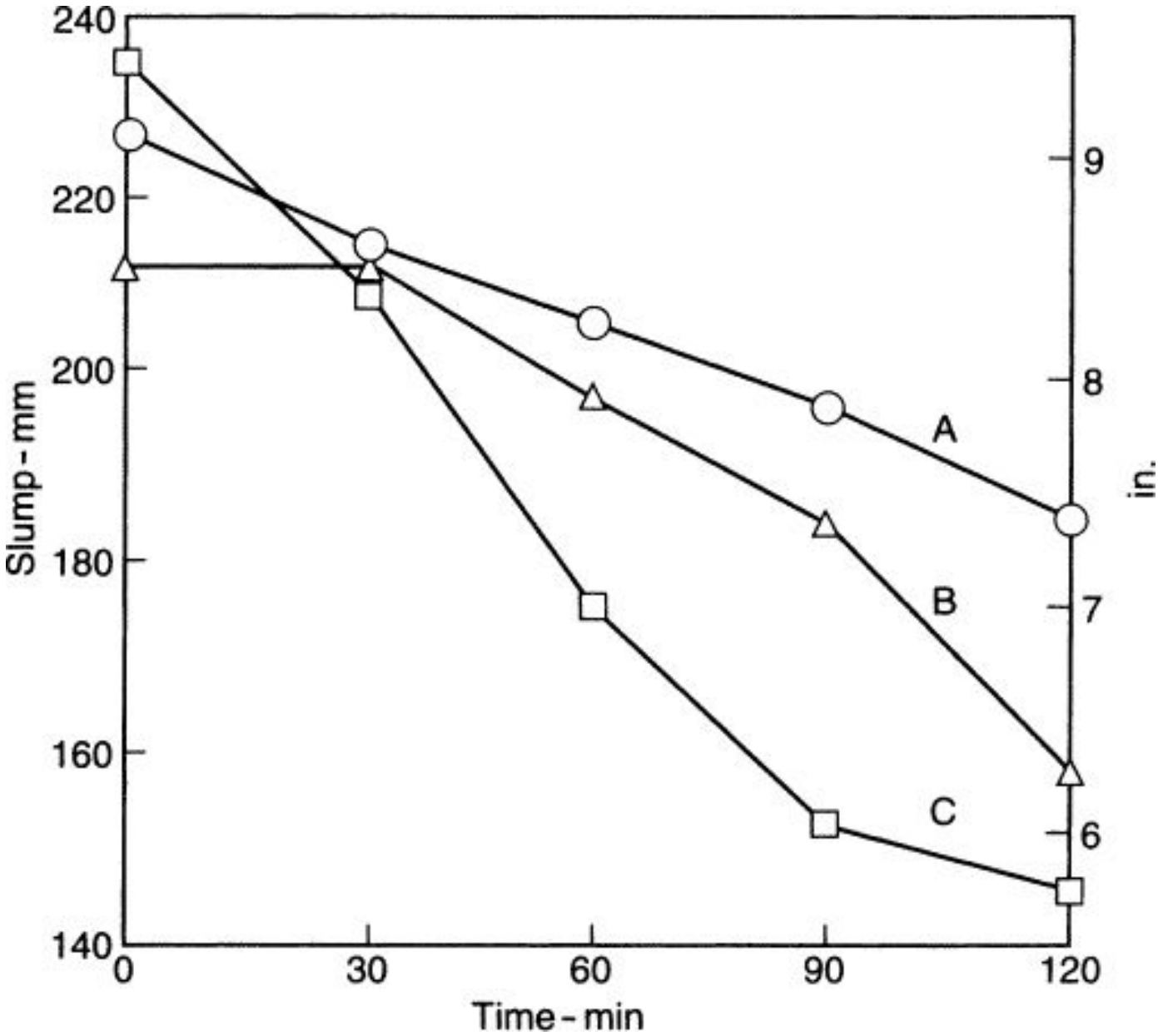


Fig. 4.18. Loss of slump with time for concretes with a water/cement ratio of 0.25 and a superplasticizer for different batching sequences: (*A*) all ingredients simultaneously; (*B*) cement and water first; (*C*) cement and fine aggregate first (based on ref. 4.118)

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In connection with mixing flowing concrete, it is worth noting that visual judgement of consistency of the mix by the mixer operator is not possible because the mix simply looks fluid.

Hand mixing

There may be rare occasions when small quantities of concrete have to be mixed by hand and, because in this case uniformity is more difficult to achieve, particular care and effort are necessary. In order to make sure that the relevant art be not forgotten, an appropriate procedure will be described.

The aggregate should be spread in a uniform layer on a hard, clean and non-porous base; cement is then spread over the aggregate, and the dry materials are mixed by turning over from one end of the tray to the other and ‘cutting’ with a shovel until the mix appears uniform. Turning three times is usually required. Water is then gradually added so that neither water by itself nor with cement can escape. The mix is turned over

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again, usually three times, until it appears uniform in colour and consistency.

It is obvious that during hand mixing no soil or other extraneous material must be allowed to become included in the concrete.

Ready-mixed concrete

Ready-mixed concrete used to be treated as a separate topic but, nowadays, with the vast majority of concrete in many countries originating from a central plant, only certain special features of ready-mixed concrete will be considered in this section.

Ready-mixed concrete is particularly useful on congested sites or in road construction where little space for a mixing plant and for extensive aggregate stockpiles is available, but perhaps the greatest single advantage of ready-mixed concrete is that it is made under better conditions of control than are normally possible on any but large construction sites. Control has to be enforced but, since the central mixing plant operates

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under near-factory conditions, a really close control of all operations of production of fresh concrete is possible. Proper care during transportation of the concrete is also ensured by the use of agitator trucks, but the placing and compaction remain, of course, the responsibility of the personnel on the site. The use of ready-mixed concrete is also advantageous when only small quantities of concrete are required or when concrete is placed only at intervals.

There are two principal categories of ready-mixed concrete. In the first, the mixing is done at a central plant and the mixed concrete is then transported, usually in an agitator truck which revolves slowly so as to prevent segregation and undue stiffening of the mix. Such concrete is known as *central-mixed* as distinct from the second category – *transit-mixed* or *truck-mixed* concrete. Here, the materials are batched at a central plant but are mixed in a mixer truck either in transit to the site or immediately prior to the concrete being discharged. Transit-mixing per-

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mits a longer haul and is less vulnerable in case of delay, but the capacity of a truck used as a mixer is only 63 per cent, or even less, of the drum while for central-mixed concrete it is 80 per cent. Sometimes, the concrete is partially mixed at a central plant in order to increase the capacity of the agitator truck. The mixing is completed *en route*. Such concrete is known as *shrink-mixed* concrete but is rarely used. Truck mixers usually have a capacity of 6 m^3 (8 yd^3) or 7.5 m^3 (10 yd^3).

It should be explained that agitating differs from mixing solely by the speed of rotation of the mixer: the agitating speed is between 2 and 6 rev/min, compared with the mixing speed of 4 to about 16 rev/min; there is thus some overlap in the definitions. It may be noted that the speed of mixing affects the rate of stiffening, while the total number of revolutions controls the uniformity of mixing. Unless the concrete has been shrink-mixed in the central plant mixer, 70 to 100 revolutions at mixing speed in the truck mixer are required. An overriding limit of 300 revolutions

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in toto is laid down by ASTM C 94-09a. This is thought to be unnecessary[4.78] unless the aggregate, especially the fine fraction, is soft and liable to grinding.

If the final part of water is put into the mixer just prior to delivery of the concrete (as may be desirable in hot weather), ASTM C 94-09a requires 30 additional revolutions at mixing speed prior to discharge.

The main problem in the production of ready-mixed concrete is maintaining the workability of the mix right up to the time of placing. Concrete stiffens with time and the stiffening may also be aggravated by prolonged mixing and by a high temperature. In the case of transit-mixing, water need not be added till nearer the commencement of mixing but, according to ASTM C 94-09a, the time during which the cement and moist aggregate are allowed to remain in contact is limited to 90 minutes; BS 5328 : 3 : 1990 superseded by BS EN 206-1 : 2000 allows 2 hours. The 90-minute limit can be relaxed by the purchaser of the con-

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crete; there is evidence[4.83] that, with the use of retarders, the time limit can be extended to 3 or even 4 hours, provided the concrete temperature at delivery is below 32 °C (90 °F).

The United States Bureau of Reclamation provides for an extension of 2 to 6 hours in the time of contact between cement and wet aggregate in transport prior to mixing. This requires 5 per cent of additional cement for every hour between these limits; thus between 5 and 20 per cent additional cement can be required.[4.97]

Retempering

The loss of slump with time was discussed on p. 205. There are two reasons for this phenomenon. First, from the instant that cement powder and water come into contact with one another, chemical reactions of hydration of cement take place. As these reactions involve fixing of water, less water is left to 'lubricate' the movement of individual particles in the mix. Second, in most ambient conditions, some of the mix water evaporates

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into the atmosphere and does so the more rapidly the higher the temperature and the lower the ambient relative humidity.

We can see, therefore, that, if a specified workability is required at the point of delivery of the concrete after a certain passage of time, this has to be ensured by the use of appropriate mix proportions and transport arrangements. Occasionally, however, delays occur in transport or other mishaps prevent a timely discharge of the concrete. If, in the meantime, a loss of slump occurs, the question arises as to whether the slump can be restored by means of addition of water coupled with remixing. Such an operation is referred to as retempering.

As retempering increases the original water/cement ratio of the mix, it is arguable that it should not be permitted where the original water/cement ratio was directly or indirectly specified. This is an appropriate stance under some circumstances but, at other times, a more flexible and sensible solution may be appropriate as long as

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the consequences of retempering are understood and appreciated.

The starting point to be considered is the overall water/cement ratio on the basis both of the original mix water and the retempering water. There is considerable evidence[4.24,4.45] that not all the retempering water should be counted as part of the free water for the purpose of calculating the water/cement ratio. The reason for this behaviour probably lies in the fact that water replacing that lost by evaporation should not be included in the effective water/cement ratio; only the water replacing that used in early hydration constitutes part of the effective mix water.

It follows from the above that the relation between strength and the overall free water/cement ratio for retempered concrete is slightly more advantageous than the usual ratio between strength and the free water/cement ratio; an example of two such relations was obtained by Hanayneh and Itani.[4.90]

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Nevertheless, retempering inevitably results in some loss of strength compared with the original concrete. A loss of 7 to 10 per cent was reported,[4.90] but it can be much higher depending on the amount of retempering water added to the mix[4.28] (see Fig. 4.19). Some empirical relationships have been suggested[4.88] but, in practice, the precise amount of retempering water may not be known, if only because partial discharge from the mixer had occurred prior to the realization of the slump loss.

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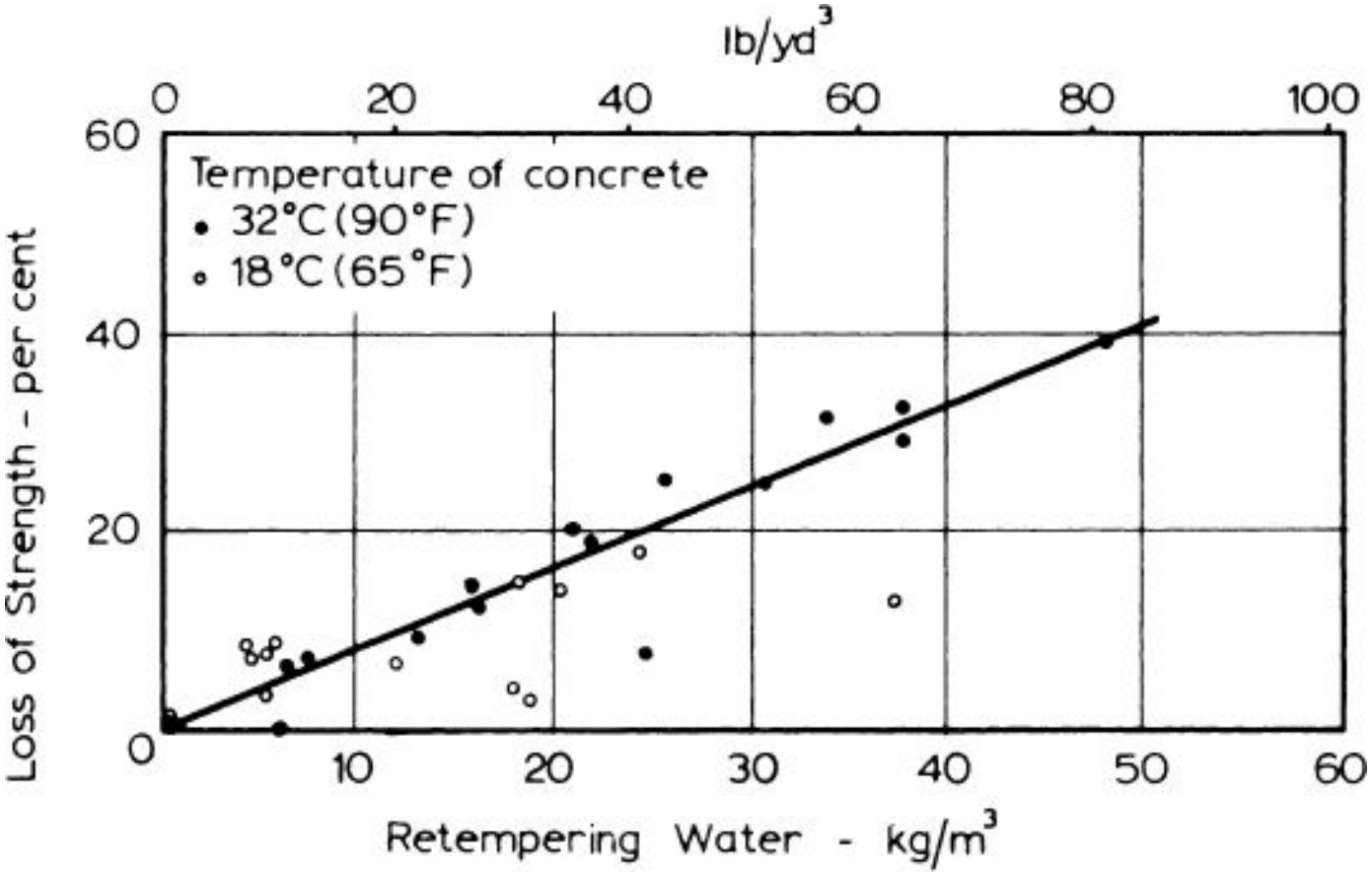


Fig. 4.19. Effect of retempering water on the strength of concrete[4.28]

The amount of water needed to raise the slump by 75 mm (3 in.) depends on the original slump level, being higher at low slumps; Burg[4.89] reported the following (in litres per cubic metre of concrete):

22 to 32 at a slump of less than 75 mm
14 to 18 at a slump of 75 to 125 mm, and
4 to 9 at a slump of 125 to 150 mm.

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Another way of viewing the preceding data is to say that the lower the water/cement ratio the more retempering water is needed. The amount of water also rises steeply with an increase in temperature so that at 50 °C (125 °F) it can be about double that at 30 °C (86 °F).[4.121]

Pumped concrete

Since this book deals primarily with the properties of concrete, the details of the means of transporting and placing are not considered; they are dealt with, for example, in ACI Guide 304R-89.[4.76] However, an exception should be made in the case of pumping of concrete because this means of transportation requires the use of mixes having special properties.

Concrete pumps

The pumping system consists essentially of a hopper into which concrete is discharged from the mixer, a concrete pump of the type shown in

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Fig. 4.20 or 4.21, and pipes through which the concrete is pumped.

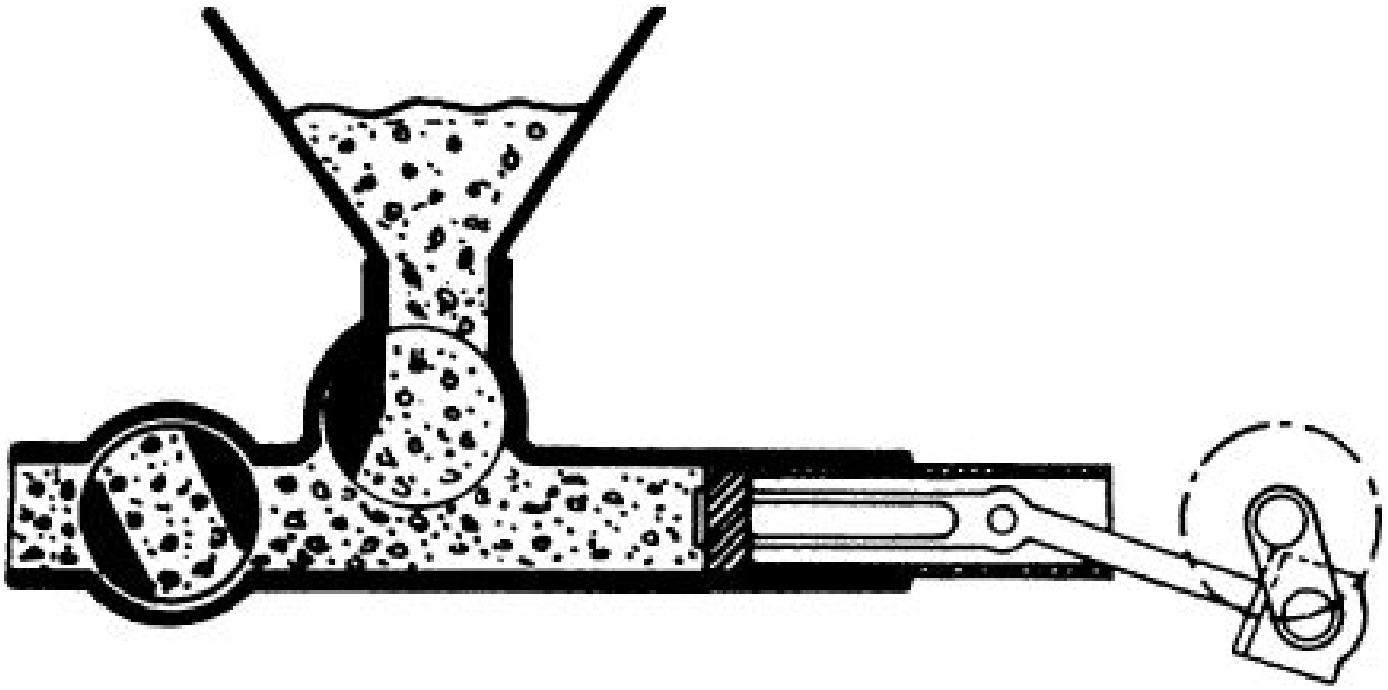


Fig. 4.20. Direct-acting concrete pump

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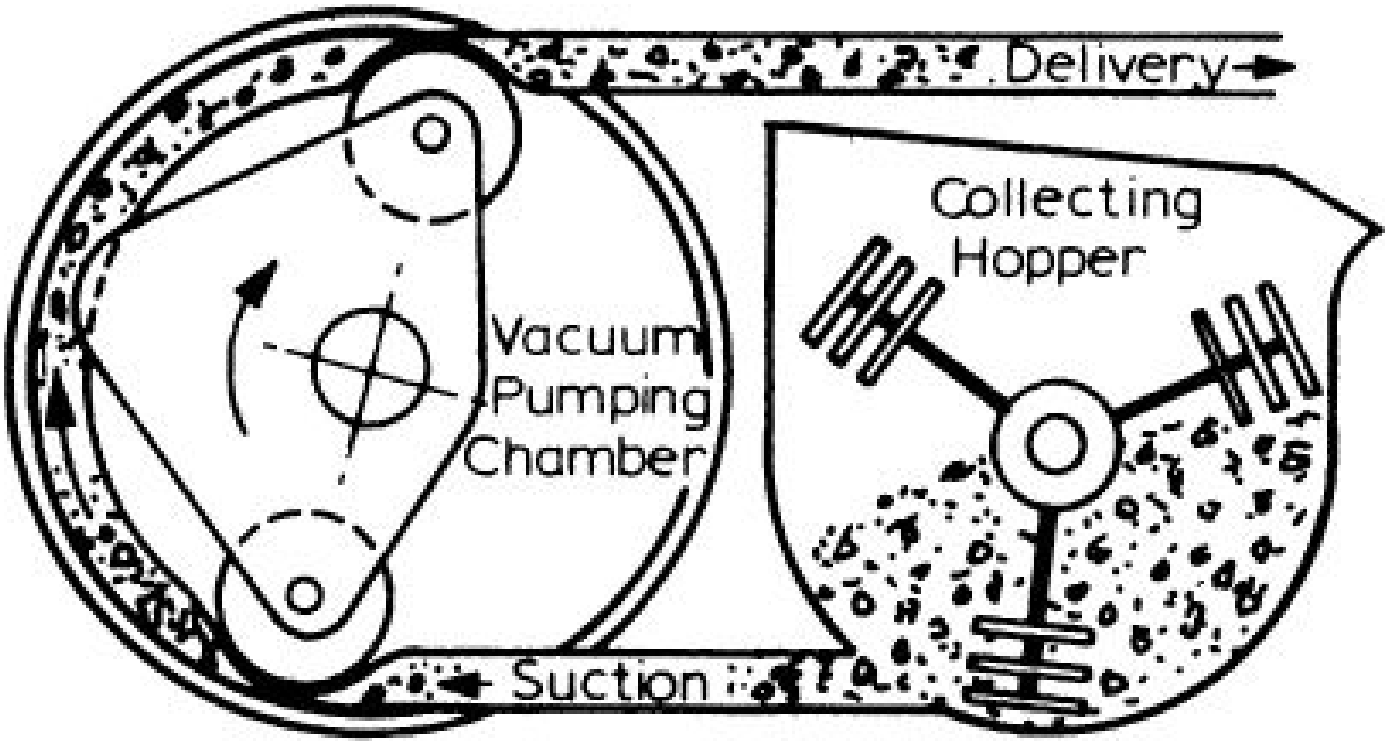


Fig. 4.21. Squeeze-type concrete pump

Many pumps are of the direct-acting, horizontal piston type with semi-rotary valves set so as to permit always the passage of the largest particles of aggregate being used: there is thus no full closure. Concrete is fed into the pump by gravity and is also partially sucked in during the suction stroke. The valves open and close with definite pauses so that concrete moves in a series of impulses but the pipe always remains full. Modern piston pumps are highly effective.

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There exist also portable *peristaltic* pumps, called squeeze pumps, for use with small diameter (up to 75 or 100 mm (3 or 4 in.)) pipes; Fig. 4.21 shows such a pump. Concrete placed in a collecting hopper is fed by rotating blades into a pliable pipe located in a pumping chamber under vacuum. This ensures that, except when actually squeezed by a roller, the pipe has a normal (cylindrical) shape so that a continuous flow of concrete is ensured. Two rotating rollers progressively squeeze the tube and thus pump the concrete in the suction pipe towards the delivery pipe.

Squeeze pumps move concrete for distances up to 90 m (300 ft) horizontally or 30 m (100 ft) vertically. However, using piston pumps, concrete can be moved up to about 1000 m (3300 ft) horizontally or 120 m (400 ft) vertically, or to proportionate combinations of distance and lift. We should note that the ratio of equivalent horizontal and vertical distances varies with the consistency of the mix and with the velocity of the

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concrete in the pipe: the greater the velocity the smaller the ratio;[4.29] at 0.1 m/s it is 24, but at 0.7 m/s it is only 4.5. Special pumps operating at high pressures can pump concrete up to 1400 m (4600 ft) horizontally or 430 m (1430 ft) vertically.[4.114] New record values, recently 600 m, continue to be reported.

When bends are used, and these should be kept to a minimum and must never be sharp, the loss of head should be allowed for in the calculation of the range of delivery: roughly, each 10° bend is equivalent to a length of pipe up to 1 m.

Pumps of different sizes are available and likewise pipes of various diameters are used, but the pipe diameter must be at least three times the maximum aggregate size. It is important to note that oversize in coarse aggregate should not be permitted so as to avoid blockage at bends.

Using squeeze pumps, an output of 20 m^3 (25 yd^3) of concrete per hour can be obtained with 75 mm (3 in.) pipes, but piston pumps with 200 mm

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(8 in.) pipes can deliver up to 130 m^3 (170 yd^3) per hour.

Pumps can be truck- or trailer-mounted and can deliver concrete through a folding boom. In Japan, a horizontal concrete distributor which automatically controls the position of the pipe is sometimes used;[4.87] this reduces the hard work of controlling the pipe end during discharge.

Use of pumping

Pumping is economical if it can be used without interruption because, at the beginning of each period of pumping, the pipes have to be lubricated by mortar (at the rate of about 0.25 m^3 per 100 m (1 yd^3 per 1000 ft) of 150 mm (6 in.) diameter pipe) and also because at the end of the operation a considerable effort is required in cleaning the pipes. However, alterations to the pipeline system can be made very quickly as special couplings are used. A short length of flexible hose near the discharge end facilitates placing but increases the friction loss. Aluminium pipes must

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not be used because aluminium reacts with the alkalis in cement and generates hydrogen. This gas introduces voids in the hardened concrete with a consequent loss of strength, unless the concrete is placed in a confined space.

The main advantages of pumping concrete are that it can be delivered to points over a wide area otherwise not easily accessible, with the mixing plant clear of the site; this is especially valuable on congested sites or in special applications such as tunnel linings, etc. Pumping delivers the concrete direct from the mixer to the form and so avoids double handling. Placing can proceed at the rate of the output of the mixer, or of several mixers, and is not held back by the limitations of the transporting and placing equipment. A high proportion of ready-mixed concrete is nowadays pumped.

Furthermore, pumped concrete is unsegregated but of course in order to be able to be pumped the mix must satisfy certain requirements. It might be added that unsatisfactory con-

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crete cannot be pumped so that any pumped concrete is satisfactory as far as its properties in the fresh state are concerned. Control of the mix is afforded by the force required to stir it in the hopper and by the pressure required to pump it.

Requirements for pumped concrete

Concrete which is to be pumped must be well mixed before feeding into the pump, and sometimes remixing in the hopper by means of a stirrer is carried out. Broadly speaking, the mix must not be harsh or sticky, too dry or too wet, i.e. its consistency is critical. A slump of between 50 and 150 mm (2 and 6 in.) is generally recommended, but pumping produces a partial compaction so that at the point of delivery the slump may be decreased by 10 to 25 mm ($\frac{1}{2}$ to 1 in.). With a lower water content, the coarse particles, instead of moving longitudinally in a coherent mass in suspension, would exert pressure on the walls of the pipe. When the water content is at the correct, or critical, value, friction develops only at

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the surface of the pipe and in a thin, 1 to 2.5 mm (0.04 to 0.1 in.), layer of the lubricating mortar. Thus, nearly all the concrete moves at the same velocity, i.e. by way of plug flow. It is possible that the formation of the lubricating film is aided by the fact that the dynamic action of the piston is transmitted to the pipe, but such a film is also caused by steel trowelling of a concrete surface. To allow for the film in the pipe, a cement content slightly higher than otherwise would be used is desirable. The magnitude of the friction developed depends on the consistency of the mix, but there must be no excess water because segregation would result.

It may be useful to consider the problems of friction and segregation in more general terms. In a pipe through which a material is pumped, there is a pressure gradient in the direction of flow due to two effects: head of the material and friction. This is another way of saying that the material must be capable of transmitting a sufficient pressure to overcome all resistances in the pipeline.

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Of all the components of concrete, it is only water that is pumpable in its natural state, and it is the water, therefore, that transmits the pressure to the other mix components.

Two types of blockage can occur. In one, water escapes through the mix so that pressure is not transmitted to the solids, which therefore do not move. This occurs when the voids in the concrete are not small enough or intricate enough to provide sufficient internal friction within the mix to overcome the resistance of the pipeline. Therefore, an adequate amount of closely packed fines is essential to create a 'blocked filter' effect, which allows the water phase to transmit the pressure but not to escape from the mix. In other words, the pressure at which segregation occurs must be greater than the pressure needed to pump the concrete.[4.30] It should be remembered, of course, that more fines mean a higher surface area of the solids and therefore a higher frictional resistance in the pipe.

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We can see thus how the second type of blockage can occur. If the fines content is too high, the friction resistance of the mix can be so large that the pressure exerted by the piston through the water phase is not sufficient to move the mass of concrete, which becomes stuck. This type of failure is more common in high strength mixes or in mixes containing a high proportion of very fine material such as crusher dust or fly ash, while the segregation failure is more apt to occur in medium or low strength mixes with irregular or gap grading.

The optimum situation, therefore, is to produce maximum frictional resistance within the mix with minimum void sizes, and minimum frictional resistance against the pipe walls with a low surface area of the aggregate. This means that the coarse aggregate content should be high, but the grading should be such that there is a low void content so that only little of the very fine material is required to produce the ‘blocked filter’ effect.

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The coarse aggregate content should be higher when the sand is fine. For example, ACI 304.2R[4.114] recommends, for aggregate with the maximum size of 20 mm ($\frac{3}{4}$ in.), the bulk volume of dry-rodded coarse aggregate of 0.56 to 0.66 when the sand fineness modulus is 2.40, and 0.50 to 0.60 when it is 3.00. Because the dry-rodded volume (see p. 128) compensates automatically for differences in particle shape, the values cited are equally appropriate for rounded and angular aggregate. It is important to remember that the dry-rodded volume is determined as a ratio of the volume of dry-rodded coarse aggregate to the volume of concrete, on the basis of ASTM Test Method C 29-09; this ratio is entirely distinct from the mass content of coarse aggregate per cubic metre of concrete in the actual mix.

Fine aggregate conforming to ASTM C 33-08; but with stricter limits at either of the permitted extremes, is suitable for use in pumped concrete. Experience has shown that, for pipes smaller than 125 mm (5 in.), 15 to 30 per cent of the fine

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aggregate should be finer than 300 μm (No. 50) sieve, and 5 to 10 per cent should be finer than 150 μm (No. 100) sieve.[4.114] Deficiency can be remedied by blending with very fine material such as crusher dust or fly ash. Crushed-rock fine aggregate can be made suitable by a small addition of rounded sand.[4.114] Grading zones found by experience to be satisfactory, are shown in Table 4.6.

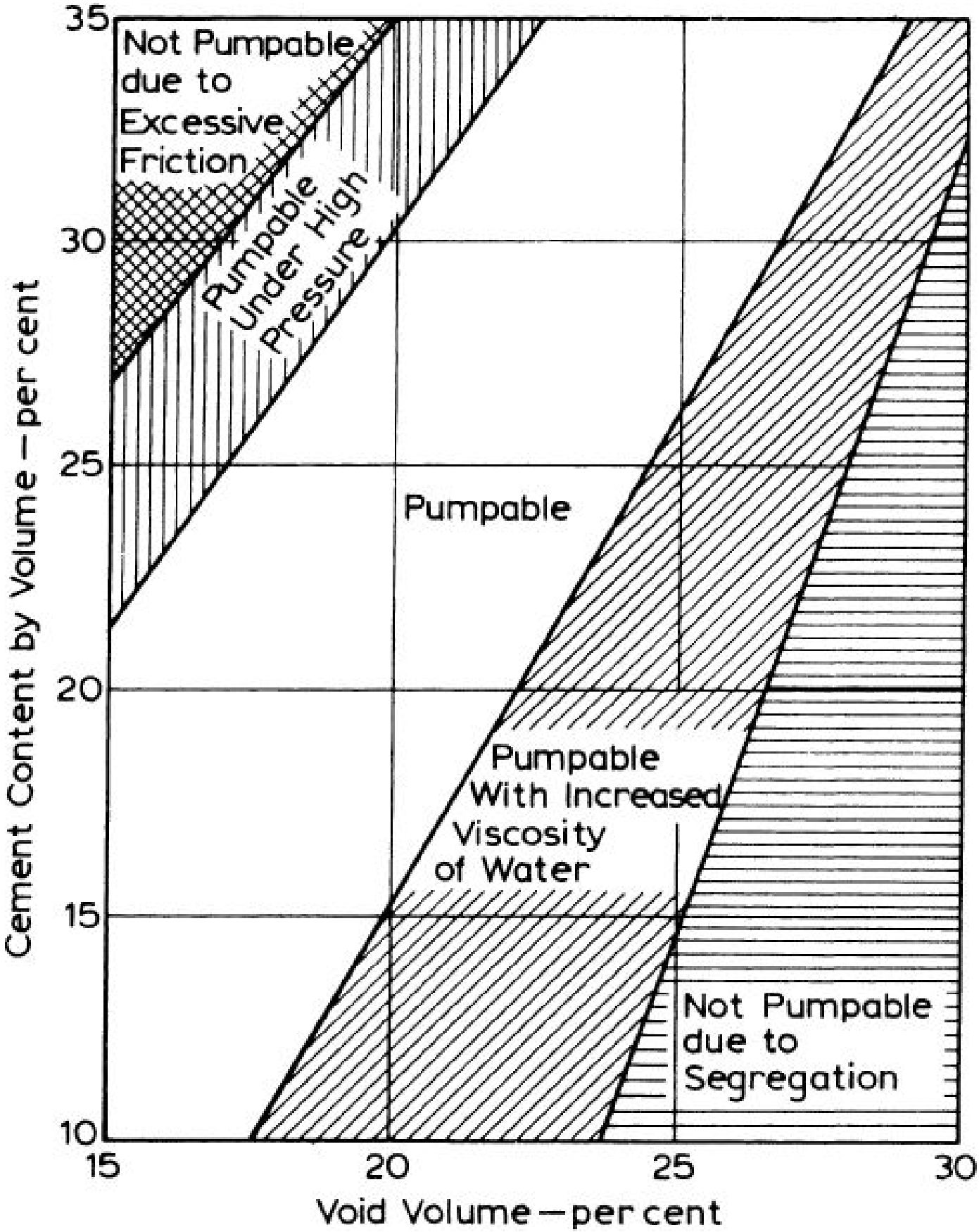
Table 4.6. Recommended Aggregate Gradings for Pumped Concrete (after ACI 304.2R-91)[4.114]

Size		*Cumulative percentage passing*	
Metric	*ASTM*	*25 mm (1 in.) max. size*	*20 mm ($\frac{3}{4}$ in.) max. size*
25 mm	1 in.	100	—
20 mm	$\frac{3}{4}$ in.	80–88	100
13 mm	$\frac{1}{2}$ in.	64–75	75–82
9.50 mm	$\frac{3}{8}$ in.	55–70	61–72
4.75 mm	No. 4	40–58	40–58
2.36 mm	No. 8	28–47	28–47
1.18 mm	No. 16	18–35	18–35
600 μm	No. 30	12–25	12–25
300 μm	No. 50	7–14	7–14
150 μm	No. 100	3–8	3–8
75 μm	No. 200	0	0

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British tests[4.49] have shown that generally the volumetric cement content (at an assumed density of 2450 kg/m^3) has to be at least equal to the void content of the aggregate but very fine material other than cement can be included with the latter. The pattern of the effect of the relation between the cement content and void content on pumpability is shown in Fig. 4.22.[4.50] However, it is only fair to add that theoretical calculations are not very helpful because the shape of the aggregate particles influences their void content. Some experimental data are shown in Fig. 4.23: they indicate that the upper limit of pumpability can be successfully exceeded by very rich concrete.[4.59]

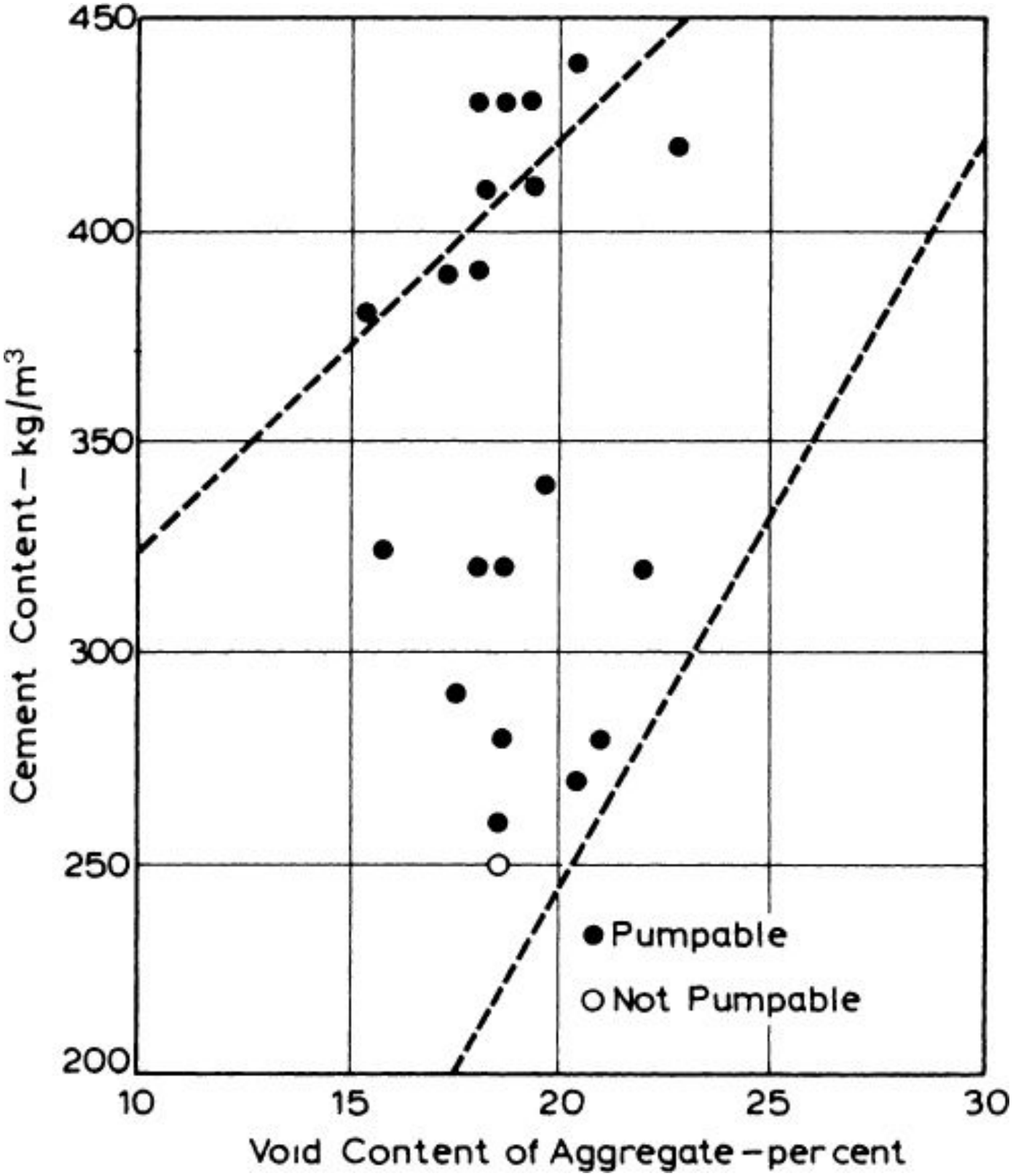
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Fig. 4.22. Pumpability of concrete in relation to cement content and void content of aggregate[4.50]

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Fig. 4.23. Limits on cement content for aggregates with various void contents with respect to pumpability of concrete[4.59]

It may be noted that a sudden rise in pressure caused by a restriction or by a reduction in the diameter of the pipe may result in segregation of the aggregate which is left behind as the cement paste moves past the obstacle.[4.31]

The shape of the aggregate influences the optimum mix proportions for good pumpability but both rounded and angular coarse aggregate can be used; the latter requires a higher volume of mortar in the mix.[4.114] Natural sands are often particularly suitable for pumping because of their rounded shape and also because their true grading is more continuous than with crushed aggregate where, within each size fraction, there is less variety in size. For both these reasons, the void content is low.[4.49] On the other hand, using combinations of size fractions of crushed aggregate, a suitable void content can be achieved. However,

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care is required as many crushed fines are deficient in the size fraction 300 to 600 μm (No. 50 to No. 30 ASTM) but have excess of material smaller than 150 μm (No. 100). When using crushed coarse aggregate, it should be remembered that crusher dust may be present and this should be taken into account in considering the grading of the fine aggregate. Generally, with crushed coarse aggregate, the fine aggregate content should be increased by about 2 per cent.[4.51]

Flowing concrete can be pumped but an over-cohesive mix with an increased sand content should be used.[4.119]

Any mix selection of concrete to be pumped must be subjected to a test. Although laboratory pumps have been used to predict the pumpability of concrete,[4.79] the performance of any given mix has to be assessed under the actual site conditions, including the equipment to be used and the distance through which the concrete is to be pumped.

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Various pumping aids[4.67] are available for the purpose of improving cohesion of the mix through increasing the viscosity of the water and of lubrication of the pipe walls. The pumping aids are meant to be used in addition to, and not instead of, the selection of appropriate mix proportions. Entrainment of a limited amount of air, 5 or possibly 6 per cent, is also helpful.[4.79] However, excess amount of air would decrease the pumping efficiency as the air would become compressed.

Pumping lightweight aggregate concrete

In the early days of the development of pumping, there were difficulties with the use of lightweight aggregate whose surface is not sealed. The reason for this is that, under pressure, the air in the voids in the aggregate contracts, and water is forced into the pores with the result that the mix becomes too dry.

A remedy was found in pre-soaking both the coarse and the fine aggregate over a period of 2 to 3 days or by a very rapid vacuum saturation.[4.114]

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Whereas the absorbed water does not form part of the free water in the mix (see p. 276) it does affect the batch proportions by mass. Pumping lightweight concrete vertically up to 320 m (1050 ft) has been reported.

The use of saturated aggregate may have implications for the resistance of concrete to freezing and thawing, and a period of several weeks may be necessary prior to exposure.[4.114] However, at very low temperatures, reliance on the waiting period is inadequate, and use of aggregate with very low absorption, coupled with the use of a special agent, may be necessary. This agent, added to the mix, enters the pores near the surface of the aggregate but, when the initial hydration of Portland cement raises the pH, the viscosity of the agent increases and it forms a high-viscosity layer which hinders absorption of water due to the pumping pressure.[4.82]

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Shotcrete

This is the name given to mortar or concrete conveyed through a hose and pneumatically projected at high velocity onto a backup surface. The force of the jet impacting on the surface compacts the material so that it can support itself without sagging or sloughing, even on a vertical face or overhead. Other names are also used for some types of shotcrete, e.g. *gunite*, but only *sprayed concrete* is sufficiently general and is indeed the preferred term in the European Union terminology.

The properties of shotcrete are no different from the properties of conventionally placed mortar or concrete of similar proportions: it is the method of placing that bestows on shotcrete significant advantages in many applications. At the same time, considerable skill and experience are required in the application of shotcrete so that its quality depends to a large extent on the performance of the operators involved, especially in control of the actual placing by the nozzle.

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Because shotcrete is pneumatically projected on a backup surface and then gradually built up, only one side of formwork or a substrate is needed. This represents economy, especially when account is taken of the absence of form ties, etc. On the other hand, the cement content of shotcrete is high. Also, the necessary equipment and mode of placing are more expensive than in the case of conventional concrete. For these reasons, shotcrete is used primarily in certain types of construction: thin, lightly reinforced sections, such as roofs, especially shell or folded plate, tunnel linings, and prestressed tanks. Shotcrete is also used in repair of deteriorated concrete, in stabilizing rock slopes, in encasing steel for fireproofing, and as a thin overlay on concrete, masonry or steel. If shotcrete is applied to a surface covered by running water, an accelerator producing flash set, such as washing soda, is used. This adversely affects strength but makes repair work possible. Admixtures for shotcrete are specified by BS EN 934-5 : 2007. Generally, shotcrete is applied in a thickness up to 100 mm (4 in.).

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There are two basic processes by which shotcrete is applied. In the *dry mix process* (which is the more common of the two, in many parts of the world) cement and damp aggregate are intimately mixed and fed into a mechanical feeder or gun. The mixture is then transferred by a feed wheel or distributor (at a known rate) into a stream of compressed air in a hose, and carried up to the delivery nozzle. The nozzle is fitted inside with a perforated manifold through which water is introduced under pressure and intimately mixed with the other ingredients. The mixture is then projected at high velocity onto the surface to be shotcreted.

The fundamental feature of the *wet mix process* is that all the ingredients, including the mixing water, are mixed together to begin with. The mixture is then introduced into the chamber of the delivery equipment and from there conveyed pneumatically or by positive displacement. A pump similar to that of Fig. 4.21 can be used. Compressed air (or in the case of pneumatically

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conveyed mix, additional air) is injected at the nozzle, and the material is projected at high velocity onto the surface to be shotcreted.

Either process can produce excellent shotcrete, but the dry mix process is better suited for use with porous lightweight aggregate and with flash set accelerators, and is also capable of greater delivery lengths, as well as of intermittent operation.[4.34] The consistency of the mix can be controlled direct at the nozzle, and higher strengths (up to 50 MPa (or 7000 psi)) can be readily achieved.[4.34] On the other hand, the wet mix process gives a better control of the quantity of mixing water (which is metered, as opposed to judgement by the nozzle operator) and of any admixture used. Also, the wet mix process leads to less dust being produced and possibly to lower rebound. The process is suitable for large-volume operation.

Because of the high velocity of the impacting jet, not all the shotcrete projected on a surface remains in position: some material rebounds. This

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consists of the coarsest particles in the mix, so that the shotcrete in situ is richer than would be expected from the mix proportions as batched. This may lead to slightly increased shrinkage. The rebound is greatest in the initial layers and becomes smaller as a plastic cushion of shotcrete is built up. Typical percentages of material rebounded are:[4.34]

	For dry mix	*For wet mix*
in floors and slabs	5 to 15	0 to 5
on sloping or vertical surfaces	15 to 30	5 to 10
on soffits	25 to 50	10 to 20.

The significance of rebound is not so much in the waste of the material as in the danger from accumulation of rebounded particles in a position where they will become incorporated in the subsequent layers of shotcrete. This can occur if the rebound collects in inside corners, at the base of walls, behind reinforcement or embedded pipes, or on horizontal surfaces. Great care in placing of shotcrete is therefore necessary, and the use of large reinforcement is undesirable. The latter also

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leads to the risk of unfilled pockets behind the obstacle to the jet.

The projected shotcrete has to have a relatively dry consistency so that the material can support itself in any position; at the same time, the mix has to be wet enough to achieve compaction without excessive rebound. The usual range of water/cement ratios is 0.30 to 0.50 for dry mix shotcrete, and 0.40 to 0.55 for the wet mix.[4.34] Recommended aggregate gradings are given in Table 4.7. Curing of shotcrete is particularly important because the large surface/volume ratio can lead to rapid drying. Recommended practice is given in ACI 506R-90[4.34] and in BS EN 14487-2 : 2006.

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Table 4.7. Recommended Aggregate Gradings for Shotcrete[4.34]

Sieve size		Cumulative percentage passing		
Metric	*ASTM*	*Grading No. 1*	*Grading No. 2*	*Grading No. 3*
19 mm	$\frac{3}{4}$ in.	—	—	100
12 mm	$\frac{1}{2}$ in.	—	100	80–95
10 mm	$\frac{1}{8}$ in.	100	90–100	70–90
4.75 mm	No. 4	95–100	70–85	50–70
2.40 mm	No. 8	80–100	50–70	35–55
1.20 mm	No. 16	50–85	35–55	20–40
600 μm	No. 30	25–60	20–35	10–30
300 μm	No. 50	10–30	8–20	5–17
150 μm	No. 100	2–10	2–10	2–10

Shotcrete exhibits durability comparable with ordinary concrete. The only reservation concerns the resistance to freezing and thawing, especially in salt water.[4.91] Air entrainment of shotcrete is possible using the wet process, but achieving an adequately low bubble spacing factor (see p. 547) presents some difficulties.[4.94] However, addition of silica fume (7 to 11 per cent by mass of cement) leads to adequate resistance to freezing and thawing.[4.95] More generally, the addition of silica fume, in proportion of 10 to 15 per cent of the cement by mass, has been found to improve the

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cohesion and adhesion of shotcrete; rebound is reduced.[4.32] Such shotcrete can be put into service at an early age.[4.96] For very rapid use in service, dry process shotcrete can be made using regulated-set cement.[4.92] The durability of such shotcrete is good.

Underwater concrete

Placing concrete under water presents some special problems. First of all, washout of the concrete by the water must be avoided so that placement should take place by discharge from a steel pipe buried within the already placed, but still mobile, concrete. The pipe, known as a *tremie*, has to remain full throughout the concreting operation. In a way, tremie placing of concrete is similar to pumping but the flow of concrete takes place under the force of gravity only. Placements to depths of 250 m have been effected.

Continued discharge of concrete makes it flow laterally, and it is therefore essential that the concrete mix has appropriate flow characteristics.

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Moreover, these characteristics cannot be directly observed. A slump of 150 to 250 mm (6 to 10 in.) is necessary, depending on the presence of embedded items. Anti-washout admixtures are effective:[4.100] they make the concrete flow when pumped or moved but, when the concrete is at rest, its viscosity is high.[4.98]

Relatively rich mixes, containing at least 360 kg/m^3 (or 600 lb/yd^3) of cementitious material with about 15 per cent of pozzolanas included to improve the flow of concrete, have been traditionally recommended.[4.76] However, Gerwick and Holland[4.100] pointed out that, in large underwater pours, internal temperatures near the centre of the concrete can reach 70 to 95 °C (160 to 200 °F) and, on subsequent cooling, cracking can develop. If the concrete is unreinforced, the cracks can be very wide. For this reason, Gerwick and Holland[4.100] suggest the use of blended cements containing about 16 per cent Portland cement, 78 per cent coarse-ground blastfurnace slag, and 6

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per cent silica fume. The concrete is pre-cooled to 4 °C (40 °F) prior to discharge into the tremie. A water/cement ratio of 0.40 to 0.45 is commonly used.

Underwater concreting is a delicate operation which, if incorrectly carried out, can have undetected but serious consequences; use of experienced personnel is necessary.

Preplaced aggregate concrete

This type of concrete is produced in two stages. In the first operation, uniformly graded coarse aggregate is placed in the forms; either rounded or crushed aggregate is suitable. In heavily reinforced areas, compaction should be used. The volume of coarse aggregate represents about 65 to 70 per cent of the overall volume to be concreted. The remaining voids are filled with mortar in the second stage.

It is clear that the aggregate in the resulting concrete is of the gap-graded type. Examples of typical coarse and fine aggregates grading are

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shown in Tables 4.8 and 4.9, respectively. Optimum packing of the aggregate particles leads to great theoretical advantages but is not necessarily achieved in practice.

Table 4.8. Typical Gradings of Coarse Aggregate for Preplaced Aggregate Concrete[4.75]

Sieve size	*mm*	*38*	*25*	*19*	*13*	*10*
	in.	$1\frac{1}{2}$	*1*	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{3}{8}$
Cumulative percentage passing		95–100	40–80	20–45	0–10	0–2

Table 4.9. Typical Grading of Fine Aggregate for Preplaced Aggregate Concrete[4.75]

Sieve size	*Metric*	*2.36 mm*	*1.18 mm*	*600 μm*	*300 μm*	*150 μm*	*75 μm*
	ASTM No.	*8*	*16*	*30*	*50*	*100*	*200*
Cumulative percentage passing		100	95–100	55–80	30–55	10–30	0–10

The coarse aggregate must be free from dirt and dust because, since these are not removed in mixing, they would impair bond. Flushing the aggregate in situ might cause an accumulation of

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dust in the lower part of the pour which would become a zone of weakness. The aggregate must be saturated and preferably gently inundated.

The second operation consists of pressure pumping of mortar through slotted pipes, typically 35 mm (or $1\frac{1}{2}$ in.) in diameter and spaced at 2 m (7 ft) centres, starting from the bottom of the mass, the pipes being gradually withdrawn. Pumping over long distances is possible. ACI 304.1R-92[4.75] describes various techniques of mortar placing.

A typical mortar consists of a blend of Portland cement and pozzolana in the ratio of between 2.5 : 1 and 3.5 : 1, by mass. This cementitious material is mixed with sand in the ratio of between 1 : 1 and 1 : 1.5, at a water/cement ratio of 0.42 to 0.50. An intrusion aid is added in order to improve the fluidity of the mortar and to hold the solid constituents in suspension. The intrusion aid also delays somewhat the stiffening of the mortar and contains a small amount of aluminium powder, which causes a slight expansion

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before setting takes place. Strengths of about 40 MPa (6000 psi) are usual but higher strengths are also possible.[4.75]

Preplaced aggregate concrete can be placed in locations not easily accessible by ordinary concreting techniques; it can also be placed in sections containing a large number of embedded items that have to be precisely located: this arises, for instance, in nuclear shields. Likewise, because the coarse and fine aggregate are placed separately, the danger of segregation of heavy coarse aggregate, especially of steel aggregate used in nuclear shields, is eliminated. In this case, pozzolana should not be used because it reduces the density of the concrete and fixes less water.[4.63] Because of the reduced segregation, preplaced aggregate concrete is also suitable for underwater construction.

The drying shrinkage of preplaced aggregate concrete is lower than that of ordinary concrete, usually 200×10^{-6} to 400×10^{-6}. The reduced shrinkage is due to the point-to-point contact of

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the coarse aggregate particles, without a clearance for the cement paste necessary in ordinary concrete. This contact restrains the amount of shrinkage that can actually be realized, but occasionally shrinkage cracking can develop.[4.53] Because of the reduced shrinkage, preplaced aggregate concrete is suitable for the construction of water-retaining and large monolithic structures and for repair work. The low permeability of preplaced aggregate concrete gives it a high resistance to freezing and thawing.

Preplaced aggregate concrete may be used in mass construction where the temperature rise has to be controlled: cooling can be achieved by circulating refrigerated water round the aggregate and thus chilling it; the water is later displaced by the rising mortar. At the other extreme, in cold weather when frost damage is feared, steam can be circulated in order to pre-heat the aggregate.

Preplaced aggregate concrete is used also to provide an exposed aggregate finish: special aggregates are placed against the surfaces and be-

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come subsequently exposed by sandblasting or by acid wash.

Preplaced aggregate concrete appears thus to have many useful features but, because of numerous practical difficulties, considerable skill and experience in application of the process are necessary for good results to be obtained.

Vibration of concrete

The purpose of compaction of concrete, known also as *consolidation*, is to achieve the highest possible density of the concrete. The oldest means of achieving this is by ramming or punning, but nowadays this technique is very rarely used. The usual method of compaction is by vibration.

When concrete is freshly placed in the form, air bubbles can occupy between 5 per cent (in a mix of high workability) and 20 per cent (in a low-slump concrete) of the total volume. Vibration has the effect of fluidifying the mortar component of the mix so that internal friction is re-

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duced and packing of coarse aggregate takes place. It is with respect to achieving a close configuration of coarse aggregate particles that the particle shape is of great importance (see p. 115). Continuing vibration expels most of the remainder of entrapped air, but total absence of entrapped air is not normally achievable.

Vibration must be applied uniformly to the entire concrete mass as otherwise some parts of it would not be fully compacted while others might be segregated due to over-vibration. However, with a sufficiently stiff and well-graded mix, the ill effects of over-vibration can be largely eliminated. Different vibrators require different consistency of concrete for most efficient compaction so that the consistency of the concrete and the characteristics of the available vibrator have to be matched. It is worth noting that flowing concrete, although it may be self-levelling, does not achieve full compaction by gravity alone. However, the necessary duration of application of

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vibration can be reduced by about one-half compared with ordinary concrete.[4.47]

Good practical guidance on compaction of concrete is given by Mass[4.72] and also in ACI Guide 309R-87.[4.73]

Internal vibrators

Of the several types of vibrators, this is the most common one. It consists essentially of a poker, housing an eccentric shaft driven through a flexible drive from a motor. The poker is immersed in concrete and thus applies approximately harmonic forces to it; hence, the alternative names of *poker-* or *immersion vibrator.*

The frequency of vibration of a vibrator immersed in concrete varies up to 12 000 cycles of vibration per minute: between 3500 and 5000 has been suggested as a desirable minimum, with an acceleration of not less than 4*g* but, more recently, vibration at 4000 to 7000 cycles has found favour.

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The poker is easily moved from place to place, and is applied at 0.5 to 1 m (or 2 to 3 ft) centres for 5 to 30 seconds, depending on the consistency of the mix but, with some mixes, up to 2 minutes may be required. The relation between the radius of action of an immersion vibrator and the frequency and amplitude of vibration is discussed in ACI 309.1R-93.[4.74]

The actual completion of compaction can be judged by the appearance of the surface of the concrete, which should be neither honeycombed nor contain an excess of mortar. Gradual withdrawal of the poker at the rate of about 80 mm/sec (3 in./sec) is recommended[4.17] so that the hole left by the vibrator closes fully without any air being trapped. The vibrator should be immersed through the entire depth of the freshly deposited concrete and into the layer below if this is still plastic or can be brought again to a plastic condition. In this manner, a plane of weakness at the junction of the two layers can be avoided and monolithic concrete is obtained. With a lift great-

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er than about 0.5 m (2 ft) the vibrator may not be fully effective in expelling air from the lower part of the layer. An immersion vibrator will not expel air from the form boundary so that 'slicing' along the form by means of a flat plate on edge is necessary. The use of absorptive linings to the form is helpful in this respect.

Internal vibrators are comparatively efficient because all the work is done directly on the concrete, unlike other types of vibrators. Pokers are made in sizes down to 20 mm ($\frac{3}{4}$ in.) diameter so that they can be used even with heavily reinforced and relatively inaccessible sections. ACI Guide 309R-87[4.73] gives useful information on internal vibrators and on selection of appropriate types. Robot-operated internal vibrators are available in some countries.

External vibrators

This type of vibrator is rigidly clamped to the formwork resting on an elastic support, so that both the form and the concrete are vibrated. As a

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result, a considerable proportion of the work done is used in vibrating the formwork, which has to be strong and tight so as to prevent distortion and leakage of grout.

The principle of an external vibrator is the same as that of an internal one, but the frequency is usually between 3000 and 6000 cycles of vibration per minute, although some vibrators reach 9000 cycles per minute. Manufacturers' data have to be inspected carefully as sometimes the number of 'impulses' is quoted, an impulse being *half* a cycle. The Bureau of Reclamation[4.7] recommends at least 8000 cycles. The power output varies between 80 and 1100 W.

External vibrators are used for precast or thin in situ sections of such shape or thickness that an internal vibrator cannot be conveniently used. These vibrators are effective for concrete sections up to 600 mm (24 in.) thick.[4.73]

When an external vibrator is used, concrete has to be placed in layers of suitable depth as

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air cannot be expelled through too great a thickness of concrete. The position of the vibrator may have to be changed as concreting progresses if the height is more than 750 mm (30 in.).[4.73]

Portable, non-clamped external vibrators may be used at sections not otherwise accessible, but the range of compaction of this type of vibrator is very limited. One such vibrator is an electric hammer, sometimes used for compaction of concrete test specimens.

Vibrating tables

This can be considered as a case of formwork clamped to the vibrator, instead of the other way round, but the principle of vibrating the concrete and formwork together is unaltered.

The source of vibration, too, is similar. Generally a rapidly rotating eccentric mass makes the table vibrate with a circular motion. With two shafts rotating in opposite directions, the horizontal component of vibration can be neutralized so that the table is subjected to a simple harmon-

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ic motion in the vertical direction only. There exist also some small good quality vibrating tables operated by an electro-magnet fed with alternating current. The range of frequencies used varies between 50 and about 120 Hz. An acceleration of about 4*g* to 7*g* is desirable.[4.17] About 1.5*g* and an amplitude of 40 μm. (0.0015 in.) are believed to be the minima necessary for compaction,[4.18] but with these values a long period of vibration may be necessary. For simple harmonic motion, the amplitude, a, and the frequency, f, are related by the equation:

$$\text{acceleration} = a(2\pi f)^2.$$

When concrete sections of different sizes are to be vibrated, and in laboratory use, a table with a variable amplitude should be used. Variable frequency of vibration is an added advantage.

In practice, the frequency may rarely be varied during the actual compaction but, at least theoretically, there are considerable advantages in in-

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creasing the frequency and decreasing amplitude as consolidation progresses. The reason for this lies in the fact that initially the particles in the mix are far apart and the movement induced has to be of corresponding magnitude. On the other hand, once partial compaction has taken place, the use of a higher frequency permits a greater number of adjusting movements in a given time; a reduced amplitude means that the movement is not too large for the space available. Vibration at too large an amplitude relative to the inter-particle space results in the mix being in a constant state of flow so that full compaction is never achieved. Bresson and Brusin[4.71] found that there is an optimum amount of energy of vibration for every mix, and various combinations of frequency and acceleration will be satisfactory. However, a prediction of the optimum in terms of mix parameters is not possible.

A vibrating table provides a reliable means of compaction of precast concrete and has the advantage of offering uniform treatment.

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A variant of the vibrating table is a *shock table* used sometimes in precast concrete manufacture. The principle of this process of compaction is rather different from the high frequency vibration discussed earlier: in a shock table, violent vertical shocks are imparted at the rate of about 2 to 4 per second. The shocks are produced by a vertical drop of 3 to 13 mm ($\frac{1}{8}$ to $\frac{1}{2}$ in.), this being achieved by means of cams. Concrete is placed in the form in shallow layers while the shock treatment progresses: extremely good results have been reported but the process is rather specialized and not widely used.

Other vibrators

Various types of vibrators have been developed for special purposes but only a very brief mention of these will be made.

A surface vibrator applies vibration through a flat plate direct to the top surface of the concrete. In this manner, the concrete is restrained in all directions so that the tendency to segregate is lim-

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ited; for this reason, a more intense vibration can be used.

An electric hammer can be used as a surface vibrator when fitted with a bit having a large flat area, say 100 mm by 100 mm (4 in. by 4 in.); one of the main applications is in compacting test cubes.

A vibrating roller is used for consolidating thin slabs. For road construction various vibrating screeds and finishers are available; these are discussed in ACI 309R-87.[4.73] A power float is used mainly for granolithic floors in order to bind the granolithic layer to the main body of the concrete, and is more an aid in finishing than a means of compaction.

Revibration

It is usual to vibrate concrete immediately after placing so that consolidation is generally completed before the concrete has stiffened. All the preceding sections refer to this type of vibration.

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It has been mentioned, however, that, in order to ensure good bond between lifts, the upper part of the underlying lift should be revibrated, provided the lower lift can still regain a plastic state; settlement cracks and the internal effects of bleeding can thus be eliminated.

This successful application of revibration raises the question whether revibration can be more generally used. On the basis of experimental results, it appears that concrete can be successfully revibrated up to about 4 hours from the time of mixing[4.19] provided the vibrator will sink under its own weight into the concrete.[4.72] Revibration at 1 to 2 hours after placing was found to result in an increase in the 28-day compressive strength as shown in Fig. 4.24. The comparison is on the basis of the same total period of vibration, applied either immediately after placing or in part then, and in part at a specified time later. An increase in strength of approximately 14 per cent has been reported,[4.19] but actual values would depend on the workability of the mix and on details

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of the procedure: other investigators have found increases of 3 to 9 per cent.[4.80] In general, the improvement in strength is more pronounced at earlier ages, and is greatest in concretes liable to high bleeding[4.20] because the trapped water is expelled on revibration. For the same reason, revibration greatly improves watertightness[4.72] and also the bond between concrete and reinforcement near the top surface of the concrete as trapped bleed water is expelled. It is possible also that some of the improvement in strength is due to a relief of the plastic shrinkage stresses around aggregate particles.

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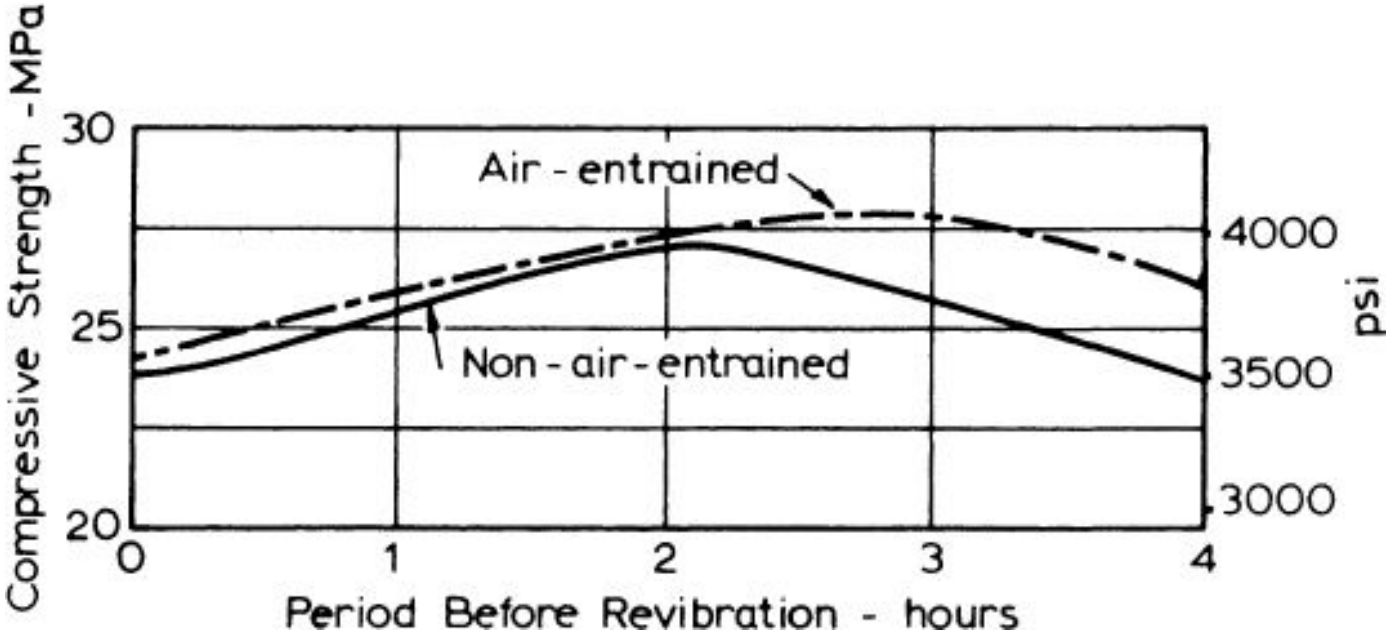


Fig. 4.24. Relation between 28-day compressive strength and the time of revibration[4.19]

Despite these advantages, revibration is not widely used as it involves an additional step in the production of concrete, and hence increased cost; also, if applied too late, revibration can damage the concrete.

Vacuum-dewatered concrete

One solution to the problem of combining a sufficiently high workability with a minimum water/cement ratio is offered by vacuum-dewatering of freshly placed concrete.

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The procedure is briefly as follows. A mix with a medium workability is placed in the forms in the usual manner. Because fresh concrete contains a continuous system of water-filled channels, the application of a vacuum to the surface of the concrete results in a large amount of water being extracted from a certain depth of the concrete. In other words, what might be termed 'water of workability' is removed when no longer needed. It may be noted that air bubbles are removed only from the surface as they do not form a continuous system.

The final water/cement ratio before the concrete sets is thus reduced and, as this ratio largely controls the strength, vacuum-dewatered concrete has a higher strength and also a higher density, a lower permeability, and a greater durability, as well as a higher resistance to abrasion, than would otherwise be obtained. However, some of the water extracted leaves behind voids, so that the full theoretical advantage of water removal may not be achieved in practice.[4.54] In fact, the

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increase in strength on vacuum treatment is proportional to the amount of water removed up to a critical value beyond which there is no significant increase, so that prolonged vacuum treatment is not useful. The critical value depends on the thickness of concrete and on the mix proportions.[4.55] Nevertheless, the strength of vacuum-dewatered concrete almost follows the usual dependence on the final water/cement ratio, as shown in Fig. 4.25.

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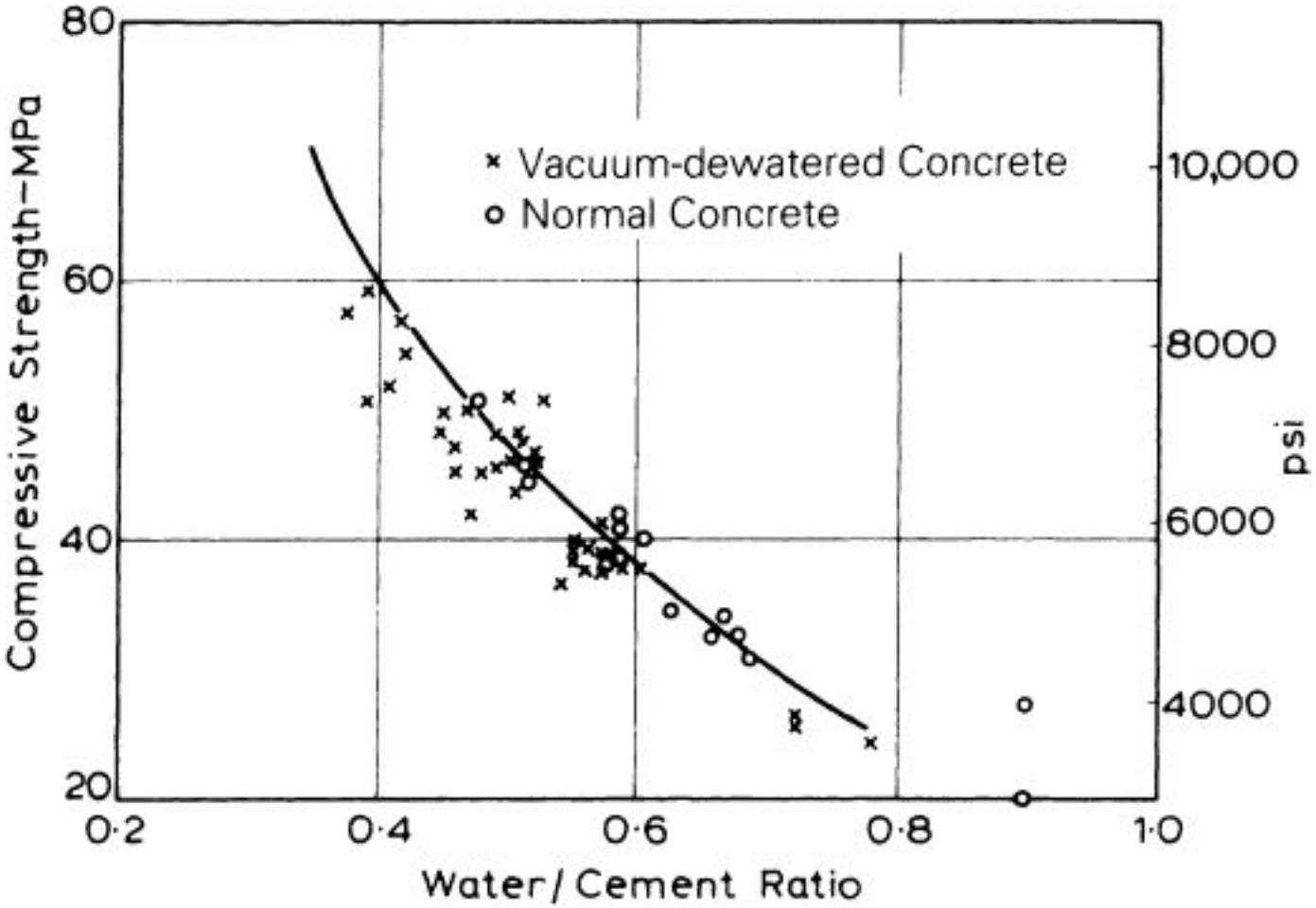


Fig. 4.25. Relation between the strength of concrete and the calculated water/cement ratio after vacuum dewatering[4.55]

The vacuum is applied through porous mats connected to a vacuum pump. The mats are placed on fine filter pads which prevent the removal of cement together with the water. The mats can be placed on top of the concrete immediately after screeding, and can also be incorporated in the inside faces of vertical forms.

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Vacuum is created by a vacuum pump; its capacity is governed by the perimeter of the mat, and not its area. The magnitude of the applied vacuum is usually about 0.08 MPa (11 psi). This vacuum reduces the water content by up to 20 percent. The reduction is greater nearer to the mat and it is usual to assume the suction to be fully effective over a depth of 100 to 150 mm (4 to 6 in.) only. The withdrawal of water produces settlement of the concrete to the extent of about 3 per cent of the depth over which the suction acts. The rate of withdrawal of water falls off with time, and it has been found that processing during 15 to 25 minutes is usually most economical. Little reduction in water content occurs beyond 30 minutes.

Strictly speaking, no suction of water takes place during vacuum-dewatering but merely a fall of pressure below atmospheric is communicated to the interstitial fluid of the fresh concrete. This would mean that compaction by atmospheric pressure is taking place. Thus, the amount of

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water removed would be equal to the contraction in the total volume of concrete and no voids would be produced. However, in practice, some voids are formed and, for the same final water/cement ratio, ordinary concrete has been found to have a somewhat higher strength than vacuum-dewatered concrete. This is discernible in Fig. 4.25.

The formation of voids can be prevented if, in addition to vacuum-dewatering, intermittent vibration is applied; under those circumstances a higher degree of consolidation is achieved and the amount of water withdrawn can be nearly doubled. In tests by Garnett,[4.21] good results were obtained with vacuum-dewatering for 20 minutes accompanied by vibration between the 4th and 8th minutes, and again between the 14th and 18th minutes.

Vacuum-dewatering can be used over a fairly wide range of aggregate/cement ratios and aggregate gradings, but a coarser grading yields more water than a finer one. Furthermore, some

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of the finest material is removed by the processing, and fine materials, such as pozzolanas, should not be incorporated in the mix. The use of a cement content not exceeding 350 kg/m^3 (590 lb/yd^3) and of water-reducing admixtures so that the slump does not exceed 120 mm (5 in.) has been recommended.[4.109]

Vacuum-dewatered concrete stiffens very rapidly so that formwork can be removed within about 30 minutes of casting, even on columns 4.5 m (15 ft) high. This is of considerable economic value, particularly in a precast concrete factory, as the forms can be re-used at frequent intervals. Usual curing is essential.

The surface of vacuum-dewatered concrete is entirely free from pitting and the uppermost 1 mm (0.04 in.) is highly resistant to abrasion. These characteristics are of special importance in concrete which is to be in contact with water flowing at a high velocity. Another useful characteristic of vacuum-dewatered concrete is that it bonds well to old concrete and can, therefore,

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be used for resurfacing road slabs and in other repair work. Vacuum treatment thus appears to be a valuable process, which is extensively used in some countries, especially for slabs and floors.[4.54]

Permeable formwork

A recent development, in some ways similar in concept to vacuum-dewatering, is the use of permeable formwork. Here, the formwork for vertical surfaces consists of a polypropylene fabric fixed to plywood backing which contains drain holes. Thus, the formwork acts as a filter through which air and bleed water escape but the cement is, for the most part, retained in the body of the concrete, although it is carried towards the formwork. A local increase in cement content of 20 to 70 kg/m^3 (30 to 110 lb/yd^3) has been reported.[4.93]

In addition to reducing the formwork pressure, the permeable formwork lowers the water/cement ratio in the surface zone, up to a depth of 20 mm; the reduction varies steadily from about 0.15 next

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to the formwork to a negligible amount at a depth of 20 mm.[4.99] The effect of the greatly reduced water/cement ratio is to reduce surface absorption and water permeability of the outer zone of the concrete, which is often critical from the durability standpoint. It should be noted, however, that 20 mm is less than the cover to reinforcement under exacting conditions of exposure. The surface hardness of the concrete is also increased; this improves the resistance of concrete to cavitation and erosion.

Because much of the surplus mix water escapes in the horizontal direction, the amount of bleed water at the top surface is reduced. This allows earlier finishing of the surface but, when ambient conditions are conducive to rapid drying, the absence of bleeding may lead to plastic shrinkage cracking. Appropriate measures need to be taken.

The surface produced by permeable formwork is free from bleed streaking and entrapped air pock-marks, thus enhancing the appearance of

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the exposed surfaces. While wet curing following formwork removal is desirable, its absence is less harmful than is the case with the usual, impermeable formwork.

Analysis of fresh concrete

In considering the ingredients of a concrete mix, we have so far assumed that the actual proportions correspond to those specified. Modern batching plants can provide a record of materials in each batch, but this does not include details of aggregate grading nor sufficient information about the moisture content of the aggregate (see p. 132). Moreover, if the batch record could be totally relied upon in all cases, there would be little need for testing the strength of hardened concrete. However, in practice, mistakes, errors and even deliberate actions can lead to incorrect mix proportions, and it is sometimes useful to determine the composition of concrete at an early stage; the two values of greatest interest are the cement content and water/cement ratio. It is the

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procedures for determining these values that are referred to as the analysis of fresh concrete.

ASTM test methods for the determination of the cement and water contents have been withdrawn. A so-called rapid analysis machine described in refs 4.57, 4.84 and 4.85 has not proved to be successful.

The U.S. Army[4.77] uses a test which relies on chloride titration for determining the water content and on calcium titration for cement content. The test can be performed in the field and takes no more than a quarter of an hour. However, the fine part (smaller than 150 μm (No. 100) sieve) of calcareous aggregate cannot be distinguished from the cement.

The use of the principle of buoyancy to determine the water/cement ratio of a mix was used by Naik an Ramme[4.86] but it requires the knowledge of the aggregate/cement ratio in the mix, which may well be uncertain or unreliable.

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A pressure-filter method has also been developed in which the material smaller than 150 μm. (No. 100) sieve is separated out by filtering and pressing dry;[4.36] the mass of cement is taken as the mass of this fraction corrected for aggregate finer than 150 μm (No. 100) sieve in the material as batched. This is a likely source of error. Separation of cement by flotation has also been developed.[4.81]

A totally different approach in the determination of cement content of fresh concrete is based on the separation of cement using a heavy liquid and a centrifuge.[4.38] This has not been very successful, especially when the finest aggregate particles have a specific gravity not significantly lower than that of cement.

A recent development is to determine the water/cement ratio by the measurement of electrical resistivity using a probe immersed in fresh concrete.[4.124] This approach can be relied on only

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with a given mix, the change in resistivity indicating a departure from the expected w/c.

As far as the determination of the water content in fresh concrete is concerned, this can be measured by estimating the degree of scattering of thermal neutrons emitted by a source placed within the bulk of the aggregate or within a sample of the mix.[4.69] Hydrogen is the most important element influencing scattering and retardation of thermal neutrons and, since hydrogen is almost exclusively bound in water, the nuclear method can provide a value of the water content with an accuracy of ±0.3 per cent. The technique also requires the dry density of the aggregate to be taken into account, and this is calculated from the back-scattering of gamma radiation from a second source. The complete apparatus comprises gamma and thermal neutron sources, neutron and scintillation detectors, and associated counters. Calibration is carried out in situ and is a time-consuming process. Use of microwave oven drying has been proposed.

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We can see that there exist no reliable and practicable procedures for the measurement of the water/cement ratio of fresh concrete. Indeed, there exists no test for the composition of fresh concrete that is convenient and reliable enough to be used as a preplacement acceptance test.

Self-compacting (self-consolidating) concrete

This type of concrete (in American parlance, self-consolidating or SCC) expels entrapped air without vibration and travels around obstacles such as reinforcement, to fill all space within the formwork. This is useful with intricate patterns of prestressing tendons and poorly accessible areas near anchorages. Vibration is noisy and therefore objectionable to neighbours, especially at night and at weekends. Avoiding this noise is the second argument for the use of self-consolidating concrete.

There is also a third reason, and that is the health effects of immersion vibrators on operat-

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ives; holding the vibrator damages nerves and blood vessels and causes the so-called ‘white finger’ or ‘hand vibration’ syndrome. This is obviously socially undesirable, and in the UK there are regulations on the use of hand-held vibrators. As yet, self-compacting concrete is not widely used in the UK; in addition to Japan, Sweden and the Netherlands are leaders in the field. In the USA, the Precast/Prestressed Concrete Institute (PCI) already has a guide on SCC, and ACI has produced a very helpful guide, 237R-07.

Interestingly, the impetus for the development of self-compacting concrete came from the desire to minimize the use of unskilled labour in Japan. There is no doubt that self-consolidating concrete will become more widespread in the near future, even for lightweight concrete.

There are three desiderata for the concrete to be classified as self-consolidating: flowing ability; passing through closely spaced reinforcement; and resistance to segregation. Various tests for each of the three properties have been pro-

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posed but no comprehensive test method has been standardized. In 2010, there were published five BS EN standards under the designation BS EN 12350: Part 8: slump-flow; Part 9: V-funnel test; Part 10: L-box test; Part 11: sieve segregation test; and Part 12: J-ring test.

The means of achieving self-compacting concrete are: the use of more fines (smaller than 600 μm) than usual; an appropriate viscosity achieved by a controlling agent; w/c of about 0.4; use of a superplasticizer; a good aggregate shape and texture; less coarse aggregate than usual (50% by volume of all solids). This may result in lower aggregate interlock, which is beneficial in shear resistance. Clearly, very good batching controls are necessary.

Self-compacting concrete is very useful for heavily reinforced members of any shape and with bottlenecks, both in precast concrete and in situ. The only limitation is that the top surface must be horizontal. Recent standards are BS EN 206-9 : 2010 and ASTM C 1712-09.

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Chapter 5. Admixtures

The early chapters described the properties of Portland cement and of a wide range of cementitious materials, as well as of the aggregate used in making concrete, together with a discussion of the influence of these materials and their combinations on the properties of fresh concrete. To a lesser extent, the influence on the properties of hardened concrete was also discussed but, before the latter topic is considered more fully, it is useful to review one more ingredient of the concrete mix: admixtures.

While admixtures, unlike cement, aggregate and water, are not an essential component of the concrete mix, they are an important and increasingly widespread component: in many countries, a mix which contains no admixtures is nowadays an exception.

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Benefits of admixtures

The reason for the large growth in the use of admixtures is that they are capable of imparting considerable physical and economic benefits with respect to concrete. These benefits include the use of concrete under circumstances where previously there existed considerable, or even insuperable, difficulties. They also make possible the use of a wider range of ingredients in the mix.

Admixtures, although not always cheap, do not necessarily represent additional expenditure because their use can result in concomitant savings, for example, in the cost of labour required to effect compaction, in the cement content which would otherwise be necessary, or in improving durability without the use of additional measures.

It should be stressed that, while properly used admixtures are beneficial to concrete, they are no remedy for poor quality mix ingredients, for use of incorrect mix proportions, or for poor workmanship in transporting, placing and compaction.

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Types of admixtures

An admixture can be defined as a chemical product which, except in special cases, is added to the concrete mix in quantities no larger than 5 per cent by mass of cement during mixing or during an additional mixing operation prior to the placing of concrete, for the purpose of achieving a specific modification, or modifications, to the normal properties of concrete.

Admixtures may be organic or inorganic in composition but their chemical character, as distinct from mineral, is their essential feature. Indeed, in American nomenclature, they are called *chemical admixtures* but in this book such a qualification is superfluous because the mineral products incorporated in the mix, almost invariably in excess of 5 per cent of the mass of cement, are referred to as cementitious materials or as additives.

Admixtures are commonly classified by their function in concrete but often they exhibit some

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additional action. The classification of ASTM C 494-10 is as follows:

Type A Water-reducing

Type B Retarding

Type C Accelerating

Type D Water-reducing and retarding

Type E Water-reducing and accelerating

Type F High-range water-reducing or superplasticizing, and

Type G High-range water-reducing and retarding, or superplasticizing and retarding

The British Standard for admixtures is BS EN 934-2 : 2009. Also relevant are BS EN 480 – numerous parts.

In practice, admixtures are marketed as proprietary products, and promotional literature sometimes includes claims of varied and wide-ranging benefits. While these may be true, some

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of the benefits occur only indirectly as a consequence of particular circumstances so that it is important to understand the specific effects of admixtures before they are used. Moreover, as ASTM C 494-10 points out, the specific effects produced may vary with the properties and proportions of the other ingredients of the mix.

Admixtures may be used in solid or liquid state. The latter is usual because a liquid can be more rapidly dispersed in a uniform manner during mixing of concrete. Properly calibrated dispensers are used, the admixture being discharged into the mixing water, or separately in dilute form but simultaneously with the mixing water, usually during the latter part of the water feed. Superplasticizers are subject to special methods of incorporation into the mix.

The dosages of the various admixtures, usually expressed as a percentage of the mass of cement in the mix, are recommended by the manufacturers but they are often varied according to circumstances.

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The effectiveness of any admixture may vary depending on its dosage in the concrete and also on the constituents of the mix, especially the properties of the cement. With some admixtures, the relevant dosage is the solids content and not the total mass of the admixture in liquid form. However, as far as the water content of the mix is concerned, the total volume of the liquid admixtures should be counted in but the solids content of superplasticizers should be excluded.

It is important that the effect of any admixture should not be highly sensitive to small variations in its dosage as such variations can occur accidentally during the production of concrete. The effect of many admixtures is influenced by temperature; for this reason their performance at extreme temperatures should be ascertained prior to use.

Admixtures should, generally speaking, not be allowed to come into contact with skin or eyes.

In addition to the chemical admixtures which will be discussed in this chapter, there exist also

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air-entraining agents which are considered in Chapter 11.

Accelerating admixtures

For brevity, these ASTM Type C admixtures will be referred to as *accelerators*. Their function is primarily to accelerate the early strength development of concrete, that is hardening (see p. 19), although they may also coincidentally accelerate the setting of concrete. If a distinction between the two actions is required, it may be useful to refer to set-accelerating properties.

Accelerators may be used when concrete is to be placed at low temperatures, say 2 to 4 °C (35 to 40 °F), in the manufacture of precast concrete (where a rapid removal of formwork is desirable) or in urgent repair work. Other benefits of using an accelerator are that it allows earlier finishing of the concrete surface and application of insulation for protection, and also putting the structure into service earlier.

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Conversely, at high temperatures, accelerators may result in too high a rate of development of heat of hydration and in shrinkage cracking.[5.4]

While accelerators are often used at very low temperatures, they are not anti-freezing agents; they depress the freezing point of concrete by no more than 2 °C (or about 3.5 °F), so that the usual anti-freezing precautions should always be taken (see p. 405). Special anti-freezing agents are being developed[5.8,5.9] but are still not fully proven.

The most common accelerator used over many decades was calcium chloride. Calcium chloride is effective in accelerating the hydration of the calcium silicates, mainly C_3S, possibly by a slight change in the alkalinity of the pore water or as a catalyst in the reactions of hydration. Although the mechanism of its action is even now imperfectly understood, there is no doubt that calcium chloride is an effective and cheap accelerator but it has one serious defect: the presence of chloride ions in the vicinity of steel reinforce-

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ment or other embedded steel is highly conducive to corrosion; this topic is discussed in Chapter 11.

Although the reactions of corrosion take place only in the presence of water and oxygen, the risks attendant on the presence of chloride ions in concrete containing steel are such that calcium chloride should never be incorporated into reinforced concrete; in prestressed concrete, the risks are even higher. In consequence, various standards and codes prohibit the use of calcium chloride in concrete containing embedded steel or aluminium. Moreover, even in plain concrete, when its durability may be impaired by outside agencies, the use of calcium chloride may be inadvisable. For instance, the resistance of cement to sulfate attack is reduced by the addition of $CaCl_2$ to lean mixes, and the risk of an alkali-aggregate reaction, when the aggregate is reactive, is increased.[5.24] However, when this reaction is effectively controlled by the use of low-alkali cement and the addition of pozzolanas, the effect of $CaCl_2$ is very small. Another undesirable feature

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of the addition of $CaCl_2$ is that it increases the drying shrinkage usually by about 10 to 15 per cent, sometimes even more,[5.24] and possibly increases also the creep.

Although the addition of $CaCl_2$ reduces the danger of frost attack during the first few days after placing, the resistance of air-entrained concrete to freezing and thawing at later ages is adversely affected. Some indication of this is given in Fig. 5.1.

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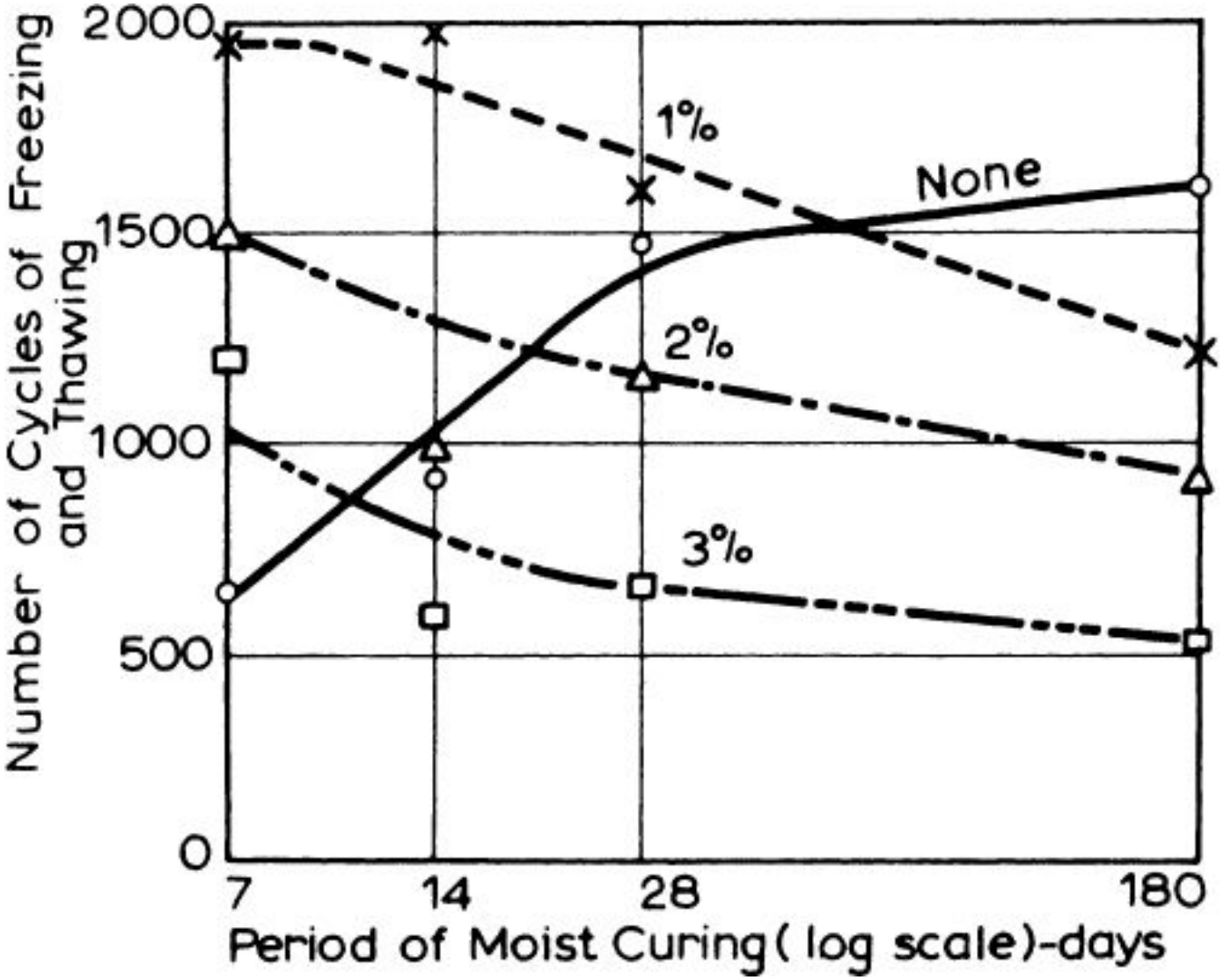


Fig. 5.1. Resistance to freezing and thawing of concrete cured moist at 4 °C (40 °F) for different contents of $CaCl_2$[5.24]

On the credit side, $CaCl_2$ has been found to raise the resistance of concrete to erosion and abrasion, and this improvement persists at all ages.[5.24] When plain concrete is steam cured, $CaCl_2$ increases the strength of concrete and per-

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mits the use of a more rapid temperature rise during the curing cycle[5.25] (see p. 370).

The action of sodium chloride is similar to that of calcium chloride but is of lower intensity. The effects of NaCl are also more variable and a depression in the heat of hydration, with a consequent loss of strength at 7 days and later, has been observed. For this reason, the use of NaCl is definitely undesirable. Barium chloride has been suggested but it acts as an accelerator only under warm conditions.[5.44]

Some researchers express the view that the use of calcium chloride does not contribute significantly to the corrosion of steel reinforcement if concrete is well proportioned and well compacted, and if the cover to the reinforcement is adequate.[5.53] Unfortunately, on site, such perfection may, from time to time, not be achieved, and the risk of using calcium chloride greatly outweighs its benefits. Moreover, experience has shown that, under extreme conditions of exposure existing in some countries, only high perform-

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ance concrete would protect the reinforcement from corrosion (see Chapter 13).

Because of this concern about the corrosion of reinforcement, the use, properties and effects of calcium chloride will not be discussed further in the present book. This concern has led to the search for chloride-free accelerators. No single accelerator has become widely accepted but a description of those which can be used may be of value.

Calcium nitrite and calcium nitrate are possible accelerators; the former also appears to be a corrosion inhibitor.[5.1] Calcium formate and sodium formate are also possibilities, although the latter would introduce sodium into the mix, and this alkali is known to influence hydration and also has a potential reaction with some aggregates (see p. 145).

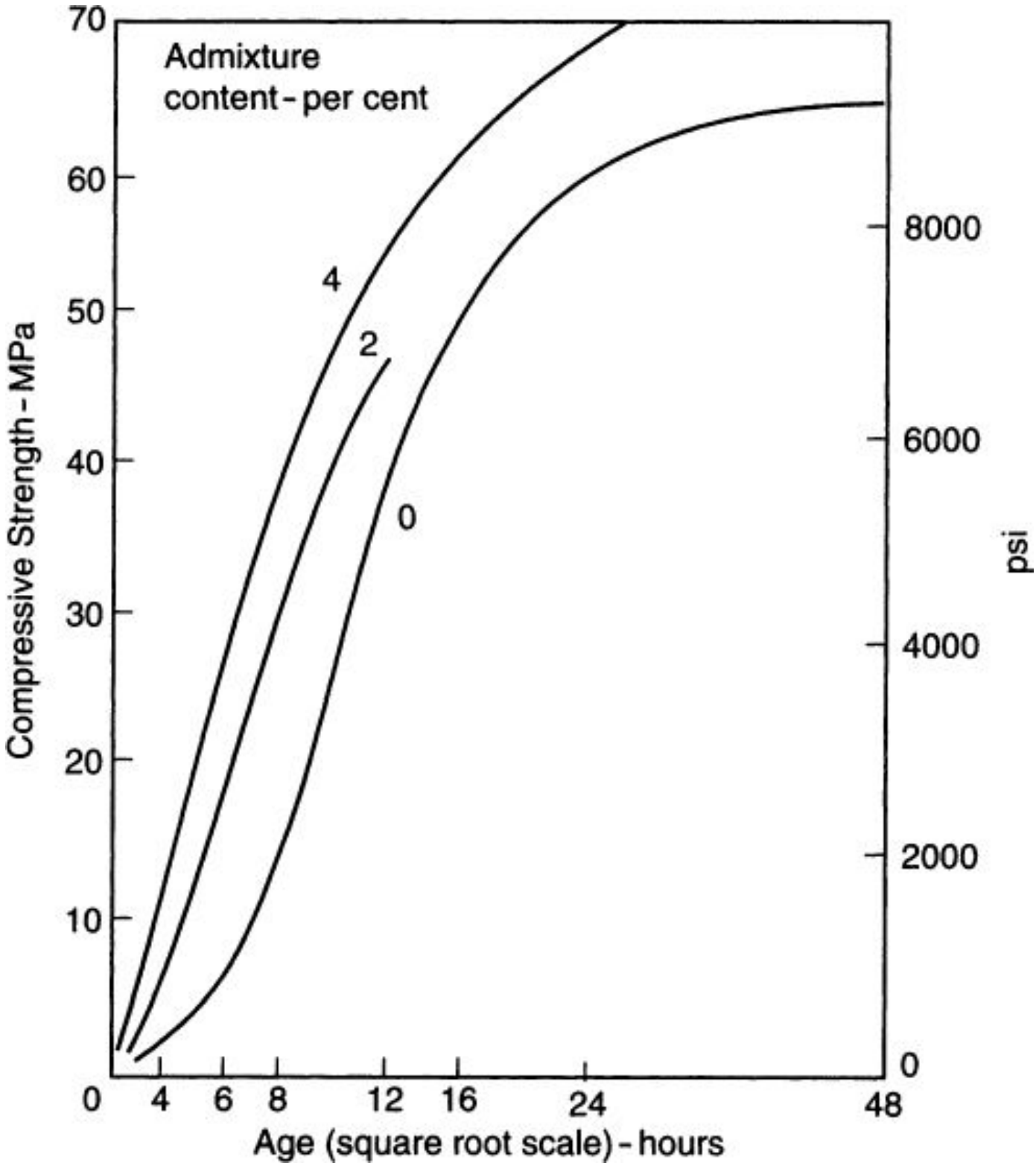
Calcium formate is effective only when used with cements which have a ratio of C_3A to SO_3 of at least 4 and have a low SO_3 content; cements produced using coal with a relatively high sulfur

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content do not satisfy this requirement.[5.7] For this reason, trial mixes involving any given cement should be made. It may be noted also that calcium formate has a very low solubility in water.[5.1] Used at dosages of 2 to 3 per cent by mass of cement, calcium formate increases the strength of concrete up to about 24 hours, the effect being greater with low C_3A cements.[5.3]

Massazza and Testolin[5.13] found that, with calcium formate, concrete could achieve at $4\frac{1}{2}$ hours the strength which would be reached only at 9 hours without the admixture, as shown in the example of Fig. 5.2. It is useful to note that calcium formate does not cause a retrogression of strength. On the other hand, the possibility of side effects of this accelerator has not been eliminated.[5.12,5.33]

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Fig. 5.2. Influence of calcium formate at various contents (by mass of cement) on the development of strength of concrete with a cement content of 420 kg/m^3 (710 lb/yd^3) and a water/cement ratio of 0.35 (cited in ref. 5.13)

Triethanolamine is a possible accelerator but it is very sensitive to dosage variation and to the composition of cement.[5.34] For this reason, triethanolamine is not used except to offset the retarding action of some water-reducing admixtures.

The precise mode of action of accelerators is still unknown. Moreover, the effect of accelerators on early strength of concrete very much depends on the particular accelerator used, as well as on the cement used, even for cements of the same nominal type. The full actual composition of the admixtures is usually not disclosed for commercial reasons so that it is necessary to ascertain the performance of any potential cement–admixture combination.

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The extent of the problem was demonstrated by Rear and Chin[5.20] who tested concretes of the same mix proportions (water/cement ratio of 0.54) made with five Type I Portland cements and three admixtures used at three dosages: No. 1 calcium-nitrite-based; No. 2 calcium-nitrate-based; and No. 3 sodium-thiocyanate-based. The range of compound composition of the cements (in per cent) was as follows:

C_3S	49 to 59
C_2S	16 to 26
C_3A	5 to 10
C_4AF	7 to 11.

The fineness of cement ranged from 327 to 429 m^2/kg measured by the Blaine method.

From the resulting compressive strengths determined at 20 °C (72 °F), shown in Table 5.1, it can be seen that there is a very wide variation in the performance of each of the admixtures when used with the different cements, as well as

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between the three admixtures. In all cases, the strength is expressed as a percentage of strength of the accelerator-free concrete.

Table 5.1. Effect of Accelerators on the Strength of Concretes made with Different Cements[5.20]

Accelerator No.	*Dosage ml/1100 kg of cement*	*Range of compressive strength (per cent) at the age of:*		
		1 day	*3 days*	*7 days*
	0	100	100	100
	1300	100–173	105–115	97–114
1	2600	112–175	107–141	111–129
	3900	111–166	111–143	113–156
	0	100	100	100
	740	64–130	90–113	100–116
2	1480	65–157	95–113	105–132
	2220	58–114	99–115	107–123
	0	100	100	100
	195	111–149	115–131	100–120
3	390	123–185	101–132	107–130
	585	121–171	115–136	104–129

The ASTM Specification C 494-10 includes a requirement that, when a Type C admixture is used, the initial set, measured by the penetration resistance test prescribed in ASTM C 403-10 be at least 1 hour earlier, but no more than $3\frac{1}{2}$

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hours earlier, than that of a control mix. The compressive strength at 3 days is to be 125 per cent of the strength of the control concrete. The strength beyond the age of 28 days is allowed to be lower than the strength of the control concrete, but retrogression of strength is not permitted. BS EN 934-2 : 2009 prescribes initial setting time, strength and air content. That standard prescribes also the requirements for other types of admixtures.

The preceding discussion indicates that no single accelerator is widely accepted. It is useful to note, at the same time, that the demand for accelerators has decreased, especially in the manufacture of precast concrete, as there exist other means of achieving a high early strength, such as the use of very low water/cement ratios in conjunction with superplasticizers. However, the use of accelerators at low placing temperatures continues.

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Retarding admixtures

A delay in the setting of the cement paste can be achieved by the addition to the mix of a retarding admixture (ASTM Type B), for brevity, referred to as a *retarder*. Retarders generally slow down also the hardening of the paste although some salts may speed up the setting but inhibit the development of strength. Retarders do not alter the composition or identity of products of hydration.[5.45]

Retarders are useful in concreting in hot weather, when the normal setting time is shortened by the higher temperature, and in preventing the formation of cold joints. In general, they prolong the time during which concrete can be transported, placed, and compacted. The delay in hardening caused by the retarders can be exploited to obtain an architectural finish of exposed aggregate: the retarder is applied to the interior surface of the formwork so that the hardening of the adjacent cement is delayed. This ce-

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ment can be brushed off after the formwork has been struck so that an exposed aggregate surface is obtained.

The use of retarders can sometimes affect structural design; for example, continuous massive pours can be used with controlled retardation of the various parts of the pour, instead of segmental construction (see p. 398).

Retarding action is exhibited by sugar, carbohydrate derivatives, soluble zinc salts, soluble borates and some other salts;[5.51] methanol is also a possible retarder.[5.12] In practice, retarders which are also water-reducing (ASTM Type B) are more commonly used; these are described in the next section.

The mechanism of the action of retarders has not been established with certainty. It is likely that they modify the crystal growth or morphology,[5.37] becoming adsorbed on the rapidly formed membrane of hydrated cement and slowing down the growth of calcium hydroxide nuc-

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lei.[5.11] These actions result in a more efficient barrier to further hydration than is the case without an admixture. The admixtures are finally removed from solution by being incorporated into the hydrated material but this does not necessarily mean the formation of different complex hydrates.[5.36] This is also the case with water-reducing and retarding admixtures, that is ASTM Class D: Khalil and Ward[5.43] showed that the linear relation between the heat of hydration and the mass of non-evaporable water is unaffected by the addition of a lignosulfonate-based admixture (see Fig. 5.3).

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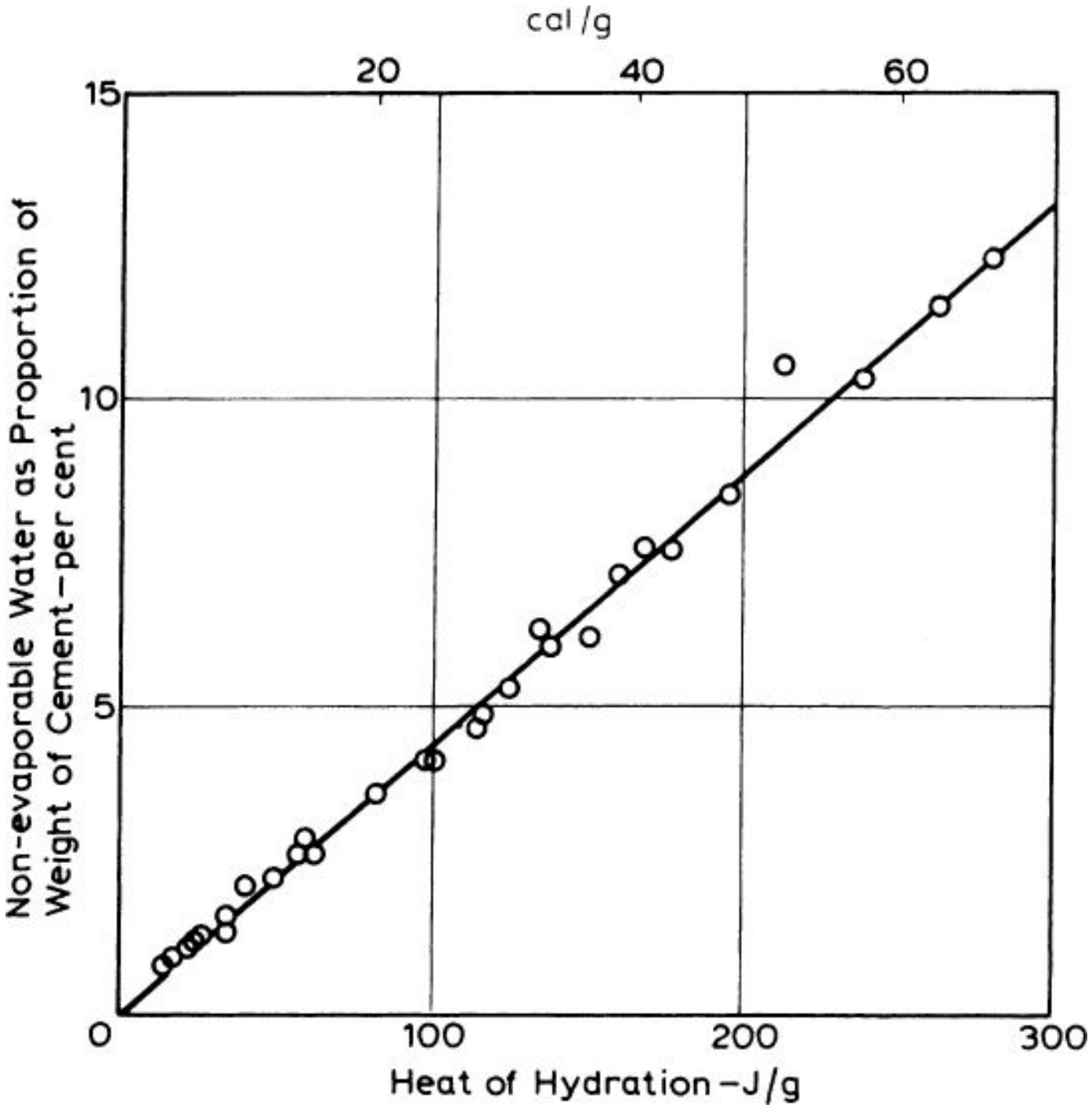


Fig. 5.3. Relation between the non-evaporable water content of cement and heat of hydration with and without a retarder[5.43]

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Great care is necessary in using retarders because, in incorrect quantities, they can totally inhibit the setting and hardening of concrete. Cases are known of seemingly inexplicable results of strength tests when sugar bags have been used for the shipment of aggregate samples to the laboratory or when molasses bags have been used to transport freshly mixed concrete. The effects of sugar depend greatly on the quantity used, and conflicting results were reported in the past.[5.6] It seems that, used in a carefully controlled manner, a small quantity of sugar (about 0.05 per cent of the mass of cement) will act as an acceptable retarder: the delay in setting of concrete is about 4 hours.[5.55] The retarding action of sugar is probably by the prevention of the formation of C-S-H.[5.50] However, the exact effects of sugar depend greatly on the chemical composition of cement. For this reason, the performance of sugar, and indeed of any retarder, should be determined by trial mixes with the actual cement which is to be used in construction.

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A large quantity of sugar, say 0.2 to 1 per cent of the mass of cement, will virtually prevent the setting of cement. Such quantities of sugar can therefore be used as an inexpensive 'kill', for instance when a mixer or an agitator has broken down and cannot be discharged. In the construction of the tunnel between England and France in the early 1990s, molasses was used to prevent setting of residual concrete as washing out underground was not possible.

When sugar is used as a controlled set retarder, the early strength of concrete is severely reduced[5.26] but, beyond about 7 days, there is an increase in strength of several per cent compared with a non-retarded mix.[5.55] This is probably due to the fact that delayed setting produces a denser hydrated cement gel (cf. p. 361).

It is interesting to note that the effectiveness of an admixture depends on the time when it is added to the mix: a delay of even 2 minutes after water has come into contact with the cement increases the retardation; sometimes, such a delay

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can be achieved by a suitable sequence of feeding the mixer. The increased retardation occurs especially with cements which have a high C_3A content because, once some C_3A has reacted with gypsum, it does not adsorb the admixture so that more of it is left to retard the hydration of the calcium silicates, which occurs through adsorption onto the calcium hydroxide nuclei.[5.36]

As retarders are frequently used in hot weather it is important to note that the retarding effect is smaller at higher temperatures (see Fig. 5.4) and some retarders cease to be effective at extremely high ambient temperatures, about 60 °C (140 °F).[5.13] Fattuhi's data[5.10] on the effectiveness of various water-reducing and set-retarding admixtures, in terms of the retardation of initial setting of concrete, are given in Table 5.2; the effect of high temperature on the final setting time is much smaller.

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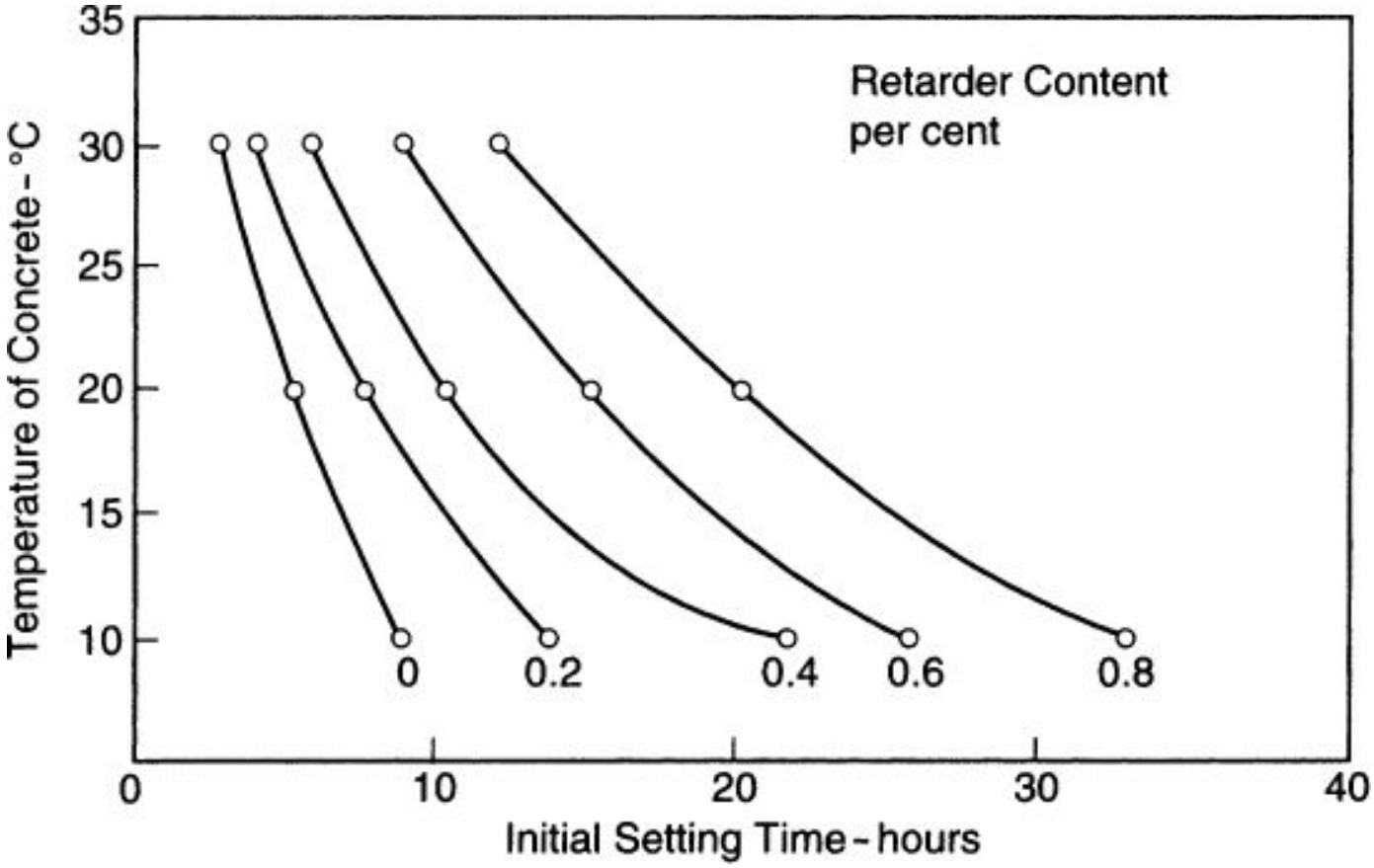


Fig. 5.4. Influence of temperature on initial setting time of concretes with various contents of retarder (by mass of cement) (cited in ref. 5.13)

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Table 5.2. Influence of Air Temperature on the Retardation of the Initial Setting Time of Concrete* by Water-reducing and Set-retarding Admixtures[5.10] (Copyright ASTM-reproduced with permission)

ASTM C 494-10 Type	*Nature of admixture*	*Retardation in initial setting time (h:min) at temperature of:*		
		30 °C (86 °F)	*40 °C (104 °F)*	*50 °C (122 °F)*
D	Sodium salt of hydroxylic acid	4:57	1:15	1:10
D	Lignin-based calcium salt	2:20	0:42	0:53
D	Calcium-lignosulfonate-based	3:37	1:07	1:25
B	Phosphate-based	—	3:20	2:30

* Measured by penetration resistance according to ASTM C 403-08.

Retarders tend to increase the plastic shrinkage because the duration of the plastic stage is extended, but drying shrinkage is not affected.[5.38]

ASTM C 494-10 requires Type B admixtures to retard the initial set by at least 1 hour but not more than $3\frac{1}{2}$ hours, as compared with a control mix. The compressive strength from the age of 3 days onwards is allowed to be 10 per cent less

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than the strength of the control concrete. The requirements of BS 5075-1 : 1982 are broadly similar. A specification for various types of admixtures is given in BS EN 934-2 : 2009 and ASTM C 494-10.

Water-reducing admixtures

According to ASTM C 494-10, admixtures which are only water-reducing are called Type A, but if the water-reducing properties are associated with retardation, the admixture is classified as Type D. There exist also water-reducing and accelerating admixtures (Type E) but these are of little interest. However, if the water-reducing admixture exhibits, as a side effect, set retardation, this can be combated by an integral incorporation of an accelerator in the mix. The most common accelerator is triethanolamine (see p. 251).

As their name implies, the function of water-reducing admixtures is to reduce the water content of the mix, usually by 5 or 10 per cent, sometimes (in concretes of very high workability) up

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to 15 per cent. Thus, the purpose of using a water-reducing admixture in a concrete mix is to allow a reduction in the water/cement ratio while retaining the desired workability or, alternatively, to improve its workability at a given water/cement ratio. Whereas aggregate which is manifestly badly graded should not be used, water-reducing admixtures improve the properties of fresh concrete made with poorly graded aggregate, e.g. a harsh mix (see pp. 165 and 747). Concrete containing a water-reducing admixture generally exhibits low segregation and good 'flowability'.

Water-reducing admixtures can also be used in pumped concrete or in concrete placed by a tremie.

The two main groups of admixtures of Type D are: (*a*) lignosulfonic acids and their salts, and (*b*) hydroxylated carboxylic acids and their salts. The modifications and derivatives of these do not act as retarders, and may even behave as acceler-

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ators[5.28] (see Fig. 5.5): they are therefore of Type A or E (see p. 246).

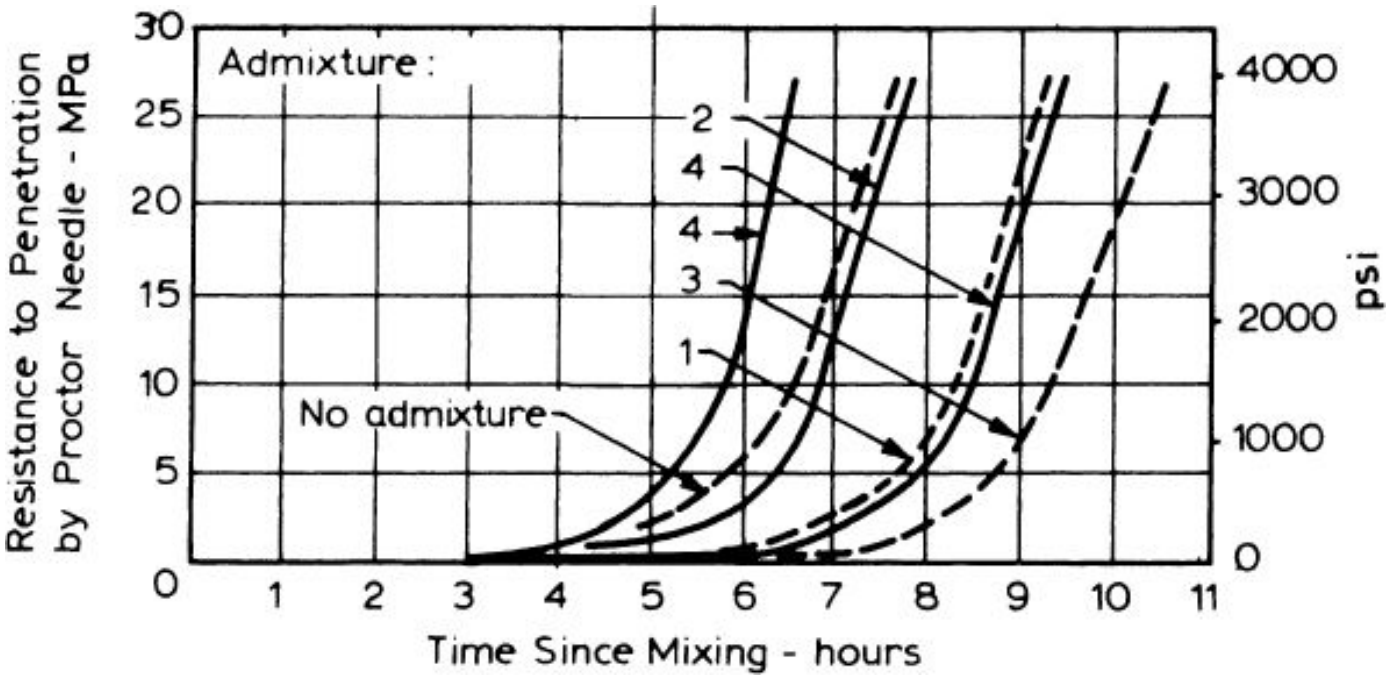


Fig. 5.5. Effect of various water-reducing admixtures on the setting time of concrete.[5.28] Numbers 1 and 2 are lignosulfonate-based; 3 and 4 are hydroxylated carboxylic acid-based

The principal active components of the admixtures are surface-active agents.[5.27] These are substances which are concentrated at the interface between two immiscible phases and which alter the physico-chemical forces acting at this interface. The substances are adsorbed on the cement

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particles, giving them a negative charge which leads to repulsion between the particles, that is to their deflocculation, and results in stabilizing their dispersion; air bubbles are also repelled and cannot attach themselves to the cement particles. Because flocculation traps some water, and also because where cement particles touch one another, their touching surfaces are not available for early hydration, water-reducing admixtures increase the surface area of cement which can undergo initial hydration and also increase the amount of water available for hydration.

In addition, the electrostatic charge causes the development around each particle of a sheath of oriented water molecules which prevent a close approach of the particles to one another. The particles have, therefore, a greater mobility, and water freed from the restraining influence of the flocculated system becomes available to lubricate the mix so that the workability is increased.[5.27] Some Type D admixtures are also adsorbed on the products of hydration of cement.

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As one effect of dispersion of cement particles, already mentioned, is to expose a greater surface area of cement to hydration, which progresses therefore at a higher rate in the early stages, there is an increase in the strength of concrete, compared with a mix of the *same* water/cement ratio but without the admixture. A more uniform distribution of the dispersed cement throughout the concrete may also contribute to a higher strength[5.27] because the process of hydration is improved. The increase in strength is particularly noticeable in very young concretes[5.29] but under certain conditions persists for a long time.

Although water-reducing admixtures affect the rate of hydration of cement, the nature of the products of hydration is unchanged[5.33] and so is the structure of the hydrated cement paste. Consequently, the use of water-reducing admixtures does not affect the resistance of concrete to freezing and thawing.[5.2] This statement is valid on condition that the water/cement ratio is not in-

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creased in conjunction with the use of the admixture. More generally, in assessing the benefits of the use of water-reducing admixtures, it is vital to use a proper base for any comparison and not simply to rely on commercial assertions. It should be noted that, even though some water-reducing admixtures may result in set retardation, they do not always reduce the rate of loss of workability with time.[5.29] A further aspect to be considered is the danger of segregation of the concrete and of bleeding.

The effectiveness of water-reducing admixtures with respect to strength varies considerably with the composition of cement, being greatest when used with cements of low alkali or low C_3A content. An example of the improvement in workability of mix with a given water content and a given dosage of lignosulfonate admixture, as a function of the C_3A content of the Portland cement used, cited by Massazza and Testolin,[5.13] is shown in Fig. 5.6.

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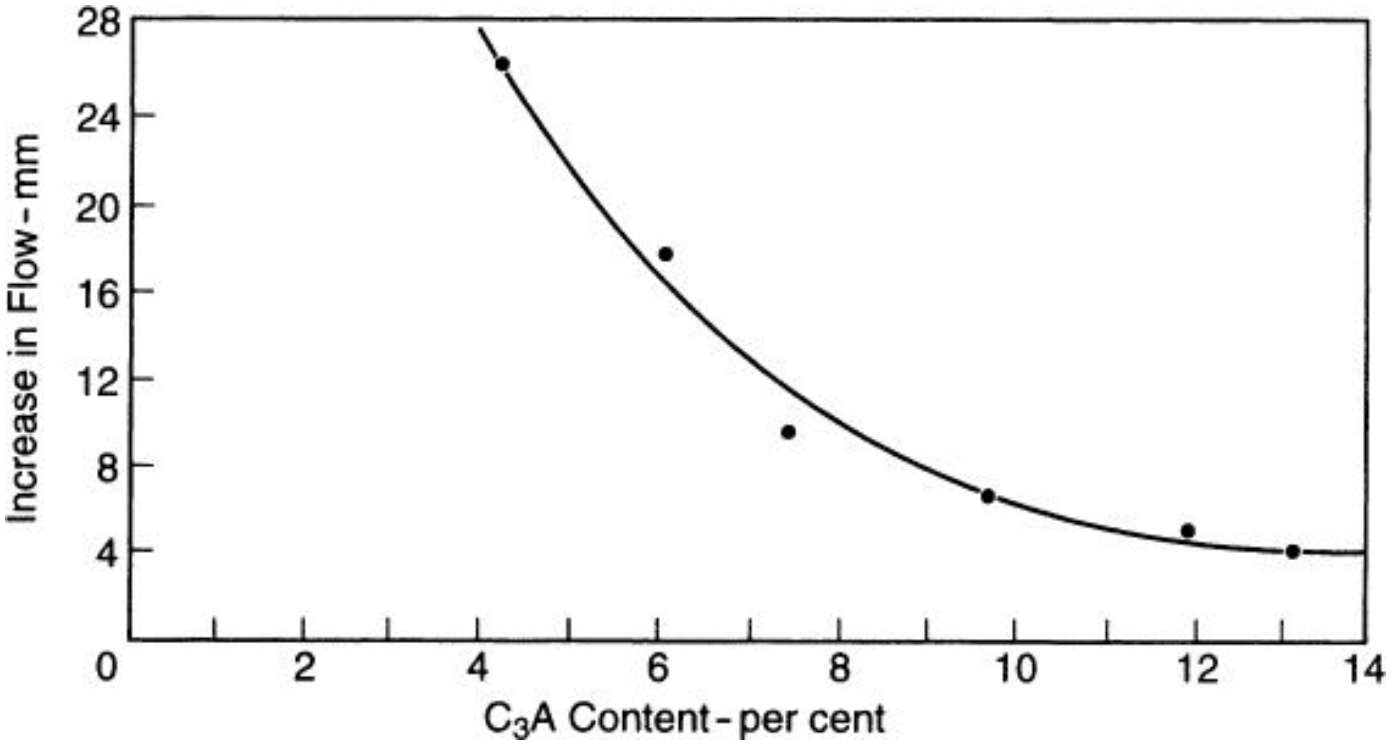


Fig. 5.6. Influence of the content of C_3A in the cement (at a constant ratio of C_3S to C_2S) on the increase in the flow of mortar (over the flow of an admixture-free mortar) at a 0.2 per cent dosage of a lignosulfonate admixture (cited in ref. 5.13)

Generally, the dosage per 100 kg of cement is lower in mixes with a high cement content. Some water-reducing admixtures are more effective when used in mixes containing pozzolanas than in Portland-cement-only mixes.

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Whereas an increased dosage of water-reducing admixture increases the workability[5.2] (see Fig. 5.7), there would be an associated considerable retardation, which is likely to be unacceptable. Long-term strength, however, is unaffected.[5.28]

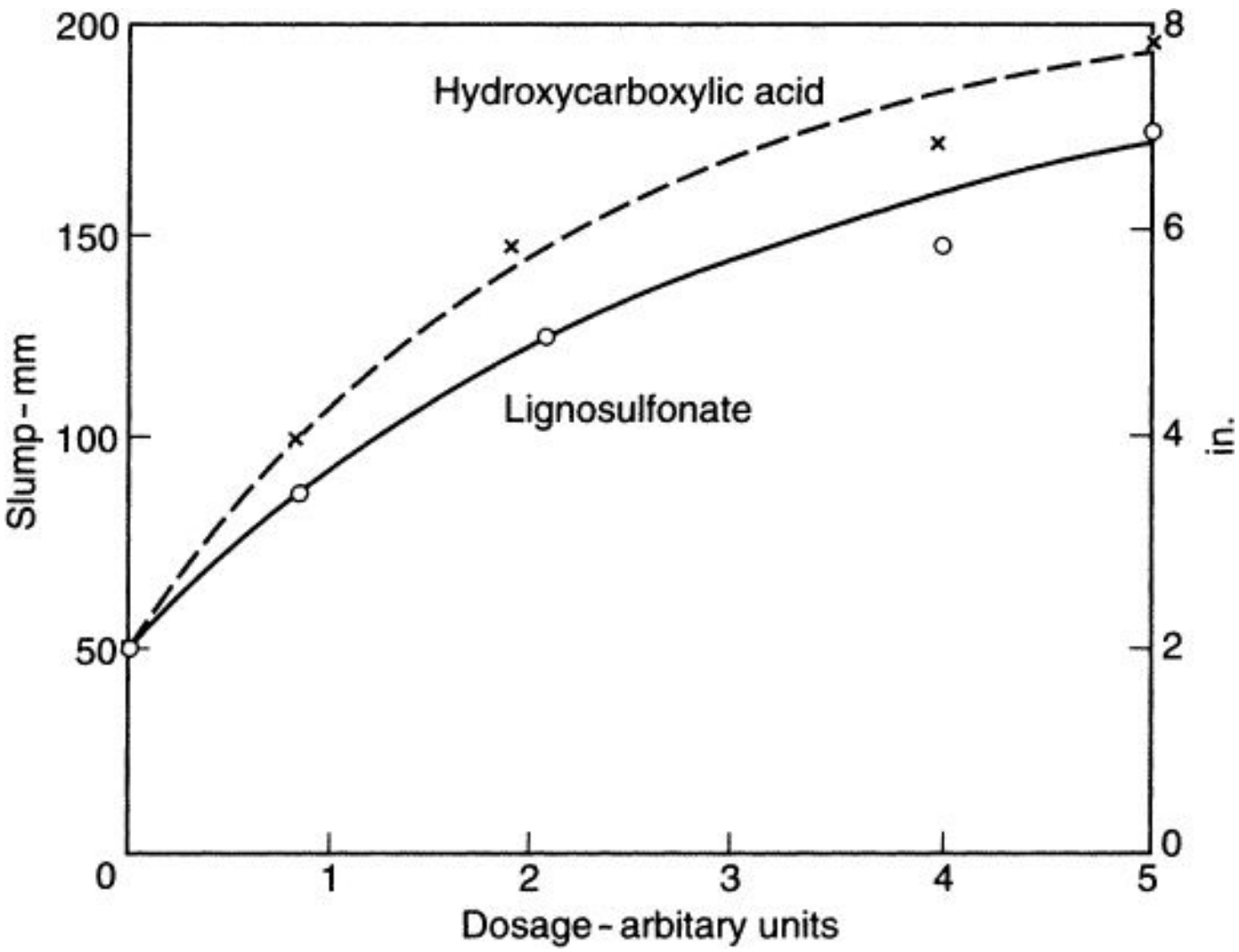


Fig. 5.7. Influence of dosage of retarders on slump (based on ref. 5.2)

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With many water-reducing admixtures, a slight delay in the introduction of the admixtures into the mix (even as low as 20 seconds from the time of contact between cement and water) enhances the performance of the admixture.

The dispersing action of a water-reducing admixture has also some effect on the dispersion of air in water[5.1] so that the admixture, especially lignosulfonate-based, may have some air-entraining effect. As this results in a reduction in the strength of concrete (see p. 561), the effect is undesirable; on the other hand, the entrained air improves the workability. Air entrainment can be counteracted by the inclusion of a small amount of a detraining agent in the water-reducing admixture; the usual agent is tributylphosphate.[5.2]

Lignosulfonate-based water-reducing admixtures increase shrinkage, but other water-reducing admixtures have been shown not to affect shrinkage.[5.13]

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With some cements, the influence of water-reducing admixtures is very small but, in general terms, admixtures are effective with all types of Portland cement and also with high-alumina cement. The actual effectiveness of any water-reducing admixture depends on the cement content, water content, type of aggregate used, presence of air-entraining agents or pozzolanas, as well as on temperature. It is, therefore, apparent that trial mixes, containing the actual materials to be used on the job, are essential in order to determine the type and quantity of admixture to achieve optimum properties: reliance on the data given by the manufacturers of admixtures is insufficient.

Superplasticizers

Superplasticizers are admixtures which are water reducing but significantly and distinctly more so than the water-reducing admixtures considered in the preceding section. Superplasticizers are also usually highly distinctive in their nature, and they make possible the production of concrete which,

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in its fresh or hardened state, is substantially different from concrete made using water-reducing admixtures of Types A, D, or E because a very low w/c or a high workability can be obtained.

For these reasons, superplasticizers are classified separately by ASTM C 494-10, and they are discussed separately in this book. ASTM C 494-10 refers to superplasticizers as "water-reducing, high range admixtures" but this name seems to be too long and too complex. On the other hand, it has to be admitted that the name 'superplasticisers' smacks of 'super'-commercialism, but it has become widely accepted and has, at least, the merit of brevity. In this book, therefore, the term superplasticizers will be used.

In the ASTM terminology, superplasticizers are referred to as Type F admixtures; when the superplasticizers are also retarding they are called Type G admixtures.

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Nature of superplasticizers

There exist four main categories of superplasticizers: sulfonated melamine-formaldehyde condensates; sulfonated naphthalene-formaldehyde condensates; modified lignosulfonates; and others such as sulfonic-acid esters and carbohydrate esters.

The first two are the most common ones. For brevity, they will be referred to as melamine-based superplasticizers and naphthalene-based superplasticizers, respectively.

Superplasticizers are water-soluble organic polymers which have to be synthesized, using a complex polymerization process, to produce long molecules of high molecular mass, and they are therefore relatively expensive. On the other hand, because they are manufactured for a specific purpose, their characteristics can be optimized in terms of length of molecules with minimum cross-linking. They also have a low content of impurities so that, even at high dosages, they do not exhibit unduly harmful side effects.

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A larger molecular mass, within limits, improves the efficiency of superplasticizers. Their chemical nature also has an effect, but no generalizations about the overall superiority of either naphthalene- or melamine-based superplasticizers is possible, probably because more than one property of a superplasticizer affects its performance and also because the chemical properties of the cement used play a role as well.[5.21]

The majority of superplasticizers are in the form of sodium salts but calcium salts are also produced; the latter, however, have a lower solubility. A consequence of the use of sodium salts is the introduction of additional alkalis into the concrete which may be relevant to the reactions of hydration of the cement and to a potential alkali–silica reaction. For this reason, the soda content of the admixtures should be known; in some countries, e.g. Germany, the content is limited to 0.02 per cent of soda by mass of cement.[5.22]

A modification of the naphthalene-based superplasticizer by the inclusion of a copolymer

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with a functional sulfonic group and carboxyl group has been developed.[5.35] This maintains the electrostatic charge on the cement particles and prevents flocculation by adsorption on the surface of cement particles. The copolymer is more active at higher temperatures, which is particularly beneficial in concreting in hot weather when high workability can be retained for up to one hour after mixing.[5.35]

When adequate information about the detailed nature of a superplasticizer is not provided, much can be learnt from specialized chemical tests.[5.15]

Physical tests make it possible readily to distinguish superplasticizers from water-reducing admixtures.[5.16]

Effects of superplasticizers

The main action of the long molecules is to wrap themselves around the cement particles and give them a highly negative charge so that they repel each other or act by steric repulsion. This results

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in deflocculation and dispersion of cement particles. The resulting improvement in workability can be exploited in two ways: by producing concrete with a very high workability or concrete with a very high strength.

At a given water/cement ratio and water content in the mix, the dispersing action of superplasticizers increases the workability of concrete, typically by raising the slump from 75 mm (3 in.) to 200 mm (8 in.), the mix remaining cohesive (see Fig. 5.8).[5.42] Even a higher slump can be achieved in self-compacting concrete. The resulting concrete can be placed with little or no compaction and is not subject to excessive bleeding or segregation. Such concrete is termed *flowing concrete* and is useful for placing in very heavily reinforced sections, in inaccessible areas, in floor slabs, and also where very rapid placing is desired. Properly compacted flowing concrete is believed to develop normal bond with reinforcement.[5.52] It should be remembered, when design-

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ing formwork, that flowing concrete can exert full hydrostatic pressure.

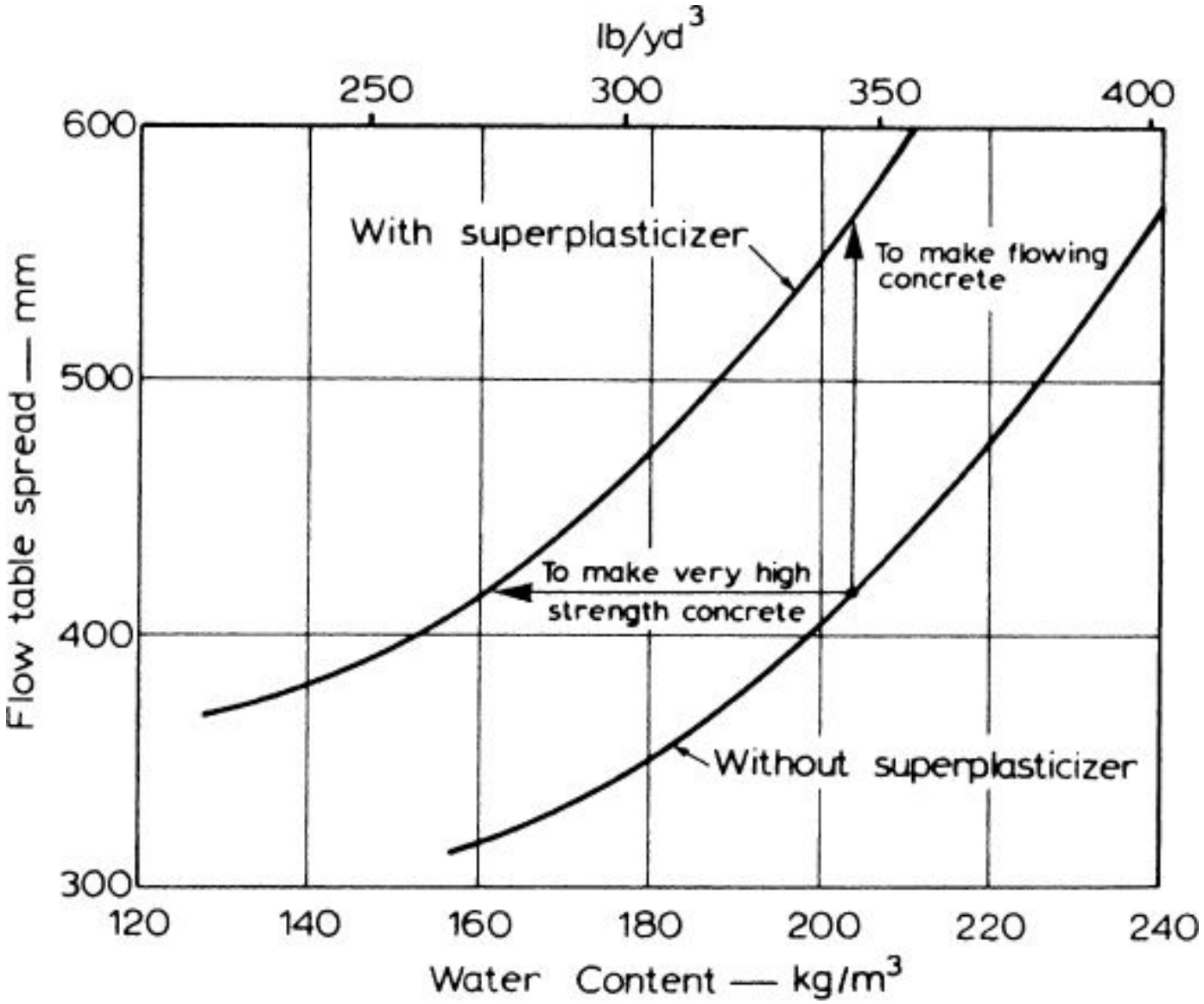


Fig. 5.8. Relation between flow table spread and water content of concrete with and without superplasticizer[5.42]

The second use of superplasticizers is in the production of concrete of normal workability but

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with an extremely high strength owing to a very substantial reduction in the water/cement ratio. Water/cement ratios down to 0.2 have been used with 28-day cylinder strengths of about 150 MPa (22 000 psi). Generally speaking, superplasticizers can reduce the water content for a given workability by 25 to 35 per cent (compared with less than half that value in the case of conventional water-reducing admixtures), and increase the 24-hour strength by 50 to 75 per cent;[5.39] an even greater increase occurs at somewhat earlier ages. Practical mixes with a cube strength of 30 MPa (4300 psi) at 7 hours have been obtained[5.39] (see Fig. 5.9). With steam-curing or high-pressure steam-curing, even higher early strengths are possible.

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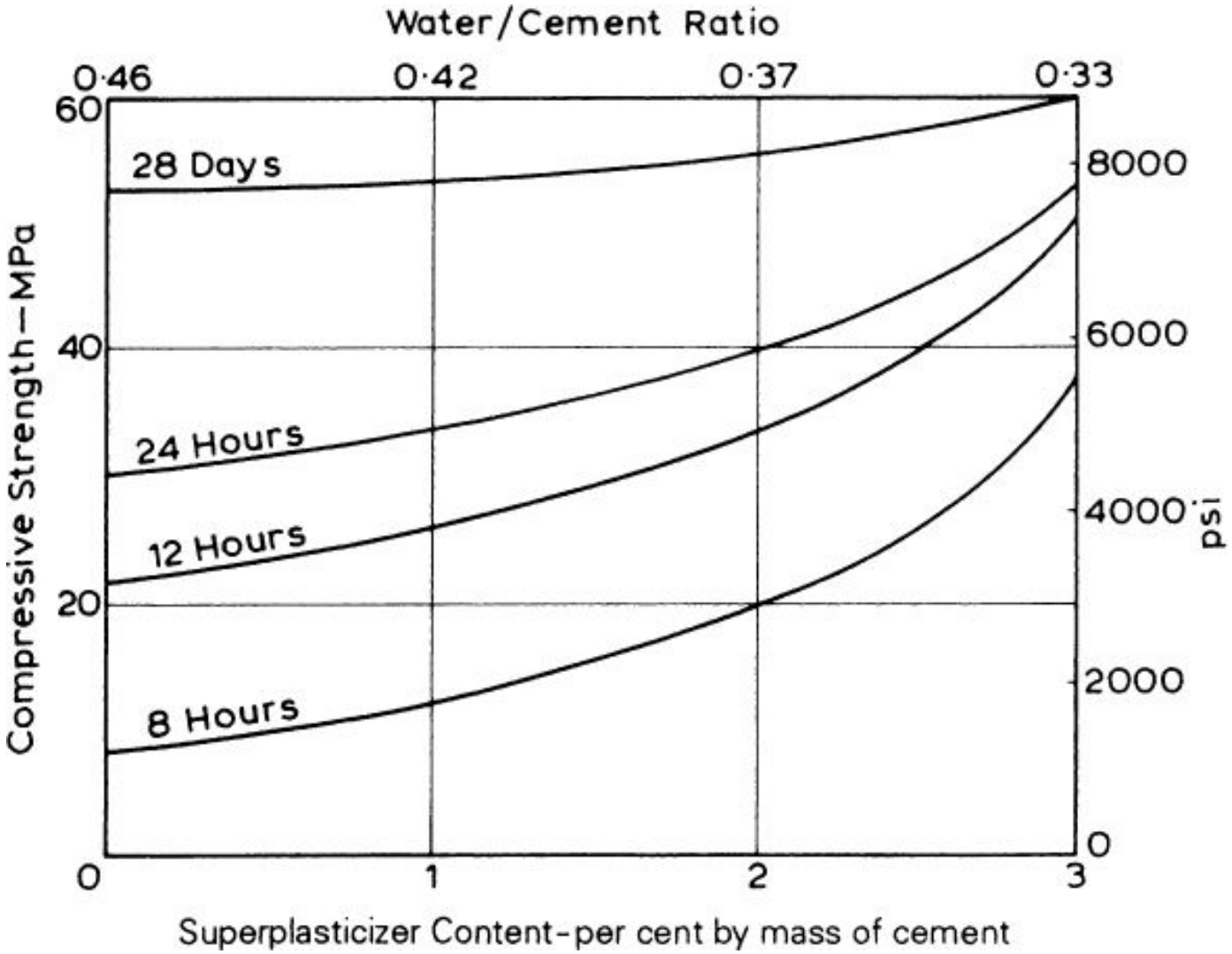


Fig. 5.9. The influence of the addition of superplasticizer on the early strength (measured on cubes) of concrete with a cement content of 370 kg/m^3 (630 lb/yd^3) and cast at room temperature. Type III cement; all concretes of the same workability[5.46]

Performance requirements for superplasticizers to produce flowing concrete and to pro-

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duce very high strength concrete are given, respectively, in ASTM C 1017-07 and ASTM C 494-10, and for both types of concrete in BS EN 934-2 : 2009. It is worth noting that the Standard requirements for improvement both in workability and in strength are greatly exceeded by the available commercial superplasticizers.

Superplasticizers do not alter fundamentally the structure of hydrated cement paste, the main effect being a better distribution of cement particles and, consequently, their better hydration. This would explain why, in some cases, the use of superplasticizers was found to increase the strength of concrete at a constant water/cement ratio. Values of a 10 per cent increase at 24 hours and a 20 per cent increase at 28 days have been quoted, but this behaviour has not been universally confirmed.[5.13]

What is important is that no retrogression of strength at long ages has ever been reported.

While the mechanism of the action of superplasticizers has not been fully explained, it is

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known that they interact with C_3A whose hydration is retarded.[5.13] A physical consequence is the formation of ettringite crystals which are small and nearly cubic in shape rather than needle-like. The cubic shape improves the mobility of the cement paste,[5.21] but is unlikely to be the main mechanism of action of superplasticizers because they also improve the workability of partially hydrated cement in which the ettringite crystals are already formed. The ultimate fate of superplasticizers is not completely known.[5.49]

Some superplasticizers do not produce appreciable set retardation, but there exist also set-retarding superplasticizers, classified by ASTM C 494-10 as Type G. In cases of naphthalene-based superplasticizers where retardation was observed, Aïtcin *et al.*[5.5] showed that this applies principally to the cement particles in the size range of 4 to 30 μm. Particles smaller than 4 μm are not affected as they are rich in SO_3 and in the alkalis; large particles undergo little initial hydration re-

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gardless of the presence or absence of a superplasticizer.[5.5]

Because superplasticizers do not significantly affect the surface tension of water, they do not entrain large amounts of air and can therefore be used at high dosages.

Dosage of superplasticizers

For increasing the workability of the mix, the normal dosage of superplasticizers is between 1 and 3 litres per cubic metre of concrete, the liquid superplasticizer containing about 40 per cent of active material. When superplasticizers are used to reduce the water content of the mix, their dosage is much higher: 5 to 20 litres per cubic metre of concrete. In the calculation of the water/cement ratio and of mix proportions in general, the volume of the liquid containing superplasticizer must be taken into account.

It is worth noting that the concentration of solids in commercial superplasticizers varies so that any comparison of performance should be

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made on the basis of the amount of solids, and not on the total mass. For practical purposes, comparison should be made on the basis of the price for a given effect.

The effectiveness of a given dosage of a superplasticizer depends on the water/cement ratio of the mix. Specifically, at a given dosage of the superplasticizer, the percentage water reduction which maintains a constant workability is much higher at low water/cement ratios than at high water/cement ratios; for example, at a water/cement ratio of 0.40, the reduction was observed to be 23 per cent, and only 11 per cent at a water/cement ratio of 0.55.[5.13]

When superplasticizers are used in very low dosages to produce high-workability normal-strength concrete, there are few problems in selecting an admixture–cement combination. At high dosages, the situation is significantly different in that the superplasticizer has to be compatible with the actual cement used, and it is not enough for the superplasticizer and the cement

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separately to conform to their respective standards. The problem of compatibility is discussed on p. 680.

Loss of workability

It is logical to assume that the first dosage of the superplasticizer must be applied soon after the cement and water have come into contact with one another. Otherwise, the initial reactions of hydration would make it impossible for the superplasticizer to effect adequate deflocculation of the cement particles. Data at variance with the preceding statement have been reported but not explained.[5.1]

The theoretical optimum time for adding a superplasticizer is what would be approximately the beginning of the dormant period without the superplasticizer. In fact, addition at that time was found to result in the highest initial workability and in the lowest rate of loss of workability.[5.30] This particular time depends on the properties of cement and would have to be established by ex-

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periment. In actual construction, it is the practicality of adding the superplasticizer that governs.

The effectiveness of superplasticizers in preventing re-agglomeration of cement particles lasts only as long as sufficient superplasticizer molecules are available to cover the exposed surface of cement particles. As some of the superplasticizer molecules become entrapped in the products of hydration of cement or react with the C_3A, the supply of superplasticizer becomes inadequate and the workability of the mix is rapidly lost. It is likely that, with prolonged mixing or agitation, some of the products of initial hydration of the cement shear off the surface of the cement particles. This enables the hydration of the hitherto unexposed cement to take place. Both the presence of the detached products of hydration and the additional hydration have the effect of reducing the workability of the mix.

An example[5.31] of the loss of workability of concrete made with a naphthalene-based superplasticizer is shown in Fig. 5.10. For comparison,

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the loss of workability of an admixture-free mix with the same initial slump is shown in the same figure. It can be seen that the loss occurs much faster with a superplasticizer but, of course, the superplasticized concrete has a lower water/cement ratio and consequently a higher strength.

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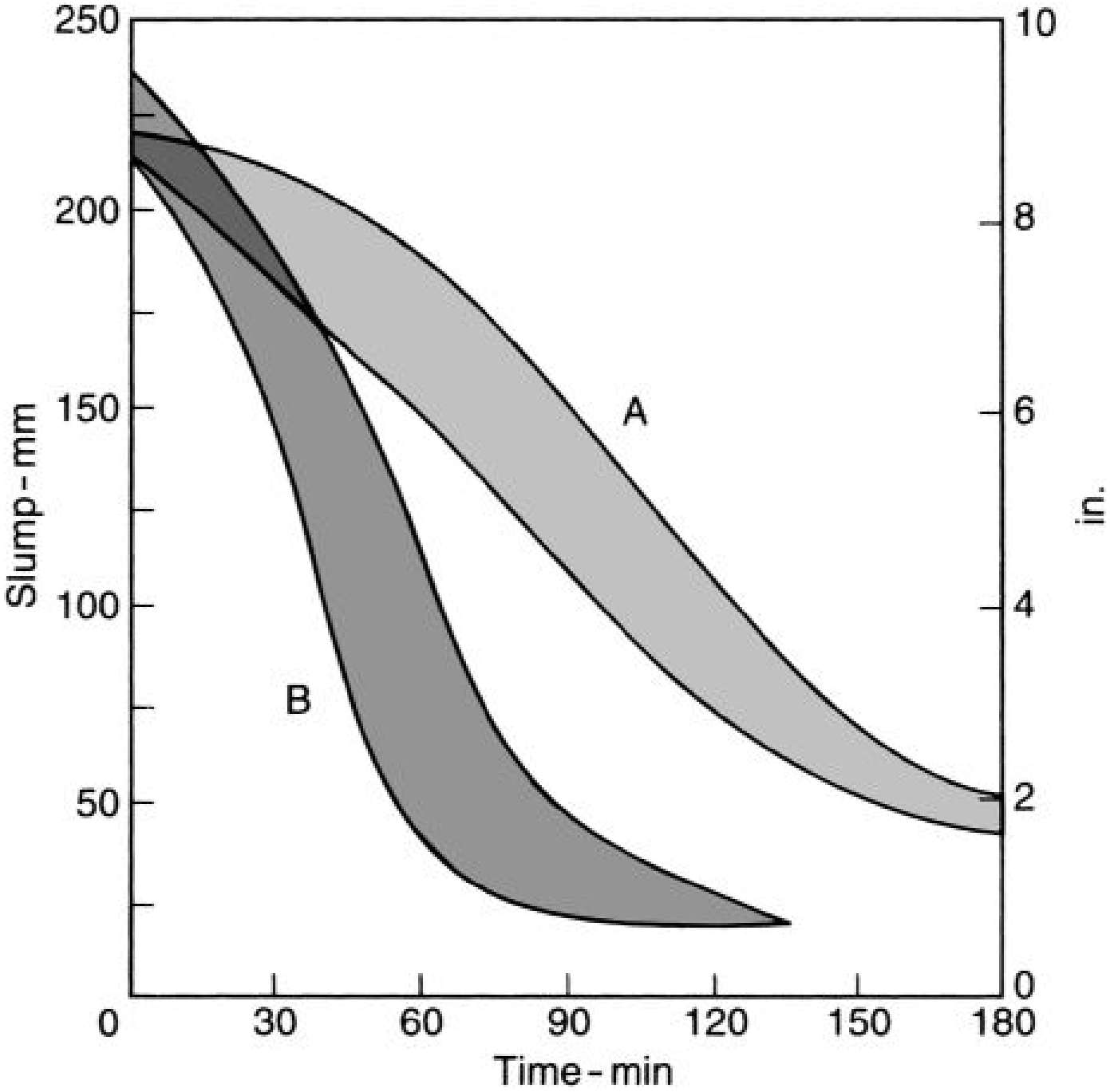


Fig. 5.10. Loss of slump with time of concretes: *(A)* water/cement ratio of 0.58 and no admixture; *(B)* water/cement ratio of 0.47 and superplasticizer (based on ref. 5.31)

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Because the effectiveness of superplasticizers is limited in duration, it may be advantageous to add the superplasticizer to the mix in two, or even three, operations. Such repeated addition, or re-dosage, is possible if an agitator truck is used to deliver the concrete to site. If the workability is to be restored by the re-dosage some time after the original mixing, the amount of superplasticizer has to be adequate to act both on the cement particles and on the products of hydration. Therefore, a high re-dosage of superplasticizer is necessary; a small re-dosage is ineffective.[5.23]

Whereas repeated addition of the superplasticizer to the mix is beneficial from the standpoint of workability, it may increase bleeding and segregation. Other possible side effects are set retardation and a change (up or down) in the amount of entrained air.[5.4] Also, the workability restored by the second dosage may decrease at a fast rate so that the re-dosage should preferably be applied immediately prior to placing and compaction of the concrete.

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An example of an effect of the application of a re-dosage of a naphthalene-based superplasticizer on workability is shown in Fig. 5.11 for a concrete with a water/cement ratio of 0.50; the initial dosage and each of the subsequent three re-dosages were the same, namely 0.4 per cent of solids by mass of cement.

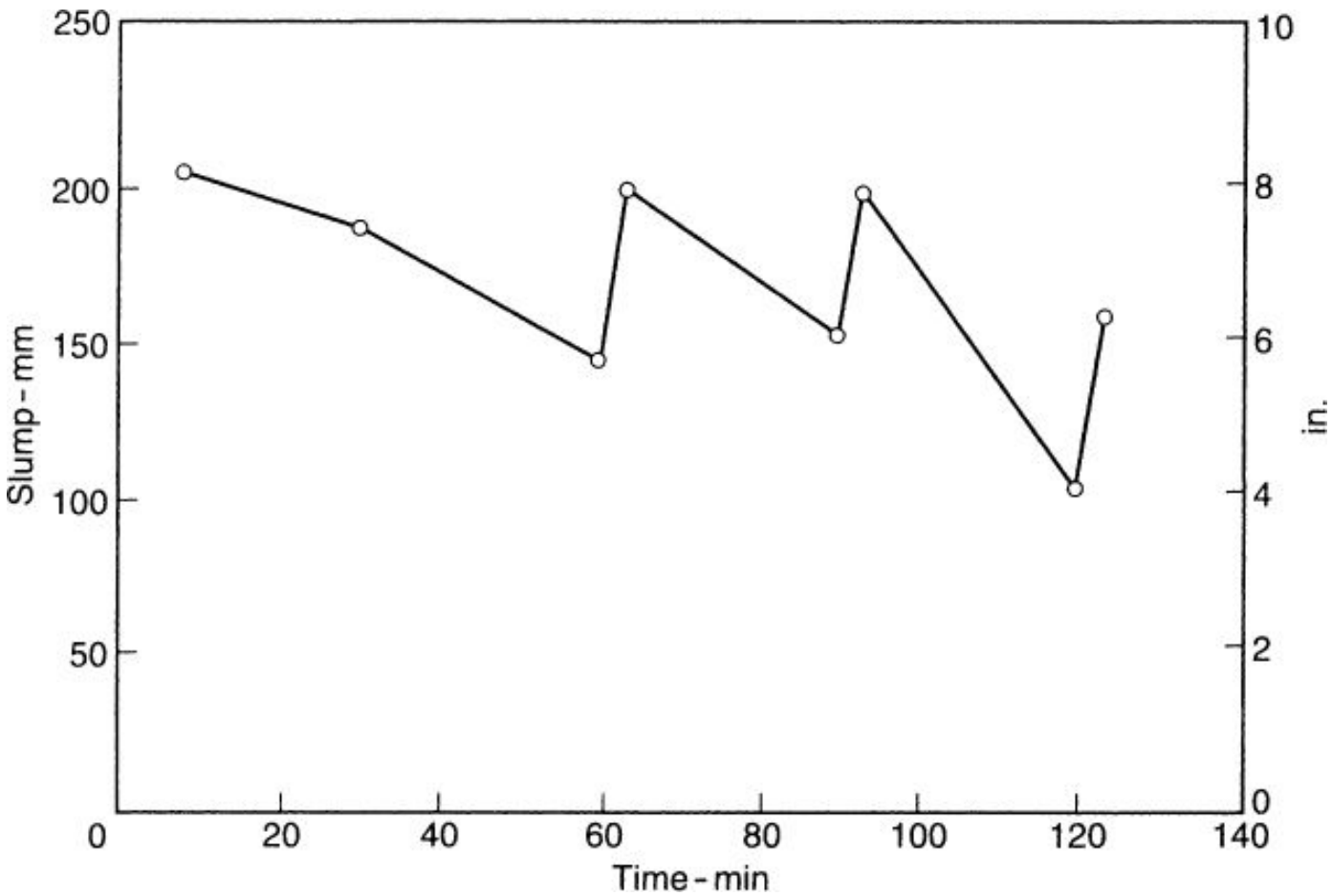


Fig. 5.11. Influence of repeated re-dosage of naphthalene-based superplasticizer on slump (based on ref. 5.1)

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The quantity of superplasticizer which needs to be added to restore the workability increases with temperature in the range of 30 to 60 °C (86 to 140 °F), and is much higher at a water/cement ratio of about 0.4 than at higher water/cement ratios. Even though the original workability is restored by a second or even a third dosage of a superplasticizer, the subsequent loss of workability becomes more rapid. However, the rate of the loss is not increased at higher temperatures.[5.18]

Nowadays, there exist superplasticizers with a long period of effectiveness so that re-dosing immediately prior to placing of concrete can be avoided. The use of such superplasticizers offers a better control of the mix proportions and is, therefore, preferable.[5.52]

Superplasticizer–cement compatibility

If a large dosage of the superplasticizer is used in order to achieve a very low water/cement ratio or if re-dosage of the superplasticizer is not possible, it is important to establish a compatible

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superplasticizer–cement combination. When the two materials are well-matched, a large single dosage can lead to the retention of high workability for a sufficiently long period: 60 to 90 minutes can be achieved and, occasionally, even 2 hours.

While assessing compatibility, the required dosage of the superplasticizer should be established. The usual approach is to determine the percentage water reduction which will result in the same workability as an admixture-free mix, using the flow-table method of ASTM C 230-08 or BS 1881-105 : 1984 (withdrawn). Alternatively, a mini-slump test developed by Kantro[5.54] can be used. Aïtcin *et al.*[5.21] favour the use of a Marsh cone for the determination of the time required for a specified volume of grout containing the given cement and superplasticizer to flow through an orifice. Generally, this time, known as Marsh flow-time, decreases with an increase in the superplasticizer dosage up to a value beyond which there is little improvement. This is the optimum dosage. Apart from reasons of eco-

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nomy, an excessive dosage of superplasticizer is undesirable as it leads to segregation. Also, there should be very little difference in workability (as measured by Marsh flow-time) at 5 and at 60 minutes after mixing. Full discussion of this topic is given on p. 680.

The laboratory determination of the superplasticizer dosage should be followed by a full-scale test but is nevertheless very valuable in rapidly verifying the suitability of a given superplasticizer with a given cement. Several properties of cement are relevant. For example, the finer the cement the higher the dosage of a superplasticizer required to obtain a given workability.[5.17] The chemical properties of cement, such as a high C_3A content (which reduces the effectiveness of a given dosage of the superplasticizer) and the nature of calcium sulfate used as a retarder, also affect the performance of superplasticizers.[5.21]

From the preceding discussion, it can be seen that a single value of dosage, sometimes recom-

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mended by the superplasticizer manufacturer, is of little value.

In searching for a suitable combination of cement and superplasticizer, it is sometimes easier to vary the superplasticizer whereas, at other times, there is a selection of cements available; what must not be assumed is that any indiscriminate combination of the two materials will be satisfactory. Reliable means of establishing compatibility of a Portland cement and a superplasticizer are available.[5.17]

Use of superplasticizers

The availability of superplasticizers has revolutionized the use of concrete in a number of ways, making it possible to place it, and to do so easily, where it was not possible to do so before. Superplasticizers make it also possible to produce concrete with significantly superior strength and other properties, now termed *high performance concrete* (see Chapter 13).

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Superplasticizers do not significantly affect the setting time of concrete except that, when used with cements having a very low C_3A content, there may be high retardation. They can be successfully used in concrete containing fly ash[5.47] and are particularly valuable when silica fume is present in the mix because that material increases the water demand of the mix.[5.32] However, if re-dosage is necessary, the quantity of the superplasticizer required is larger than when the concrete contains no silica fume.[5.19]

Superplasticizers do not influence shrinkage, creep, modulus of elasticity[5.41] or resistance to freezing and thawing.[5.40] They have no effect *per se* on the durability of concrete.[5.14] Specifically, durability on exposure to sulfates is unaffected.[5.41] The use of superplasticizers with an air-entraining admixture requires caution as sometimes the actual amount of entrained air is modified by the superplasticizer. The influence of superplasticizers on air entrainment and on the res-

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ulting resistance of concrete to freezing and thawing is considered on p. 554.

Special admixtures

In addition to the admixtures so far considered in the present chapter, there exist also admixtures for other purposes, such as air detrainment, anti-bacterial action, and waterproofing, but these are not sufficiently standardized to make reliable generalizations possible. Moreover, some of the names under which certain admixtures are sold give an exaggerated impression of their performance.

This is not to say that these admixtures are not beneficial: under many circumstances, they serve a very useful purpose, but their performance needs to be carefully established prior to use.

Waterproofing admixtures

Concrete absorbs water because surface tension in capillary pores in the hydrated cement paste

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‘pulls in’ water by capillary suction. Waterproofing admixtures aim at preventing this penetration of water into concrete. Their performance is very much dependent on whether the applied water pressure is low, as in the case of rain (other than driven by wind) or capillary rise, or whether a hydrostatic pressure is applied, as in the case of water-retaining structures or structures such as basements in waterlogged ground. The term ‘waterproofing’ is therefore of dubious validity.

Waterproofing admixtures may act in several ways but their effect is mainly to make concrete hydrophobic. By this is meant an increase in the contact angle between the walls of the capillary pores and water, so that water is ‘pushed out’ of the pores.

One action of waterproofing admixtures is through reaction with the calcium hydroxide in hydrated cement paste; examples of products used are stearic acid and some vegetable and animal fats. The effect is to make the concrete hydrophobic.

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Another action of waterproofing admixtures is through coalescence on contact with the hydrated cement paste which, because of its alkalinity, breaks down the ‘waterproofing’ emulsion; an example is an emulsion of very finely divided wax. The effect here, too, is to make the concrete hydrophobic.

The third type of waterproofing admixture is in the form of very fine material containing calcium stearate or some hydrocarbon resins or coal tar pitches which produce hydrophobic surfaces.[5.2]

While imparting hydrophobic properties to concrete is valuable, in practice, complete coating of all surfaces of capillary pores is difficult to attain, with the consequence that full waterproofing is unlikely to be achieved.[5.3]

Some waterproofing admixtures, in addition to their hydrophobic action, also effect pore blocking through a coalescent component. Unfortunately, little information is available to make

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it possible to explain and classify the actions involved so that reliance has to be based on the manufacturers' data coupled with experimental evidence on the performance of any particular waterproofing admixture. It should be stressed that the experience should be over a sufficiently long period to demonstrate the stability of the waterproofing admixture.

A side effect of some waterproofing admixtures is to improve the workability of the mix owing to the presence of finely divided wax or bituminous emulsions, which entrain some air. They also improve cohesion of the concrete but may result in a 'sticky' mix.[5.3]

Because of the nature of the waterproofing admixtures, they are not effective in resisting attack by aggressive gases.[5.2]

A final point to be made about waterproofing admixtures is that, because their exact composition is often unknown, it is vital to ascertain that they contain no chlorides if the concrete is likely

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to be used in a situation which is sensitive to chloride-induced corrosion.

Waterproofing admixtures should be distinguished from *water repellents*, based on silicone resins, which are applied to the concrete surface. *Waterproof membranes* are emulsion-based bitumen coatings, possibly with rubber latex, which produce a tough film with some degree of elasticity. Consideration of these materials is outside the scope of this book.

Anti-bacterial and similar admixtures

Some organisms such as bacteria, fungi or insects can adversely affect concrete. The possible mechanisms[5.3] are: releasing corrosive chemicals through metabolic action, and creation of an environment which promotes corrosion of steel. Staining of the surface can also result.

The usual agent in bacterial attack is an organic or mineral acid which reacts with hydrated cement paste. Initially, the alkaline pore water in hydrated cement paste neutralizes the acid but

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continuing action of bacteria results in deeper attack.

Because the rough surface texture of concrete shelters the bacteria, surface cleaning is ineffective, and it is necessary to incorporate in the mix some special admixtures which are toxic to the attacking organisms: these may be anti-bacterial, fungicidal or insecticidal.

Fuller details of bacterial attack are given by Ramachandran.[5.3] Useful information about anti-bacterial admixtures is given in ACI 212.3R-91,[5.4] which lists some effective admixtures. It can be added that copper sulfate and pentachlorophenol have been found to control the growth of algae or lichen on hardened concrete but their effectiveness is lost with time.[5.48] Clearly, admixtures which may prove toxic should not be used.

Conversely, some bacteria introduced into the concrete mix may heal cracks by way of precipitating calcite. These bacteria are spore-forming

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and alkali resistant.[5.56] Laboratory experiments demonstrated successful healing of cracks when water enters the cracks. However, a substrate of organic carbon is necessary, but such carbon in the mix may be deleterious. Thus further studies are necessary. Also, there is a question of cost of incorporation of the bacteria in the mix.

Remarks about the use of admixtures

Admixtures whose performance is known from experience at normal ambient temperatures may behave differently at very high or very low temperatures.

Some admixtures do not tolerate exposure to freezing temperatures while stored and become unusable; most of the others require thawing and remixing. Very few are unaffected by freezing temperatures.

Admixtures, whose performance when used separately is known, may not be compatible when used together; for this reason, it is essential to use trial mixes for any combination of admixtures.

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Even if two admixtures are compatible when introduced into the mix, they may interact adversely if they come into contact with one another prior to being introduced into the mixer. This is, for example, the case with the combination of a water-reducing admixture of the lignosulfonate type and an air-entraining admixture of a vinsol resin-based type.[5.29] In consequence, it is a wise precaution to discharge the various admixtures into the mixer separately and at different locations, and possibly also at different times. Details of admixture batching systems are given in ACI 212.3R-1991.[5.4]

When being discharged into the mixer, admixtures have to be not only accurately metered, but it is also important that they be discharged in the correct part of the mixing cycle and at the correct rate. Changes in concrete mixing procedure can affect the performance of admixtures.

It is important to know whether any admixture to be used contains chlorides because, generally, there is specified a limit on the *total* chloride ion

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content in the concrete mix so that all sources of chlorides have to be taken into account (see Chapter 11). Even the so-called 'chloride-free' admixtures may contain small amounts of chloride ions originating from the water used in the manufacture of the admixture. When there is high sensitivity to the chloride content of the concrete, for instance for use in prestressed concrete, the exact chloride content of the admixture to be used should be ascertained.[5.4]

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Chapter 6. Strength of concrete

Strength of concrete is commonly considered its most valuable property, although, in many practical cases, other characteristics, such as durability and permeability, may in fact be more important. Nevertheless, strength usually gives an overall picture of the quality of concrete because strength is directly related to the structure of the hydrated cement paste. Moreover, the strength of concrete is almost invariably a vital element of structural design and is specified for compliance purposes.

The mechanical strength of cement gel was discussed on p. 34; in this chapter some empirical relations concerning the strength of concrete will be discussed.

Water/cement ratio

In engineering practice, the strength of concrete at a given age and cured in water at a prescribed

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temperature is assumed to depend primarily on two factors only: the water/cement ratio and the degree of compaction. The influence of air voids on strength was discussed on p. 187, and at this stage we shall consider fully-compacted concrete only: for mix proportioning purposes, this is taken to mean that the hardened concrete contains about 1 per cent of air voids.

When concrete is fully compacted, its strength is taken to be inversely proportional to the water/cement ratio. This relation was preceded by a so-called 'law', but really a rule, established by Duff Abrams in 1919. He found strength to be equal to:

$$f_c = \frac{K_1}{K_2^{w/c}}$$

where *w/c* represents the water/cement ratio of the mix (originally taken by volume), and K_1 and K_2 are empirical constants. The general form of the strength versus water/cement ratio curve is shown in Fig. 6.1.

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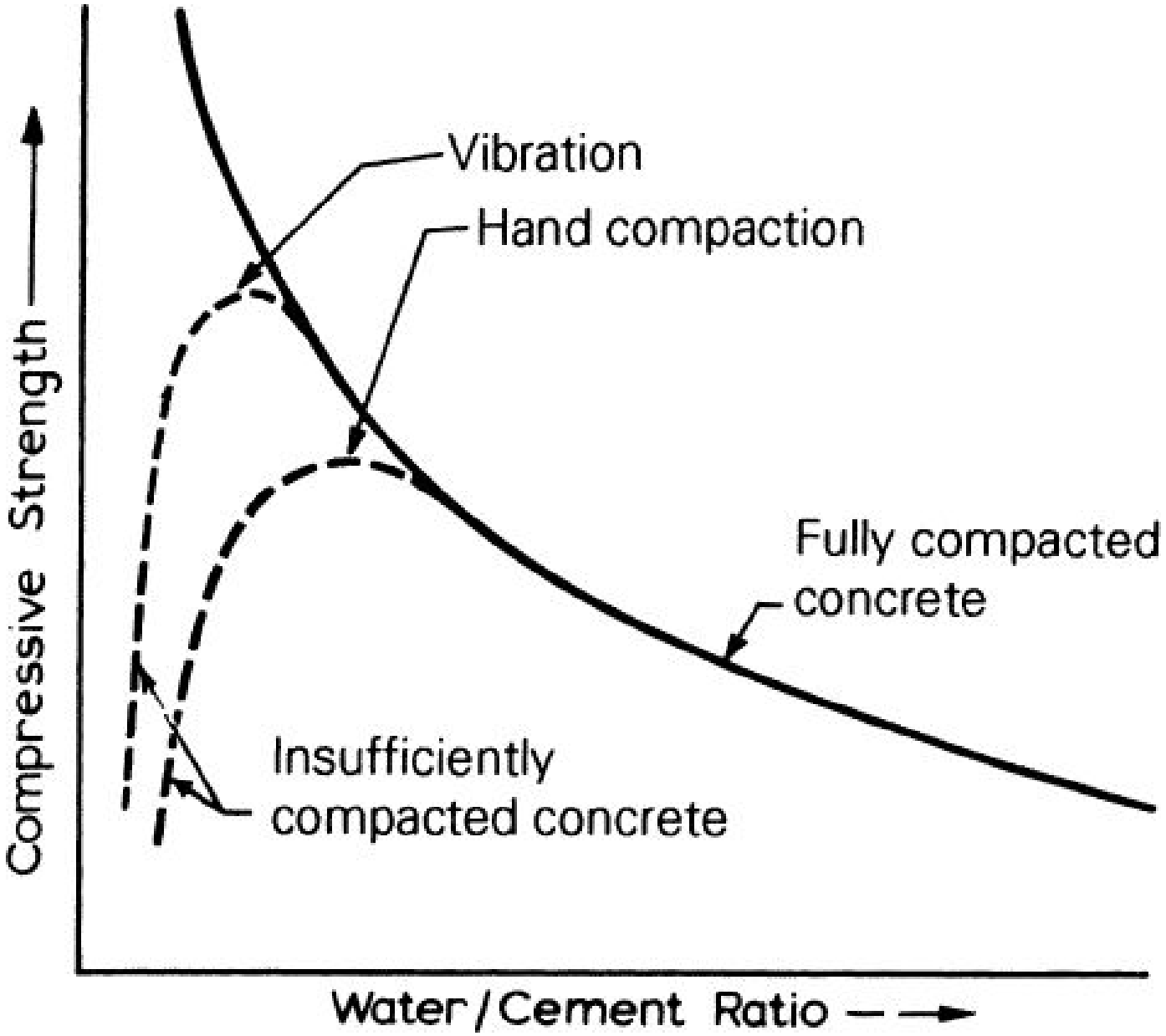


Fig. 6.1. The relation between strength and water/cement ratio of concrete

Abrams' rule, although established independently, is similar to a general rule formulated by René Féret in 1896 in that they both relate strength of concrete to the volumes of water and cement. Féret's rule was in the form:

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$$f_c = K\left(\frac{c}{c + w + a}\right)^2$$

where f_c is the strength of concrete, c, w and a are the absolute volumetric proportions of cement, water, and air, respectively, and K is a constant.

It may be recalled that the water/cement ratio determines the porosity of the hardened cement paste at any stage of hydration (see p. 30). Thus the water/cement ratio and the degree of compaction both affect the volume of voids in concrete, and this is why the volume of air in concrete is included in Féret's expression.

The relation between strength and the volume of voids will be discussed more fully in a later section. At this stage, we are concerned with the usual practical relation between strength and the water/cement ratio. Figure 6.1 shows that the range of the validity of the water/cement ratio rule is limited. At very low values of the water/cement ratio, the curve ceases to be followed

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when full compaction is no longer possible; the actual position of the point of departure depends on the means of compaction available. It seems also that mixes with a very low water/cement ratio and an extremely high cement content (probably above 530 kg/m^3 (900 lb/yd^3)) exhibit retrogression of strength when large-size aggregate is used. Thus, at later ages, in this type of mix, a lower water/cement ratio would not lead to a higher strength. This behaviour may be due to stresses induced by shrinkage, whose restraint by aggregate particles causes cracking of the cement paste or a loss of the cement–aggregate bond.[6.2]

From time to time, the water/cement ratio rule has been criticized as not being sufficiently fundamental. Nevertheless, *in practice* the water/cement ratio is the largest single factor in the strength of fully compacted concrete. Perhaps the best statement of the situation is that by Gilkey:[6.74]

"For a given cement and acceptable aggregates, the strength that may be developed by a

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workable, properly placed mixture of cement, aggregate, and water (under the same mixing, curing, and testing conditions) is influenced by the:

(*a*) ratio of cement to mixing water

(*b*) ratio of cement to aggregate

(*c*) grading, surface texture, shape, strength, and stiffness of aggregate particles

(*d*) maximum size of the aggregate."

We can add that factors (*b*) to (*d*) are of lesser importance than factor (*a*) when usual aggregates up to 40 mm ($1\frac{1}{2}$ in.) maximum size are employed. Those factors are, nevertheless, present because, as pointed out by Walker and Bloem,[6.74] "the strength of concrete results from: (1) the strength of the mortar; (2) the bond between the mortar and the coarse aggregate; and (3) the strength of the coarse aggregate particle, i.e. its ability to resist the stresses applied to it".

Figure 6.2 shows that the graph of strength versus water/cement ratio is approximately in the shape of a hyperbola. This applies to concrete

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made with any given type of aggregate and at any given age. It is a geometrical property of a hyperbola $y = k/x$ that y against $1/x$ plots as a straight line. Thus, the relation between the strength and the *cement/water* ratio is approximately linear in the range of cement/water ratios between about 1.2 and 2.5. This linear relationship, first suggested in ref. 6.4, has been confirmed by Alexander and Ivanusec[6.112] and by Kakizaki *et al.*[6.58] It is clearly more convenient to use than the water/cement ratio curve, particularly when interpolation is desired. Figure 6.3 shows the data of Fig. 6.2 plotted with the cement/water ratio as abscissa. The values used apply to the given cement only, and in any practical case the actual relation between strength and cement/water ratio has to be determined.

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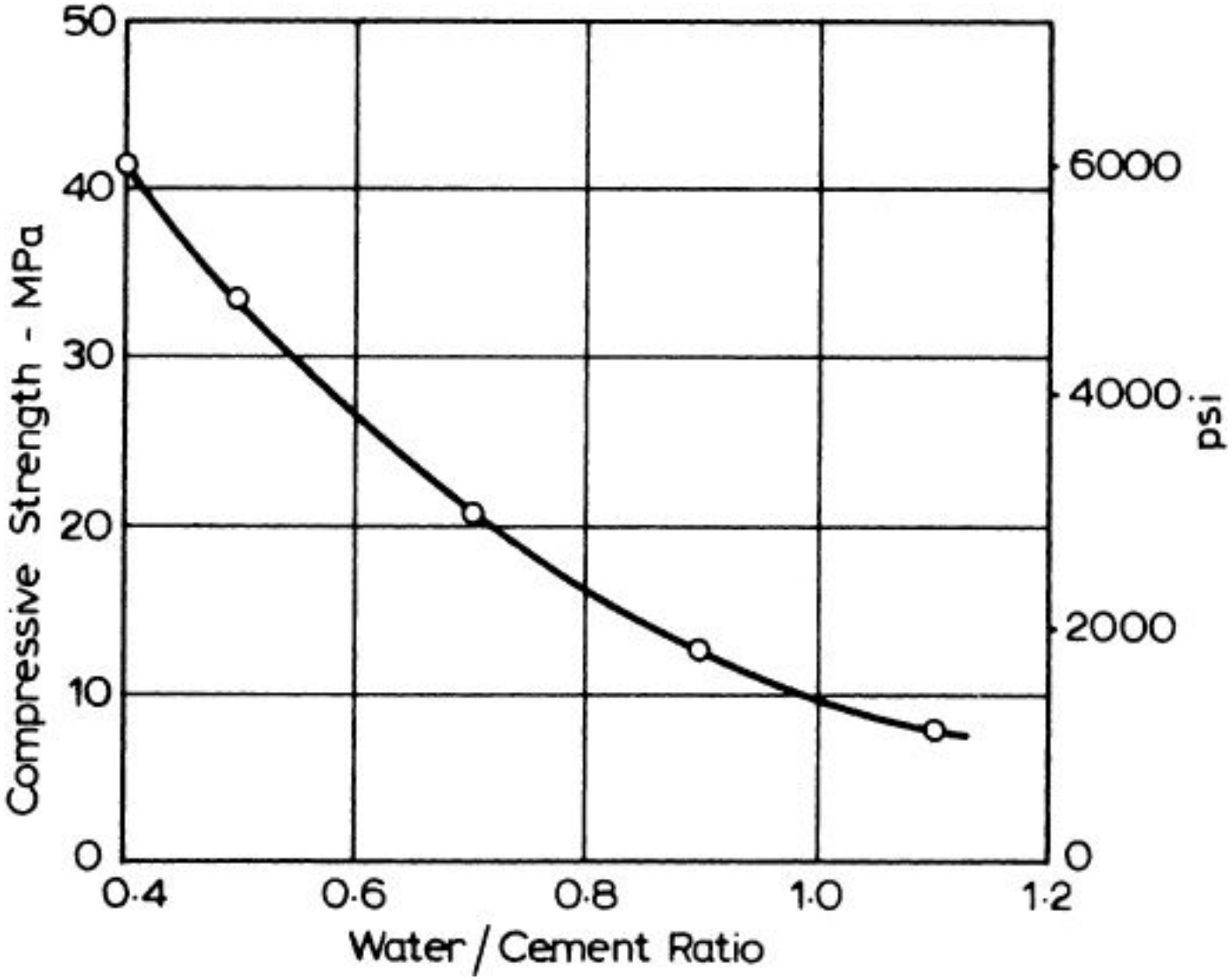


Fig. 6.2. Relation between 7-day strength and water/cement ratio for concrete made with a rapid-hardening Portland cement

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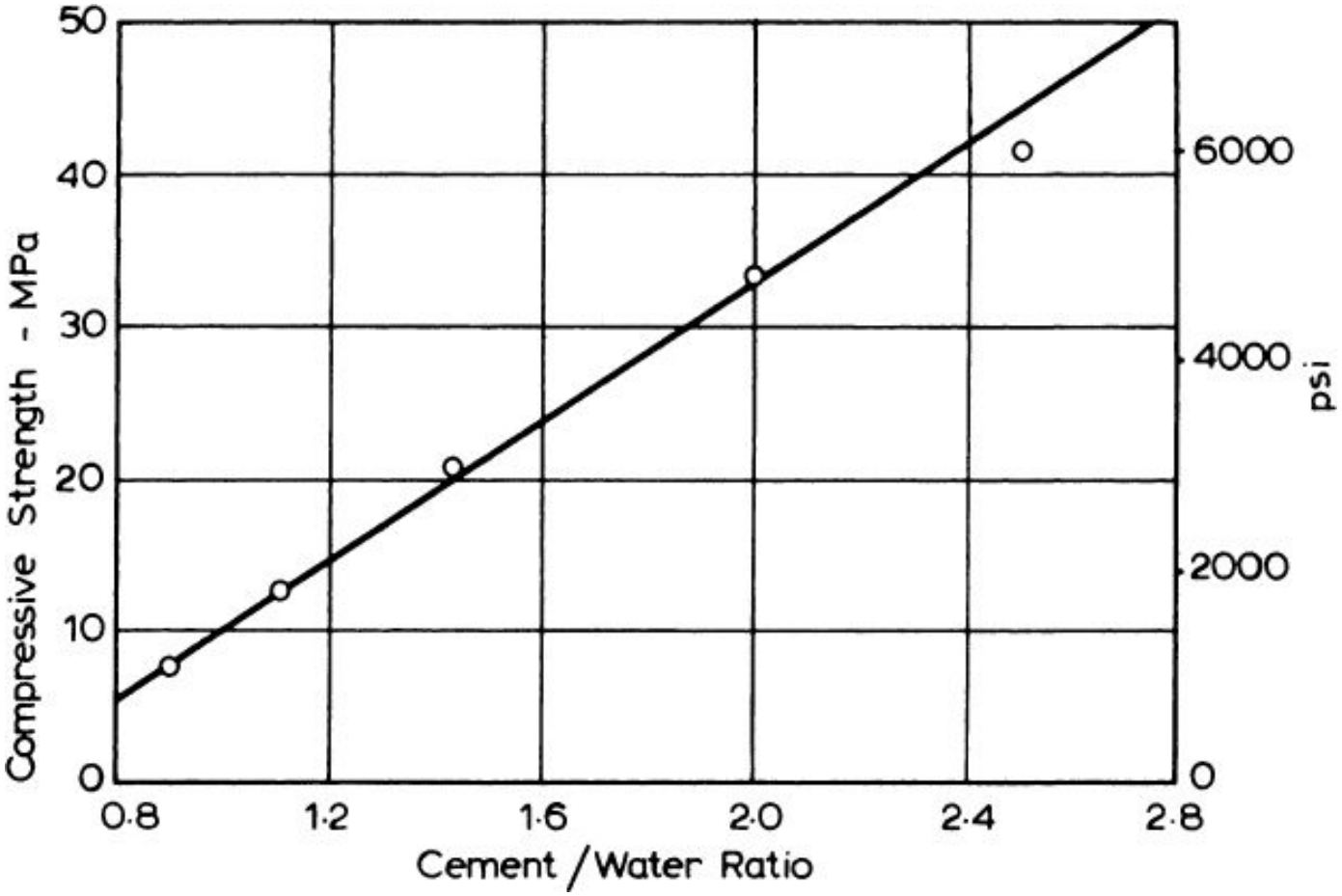


Fig. 6.3. A plot of strength against cement/water ratio for the data of Fig. 6.2

The linearity of the relation between strength and cement/water ratio does not extend beyond the cement/water ratio of 2.6, which corresponds to the water/cement ratio of 0.38. In fact, for cement/water ratios larger than 2.6, there exists a different, but still linear, relation with strength,[6.59] as shown in Fig. 6.4. This figure represents calculated values for cement pastes which

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have achieved maximum possible hydration. For water/cement ratios smaller than 0.38, the maximum possible hydration is less than 100 per cent (see p. 27); consequently, the slope of the curve is different from that for higher values of the water/cement ratio. This observation is worth remembering as nowadays mixes with water/cement ratios both somewhat above and somewhat below 0.38 are often used.

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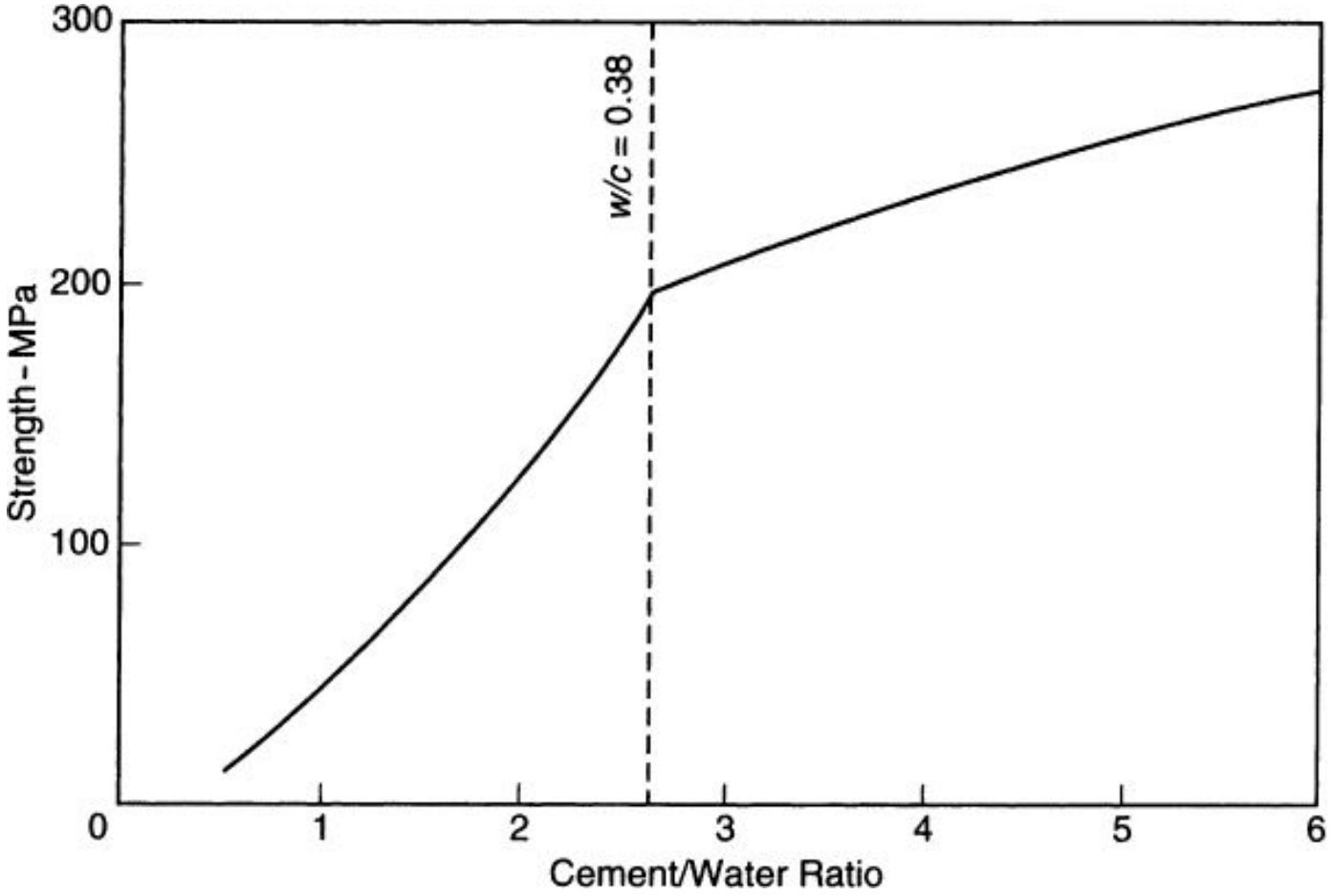


Fig. 6.4. Relation between calculated strength of neat cement paste and cement/water ratio. Maximum possible hydration is assumed to have taken place (based on ref. 6.59)

The pattern of strength of high-alumina cement concrete is somewhat different from that of concrete made with Portland cement, in that strength increases with the cement/water ratio at a progressively decreasing rate.[6.4]

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It must be admitted that the relations discussed here are not precise, and other approximations can be made. For instance, it has been suggested that, as an approximation, the relation between the logarithm of strength and the natural value of the water/cement ratio can be assumed to be linear[6.3] (cf. Abrams' expression). As an illustration, Fig. 6.5 gives the relative strength of mixes with different water/cement ratios, taking the strength at the water/cement ratio of 0.4 as unity.

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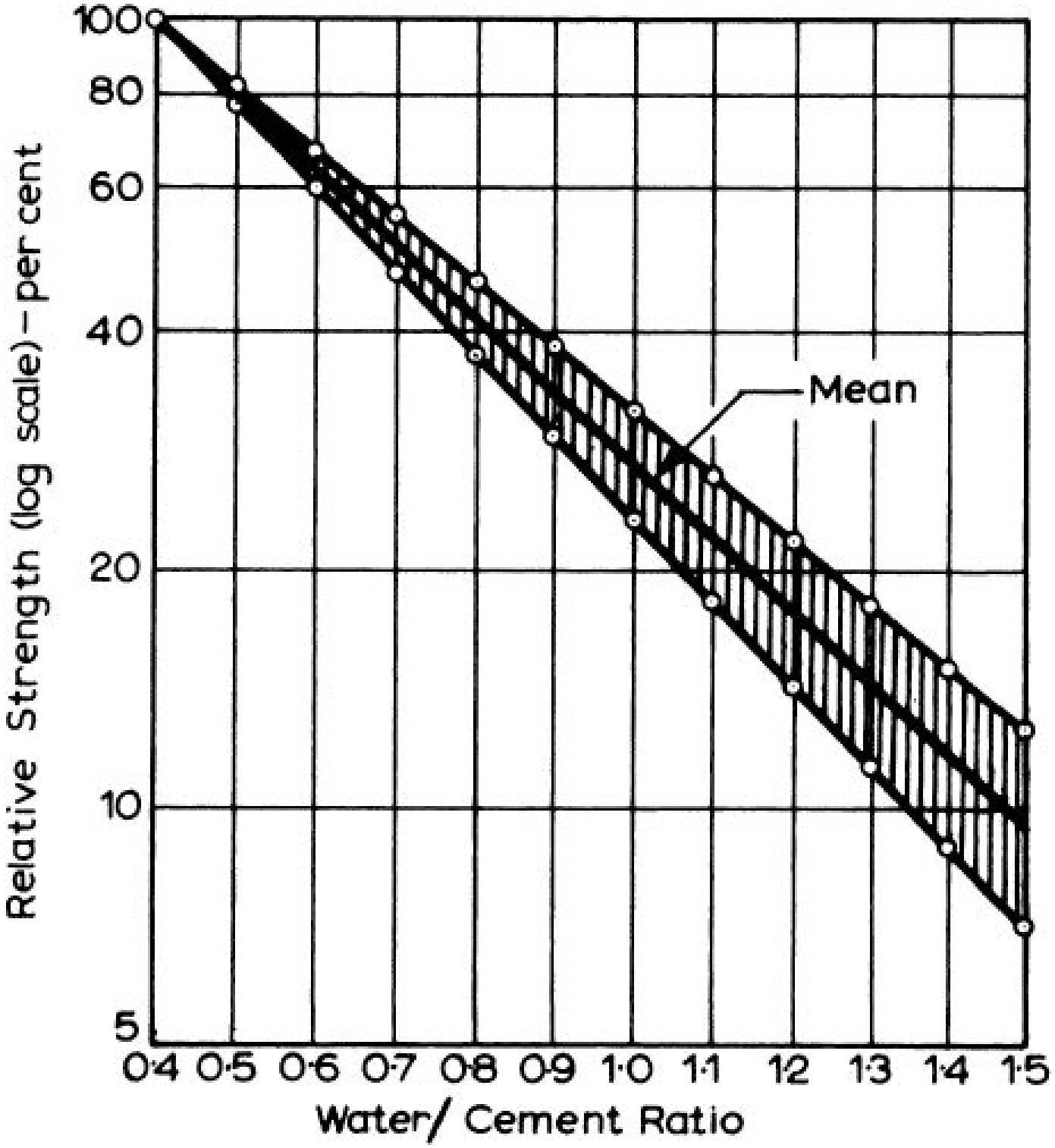


Fig. 6.5. Relation between logarithm of strength and water/cement ratio[6.3]

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Effective water in the mix

The practical relations discussed so far involve the quantity of water in the mix. This needs a more careful definition. We consider as effective that water which occupies space outside the aggregate particles when the gross volume of concrete becomes stabilized, i.e. approximately at the time of setting. Hence the terms *effective*, *free*, or *net* water/cement ratio.

Generally, water in concrete consists of that added to the mix and that held by the aggregate at the time when it enters the mixers. A part of the latter water is absorbed within the pore structure of the aggregate (see p. 129) while some exists as free water on the surface of the aggregate and is therefore no different from the water added direct into the mixer. Conversely, when the aggregate is not saturated and some of its pores are therefore air-filled, a part of the water added to the mix will be absorbed by the aggregate during the first half-hour or so after mixing. Under

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such circumstances the demarcation between absorbed and free water is a little difficult.

On a site, the aggregate is as a rule wet, and the water in excess of that required for the aggregate to be in a saturated and surface-dry condition is considered to be the effective water of the mix. For this reason, the mix proportioning data are based usually on the water in excess of that absorbed by the aggregate, that is the free water. On the other hand, some laboratory tests refer to the *total* water added to a dry aggregate. Care is, therefore, necessary in translating laboratory results into mix proportions to be used on a site, and all reference to water/cement ratio must make it clear if total rather than free water is considered.

Gel/space ratio

The influence of the water/cement ratio on strength does not truly constitute a law because the water/cement ratio rule does not include many qualifications necessary for its validity. In particular, strength at any water/cement ratio de-

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pends on: the degree of hydration of cement and its chemical and physical properties; the temperature at which hydration takes place; the air content of the concrete; and also the change in the effective water/cement ratio and the formation of cracks due to bleeding.[6.5] The cement content of the mix and the properties of the aggregate–cement paste interface are also relevant.

It is more correct, therefore, to relate strength to the concentration of the solid products of hydration of cement in the space available for these products; in this connection it may be relevant to refer again to Fig. 1.10. Powers[6.6] has determined the relation between the strength development and the gel/space ratio. This ratio is defined as the ratio of the volume of the hydrated cement paste to the sum of the volumes of the hydrated cement and of the capillary pores.

On p. 27, it was shown that cement hydrates to occupy more than twice its original volume; in the following calculations the products of hydration of 1 ml of cement will be assumed to occupy

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2.06 ml; not all the hydrated material is gel, but as an approximation we can consider it as such. Let

c = mass of cement

v_c = specific volume of cement, that is volume of unit mass

w_o = volume of mixing water, and

α = the fraction of cement that has hydrated.

Then,[6.7] the volume of gel is $2.06cv_c\alpha$, and the total space available to the gel is $cv_c\alpha + w_o$. Hence, the gel/space ratio is

$$r = \frac{2.06v_c\alpha}{v_c\alpha + \dfrac{w_o}{c}}.$$

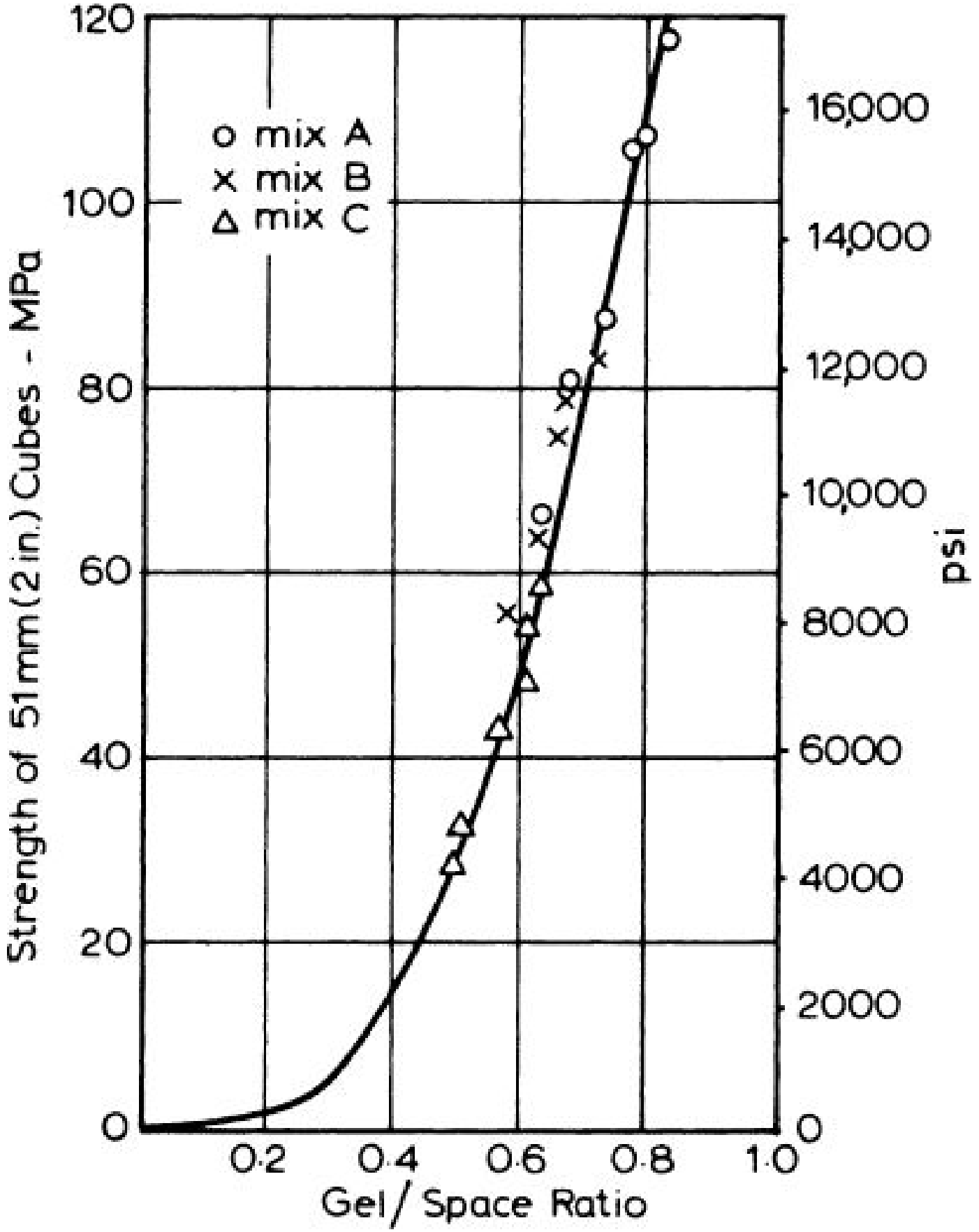
Taking the specific volume of dry cement as 0.319 ml/g, the gel/space ratio becomes:

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$$r = \frac{0.657\alpha}{0.319\alpha + \dfrac{w_o}{c}}.$$

The compressive strength of concrete tested by Powers[6.7] was found to be $234r^3$ MPa ($34\,000r^3$ psi), and is independent of the age of the concrete or its mix proportions. The actual relation between the compressive strength of mortar and the gel/space ratio is shown in Fig. 6.6: it can be seen that strength is approximately proportional to the cube of the gel/space ratio, and the figure 234 MPa (34 000 psi) represents the intrinsic strength of gel for the type of cement and of specimen used.[6.8] Numerical values differ little for the usual range of Portland cements except that a higher C_3A content leads to a lower strength at a given gel/space ratio.[6.5]

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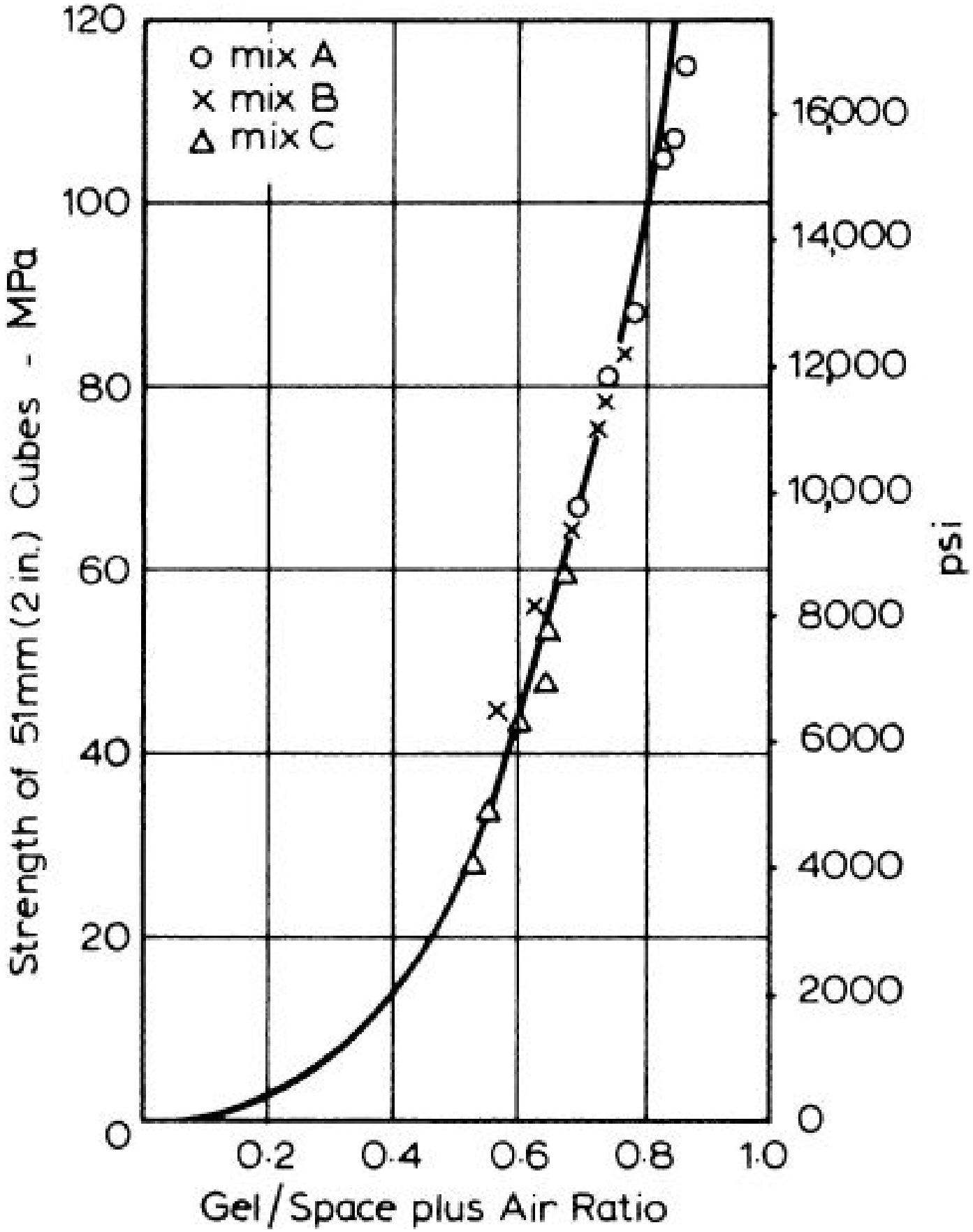
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Fig. 6.6. Relation between the compressive strength of mortar and gel/space ratio[6.8]

These calculations require a small modification to take account of the fact that the specific gravity of the adsorbed water is 1.1 (see p. 37). Therefore, the actual volume of voids is somewhat larger than assumed.

If the volume of air present in the cement paste is A, the ratio w_o/c in the above expression is replaced by $(w_o + A)/c$ (see Fig. 6.7). The resulting expression for strength is similar to that of Féret but the ratio used here involves a quantity proportional to the volume of hydrated cement instead of the total volume of content, and is thus applicable at any age.

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Fig. 6.7. Relation between the compressive strength of mortar and gel/space ratio, modified to include entrapped air voids[6.7]

The expression relating strength to the gel/space ratio can be written in a number of ways. It may be convenient to utilize the fact that the volume of non-evaporable water, w_n, is proportional to the volume of the gel; and also that the volume of mixing water, w_o, is related to the space available for the gel. The strength, f_c, in pounds per square inch, for f_c greater than about 2000 psi, when the relation is approximately linear, can then be written (using the original American units) in the form:[6.6]

$$f_c = 34\,200\frac{w_n}{w_o} - 3600.$$

Alternatively, the surface area of gel, V_m, can be used. Then (still in the American units):

$$f_c = 120\,000\frac{V_m}{w_o} - 3600.$$

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Figure 6.8 shows Powers' actual data[6.6] for cements with low C_3A contents.

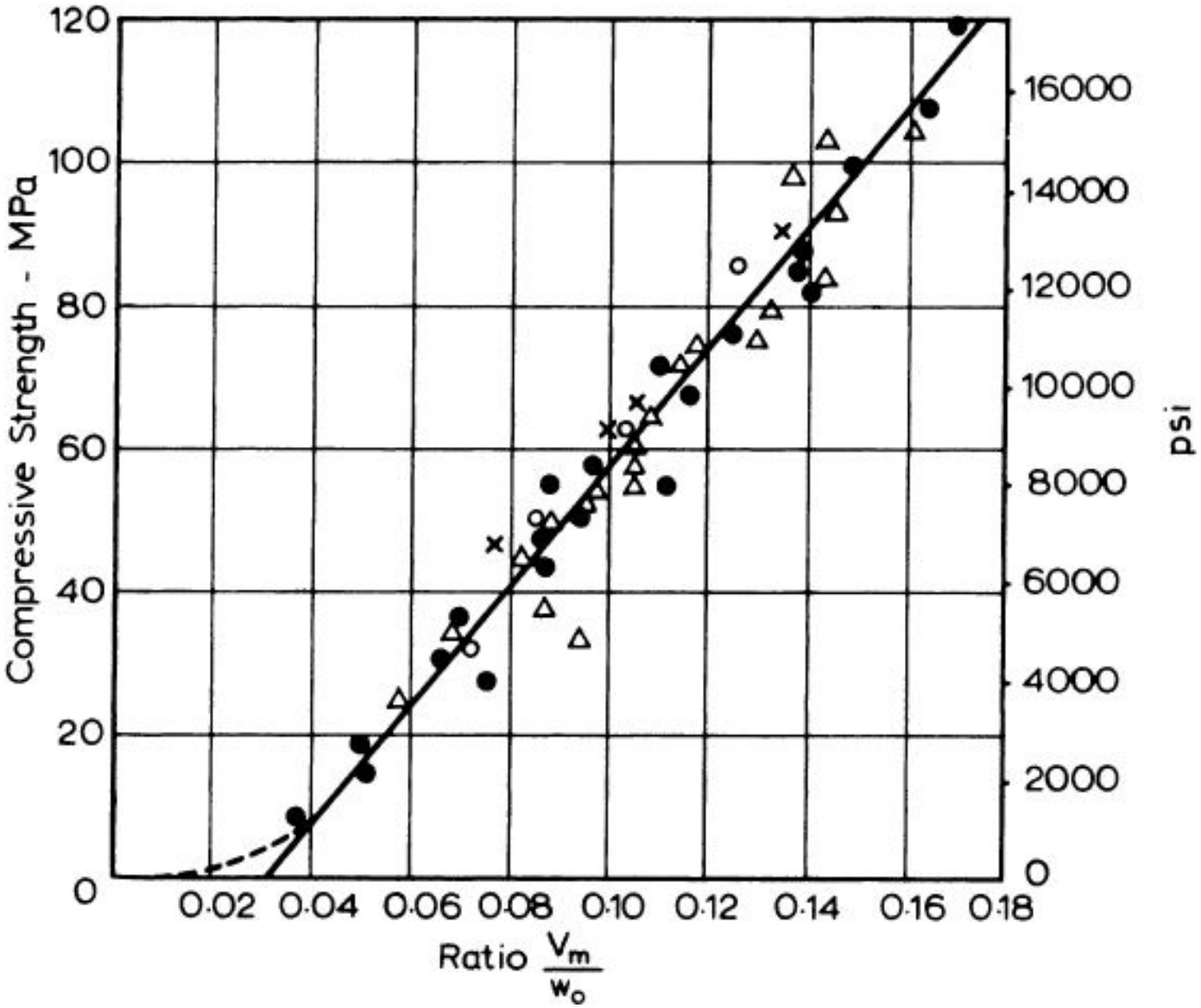


Fig. 6.8. Relation[6.6] between the strength of cement paste and the ratio of surface area of gel V_m to the volume of mixing water w_o

The above expressions have been found to be valid for many cements but the numerical coeffi-

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cients may depend on the intrinsic strength of the gel produced by a given cement. In other words, the strength of the cement paste depends primarily on the physical structure of the gel but the effects of the chemical composition of cement cannot be neglected; however, at later ages, these effects become minor only. Another way of recognizing the properties of the gel is to say that strength depends primarily on porosity but it is also affected by the ability of the material to resist crack propagation, which is a function of bonding. Poor bond between two crystals can be considered to be a crack.[6.35]

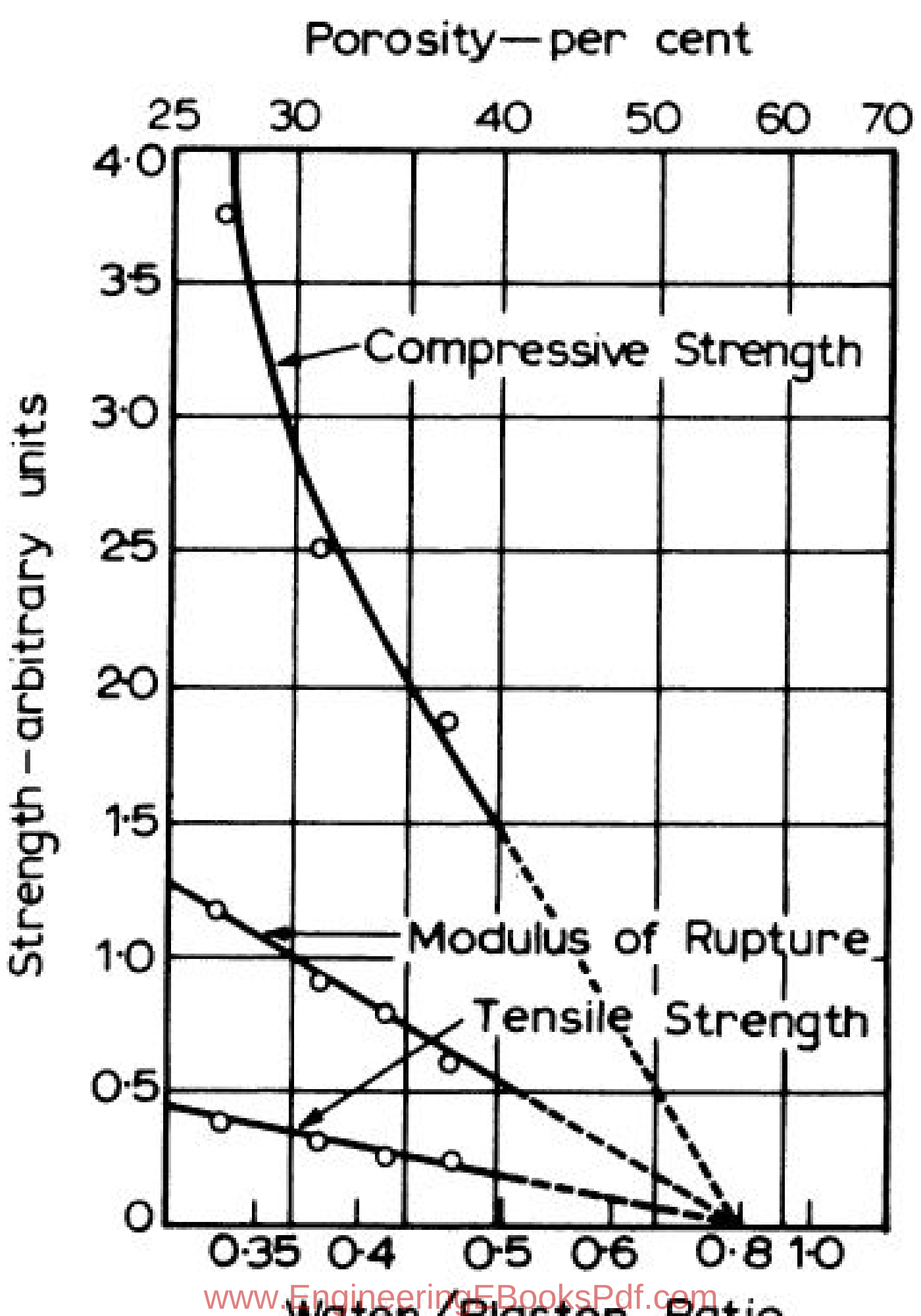
Porosity

The discussion in the preceding two sections showed that the strength of concrete is fundamentally a function of the volume of voids in it. The relation between strength and the total volume of voids is not a unique property of concrete but is found also in other brittle materials in which water leaves behind pores: for instance,

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the strength of plaster is also a direct function of its void content[6.1] (see Fig. 6.9). Moreover, if the strengths of different materials are expressed as a fraction of their respective strengths at a zero porosity, a wide range of materials conform to the same relation between relative strength and porosity, as shown in Fig. 6.10 for plaster, steel, iron,[6.72] alumina and zirconia.[6.73] This general pattern is of interest in understanding the role of voids in the strength of concrete. Moreover, the relation of Fig. 6.10 makes it clear why cement compacts (see p. 281), which have a very low porosity, have a very high strength.

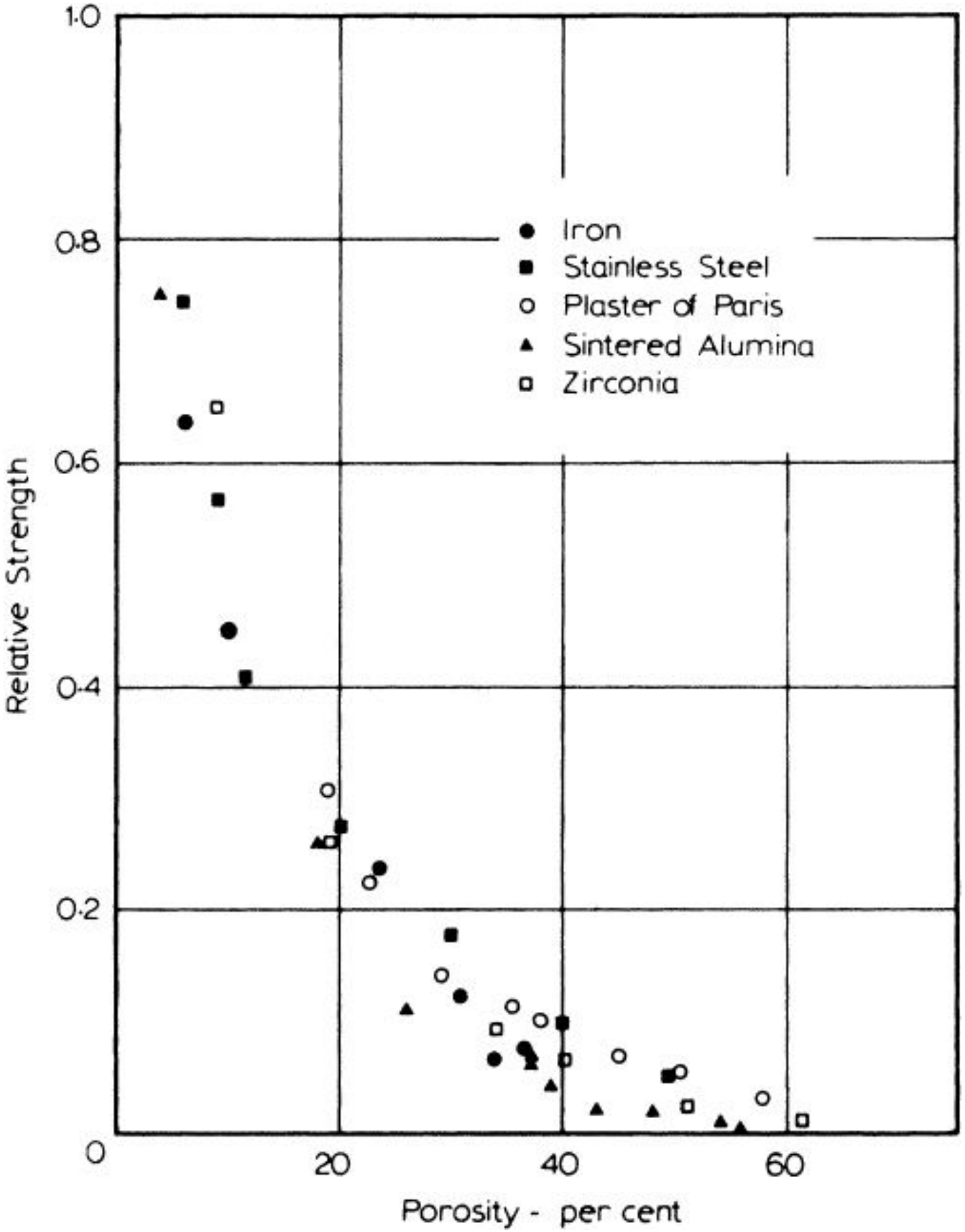
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Fig. 6.9. Strength of plaster as a function of its void content[6.1]

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Fig. 6.10. Influence of porosity on relative strength of various materials

Strictly speaking, strength of concrete is influenced by the volume of all voids in concrete: entrapped air, capillary pores, gel pores, and entrained air, if present.[6.10] An example of the calculation of the total void content may be of interest and is given below.

Let the given mix have proportions of cement, fine aggregate and coarse aggregate of 1 : 3.4 : 4.2, with a water/cement ratio of 0.80. The entrapped air content has been measured to be 2.3 per cent. Given that the specific gravity of the fine and coarse aggregates is, respectively, 2.60 and 2.65, and on the assumption that the specific gravity of cement is 3.15, the volumetric ratio of cement : fine aggregate : coarse aggregate: water is

$$(1/3.15) : (3.4/2.60) : (4.2/2.65) : (0.80) = 0.318 : 1.31 : 1.58 : 0.80.$$

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Because the air content is 2.3 per cent, the volume of the remaining materials must add up to 97.7 per cent of the total volume of concrete. Thus, on a percentage basis, the volumes are as follows:

Cement (dry)	= 7.8
Fine aggregate	= 32.0
Coarse aggregate	= 38.5
Water	= 19.4
Total	= 97.7 per cent.

We know that, in the given case, 0.7 of the cement has hydrated after 7 days of curing in water (see, for instance, ref. 6.32). Therefore, continuing in percentage volume units, we find the volume of the cement which has hydrated to be 5.5 and the volume of unhydrated cement 2.3.

The volume of combined water is 0.23 of the mass of cement which has hydrated (see p. 26), i.e. $0.23 \times 5.5 \times 3.15 = 4.0$. On hydration, the volume of the solid products of hydration becomes smaller than the sum of volumes of the

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constituent cement and water by 0.254 of the volume of combined water (see p. 26). Hence, the volume of the solid products of hydration is:

$$5.5 + (1 - 0.254) \times 4.0 = 8.5.$$

Since the gel has a characteristic porosity of 28 per cent (see p. 26), the volume of gel pores is w_g such that $w_g/(8.5 + w_g) = 0.28$, whence the volume of gel pores is 3.3. Thus, the volume of hydrated cement paste, inclusive of gel pores, is 8.5 + 3.3 = 11.8. Now, the volume of dry cement which has hydrated and of mixing water is 5.5 +19.4 = 24.9. Hence, the volume of capillary pores is 24.9 – 11.8 = 13.1. Thus, the voids are:

Capillary pores	= 13.1
Gel pores	= 3.3
Air	= 2.3
Total void content	= 18.7 per cent.

The influence of the volume of pores on strength can be expressed by a power function of the type:

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$$f_c = f_{c,0}(1 - p)^n$$

where p = porosity, that is, the volume of voids expressed as a fraction of the total volume of concrete

f_c = strength of concrete with porosity p

$f_{c,0}$ = strength at zero porosity, and

n = a coefficient, which need not be constant.[6.33]

The exact form of the relation is, however, uncertain. Tests on pressed and heat-treated cement compacts, as well as on ordinary cement paste, leave us still in doubt as to whether the logarithm of porosity is linearly related to strength or to its logarithm. Figures 6.11 and 6.12 illustrate the uncertainty. As far as the strength of individual cement compounds is concerned, it has been found to be linearly related to porosity (see Fig. 6.13).[6.65]

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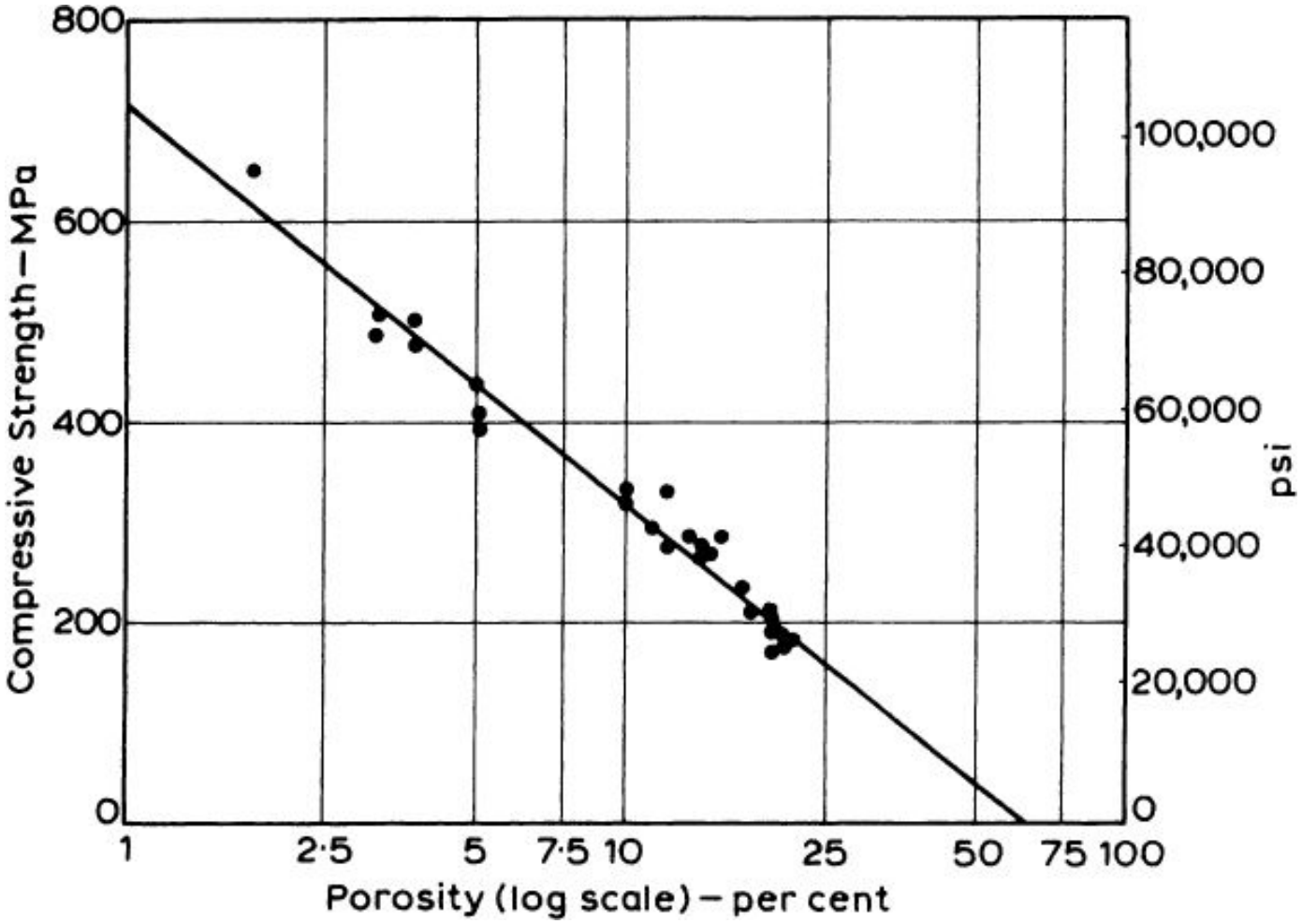


Fig. 6.11. Relation between compressive strength and logarithm of porosity of cement paste compacts for various treatments of pressure and high temperature[6.34]

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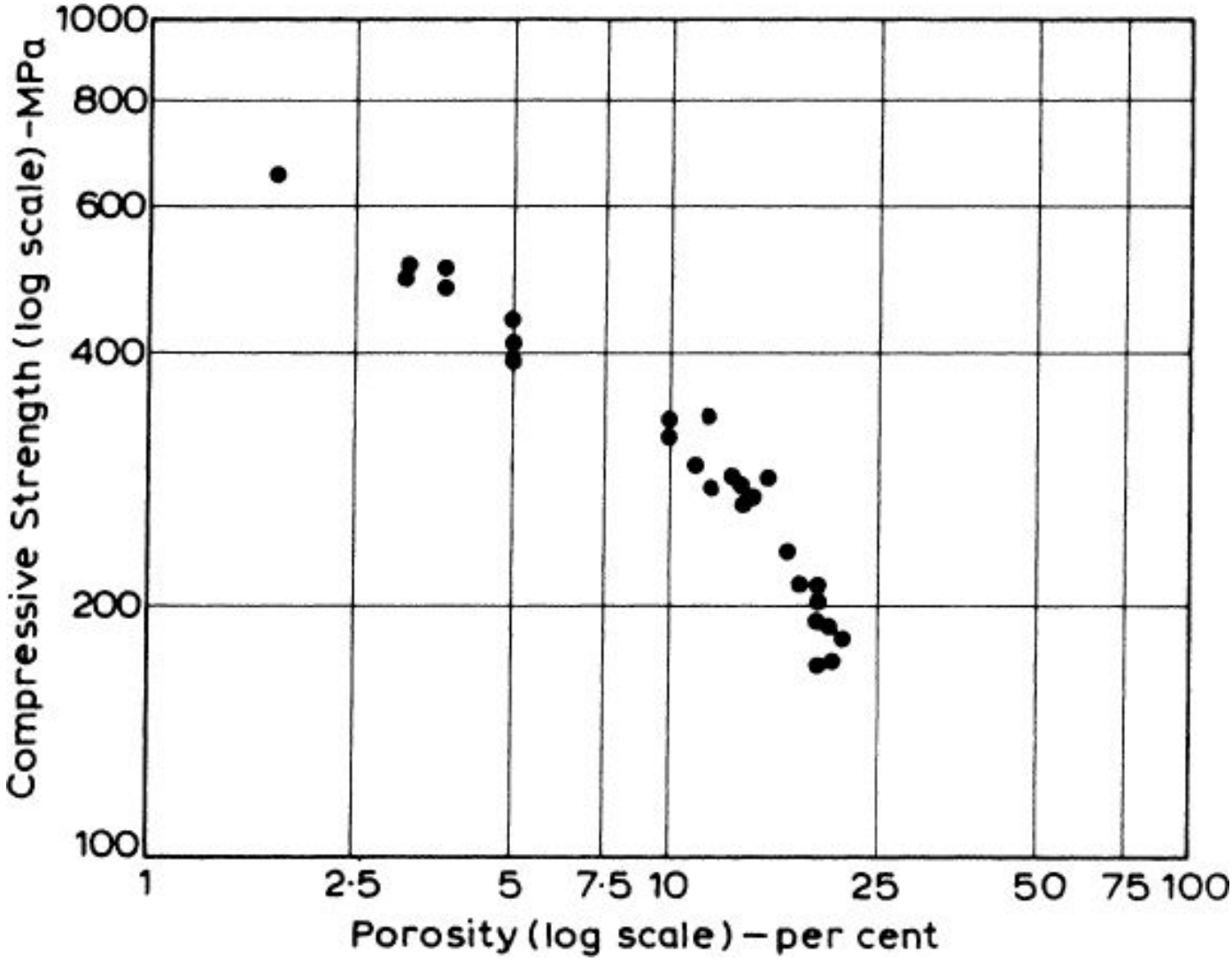


Fig. 6.12. Relation between logarithm of compressive strength and logarithm of porosity of cement paste compacts for various treatments of pressure and high temperature (after ref. 6.34)

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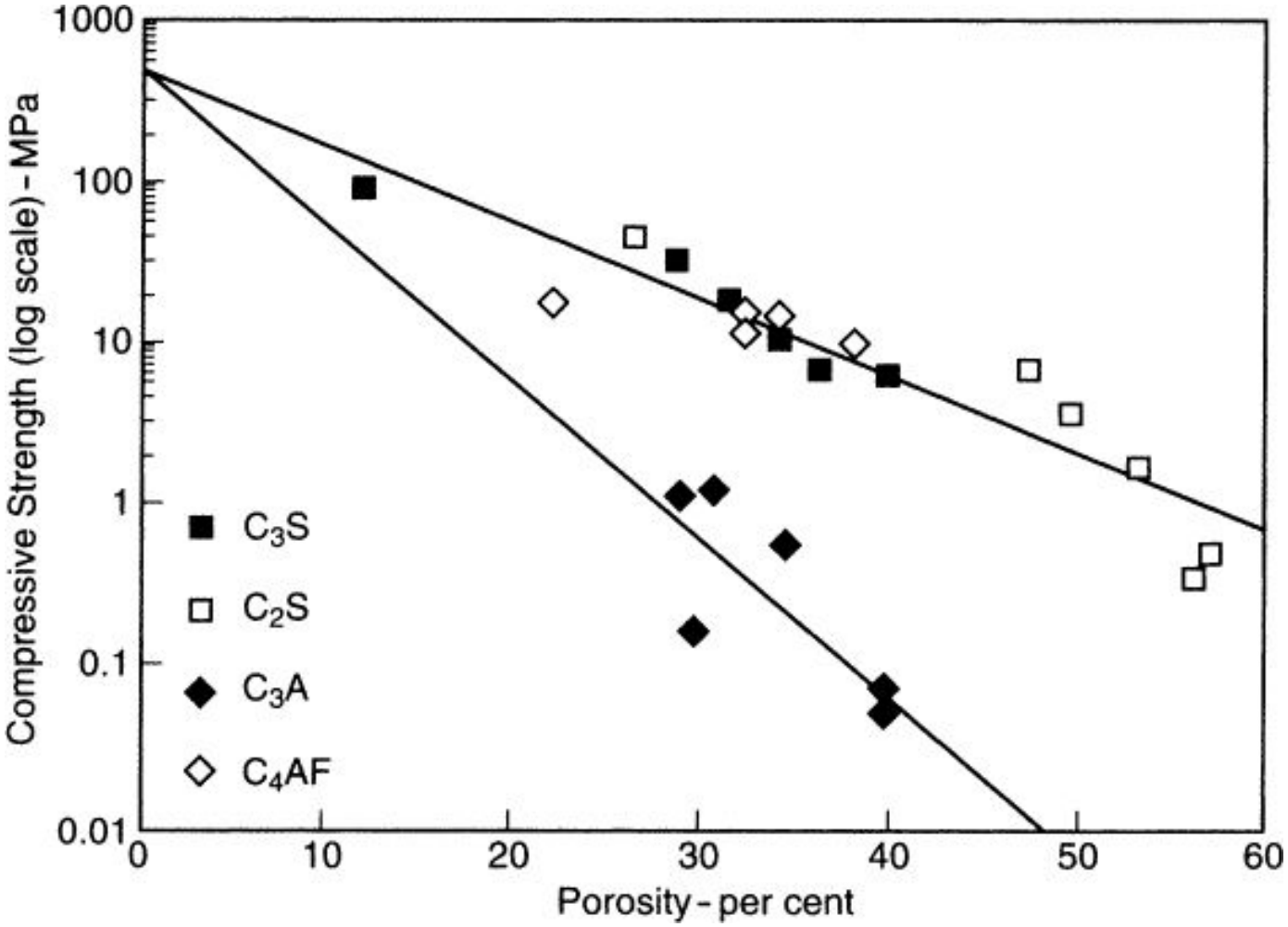


Fig. 6.13. Relation between compressive strength and porosity of pure compounds[6.65]

In addition to their volume, the shape and size of pores are also factors. The shape of the solid particles and their modulus of elasticity also influence the stress distribution and, therefore, stress concentration, within concrete. An example of pore distribution in concrete is shown

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in Fig. 6.14.[6.68] Similar results were obtained by Hearn and Hooton.[6.113]

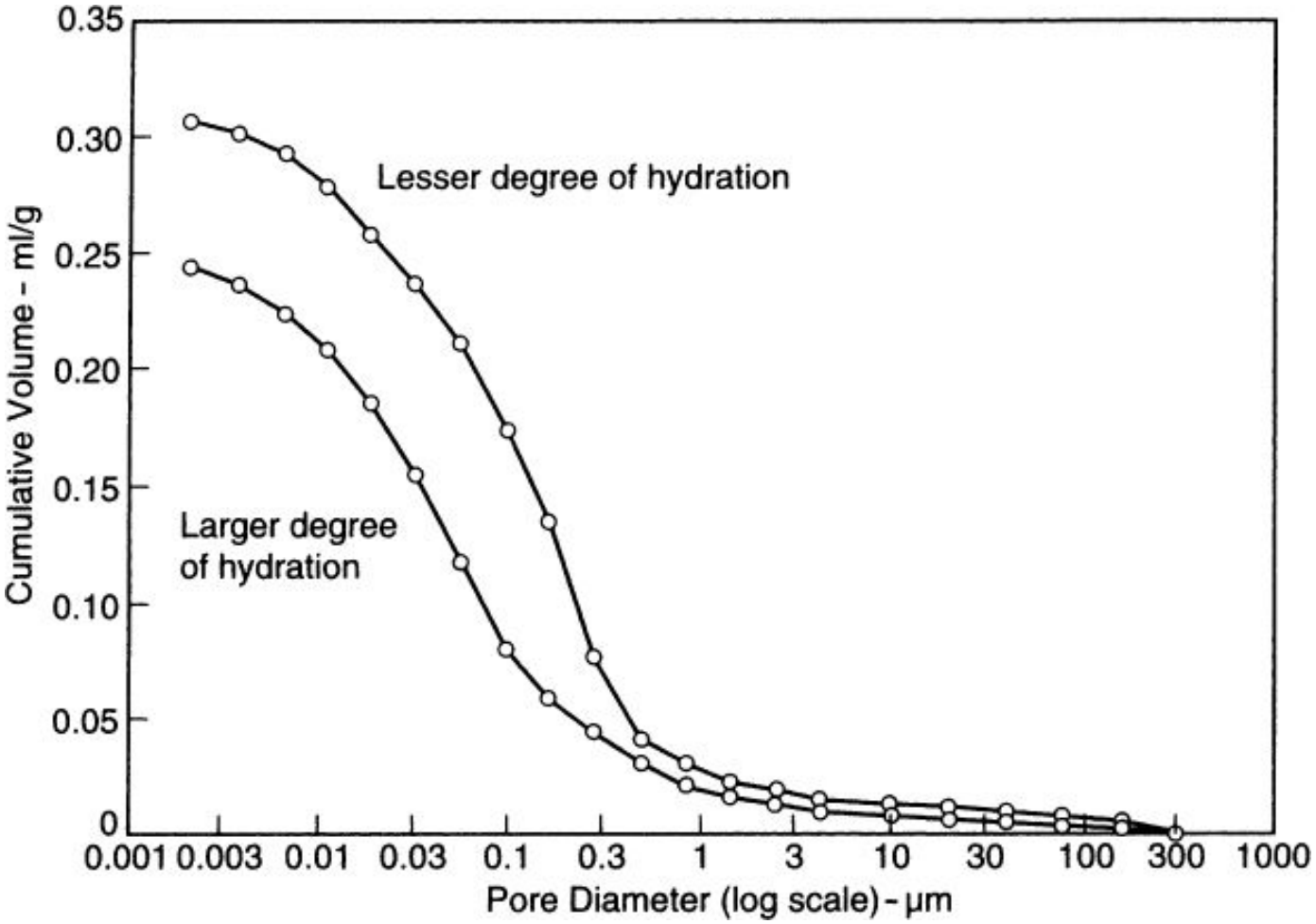


Fig. 6.14. Cumulative volume of pores larger than indicated pore diameter in concrete with a water/cement ratio of 0.45 at 20 °C (based on ref. 6.68)

The effect of porosity on the strength of hydrated cement paste has been studied widely. Care is required in translating observations on

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laboratory-made specimens of neat cement paste into usable information about concrete, but an understanding of the effect of porosity on strength of hydrated cement paste is valuable.

There is no doubt that porosity defined as the total volume of the overall volume of pores larger than gel pores, expressed as a percentage of the overall volume of the hydrated cement paste, is a primary factor influencing the strength of the cement paste. A linear relation between strength and porosity, within the range of the latter between 5 and 28 per cent, was established by Rössler and Odler.[6.63] The effect of pores smaller than 20 nm in diameter was found to be negligible.[6.64] The relation between the strength of mortar and porosity based on volume of pores larger than 20 nm in diameter is shown in Fig. 6.15.[6.66] Consequently, in addition to total porosity, the effect of pore size distribution on strength must be considered. Generally, at a given porosity, smaller pores lead to a higher strength of the cement paste.

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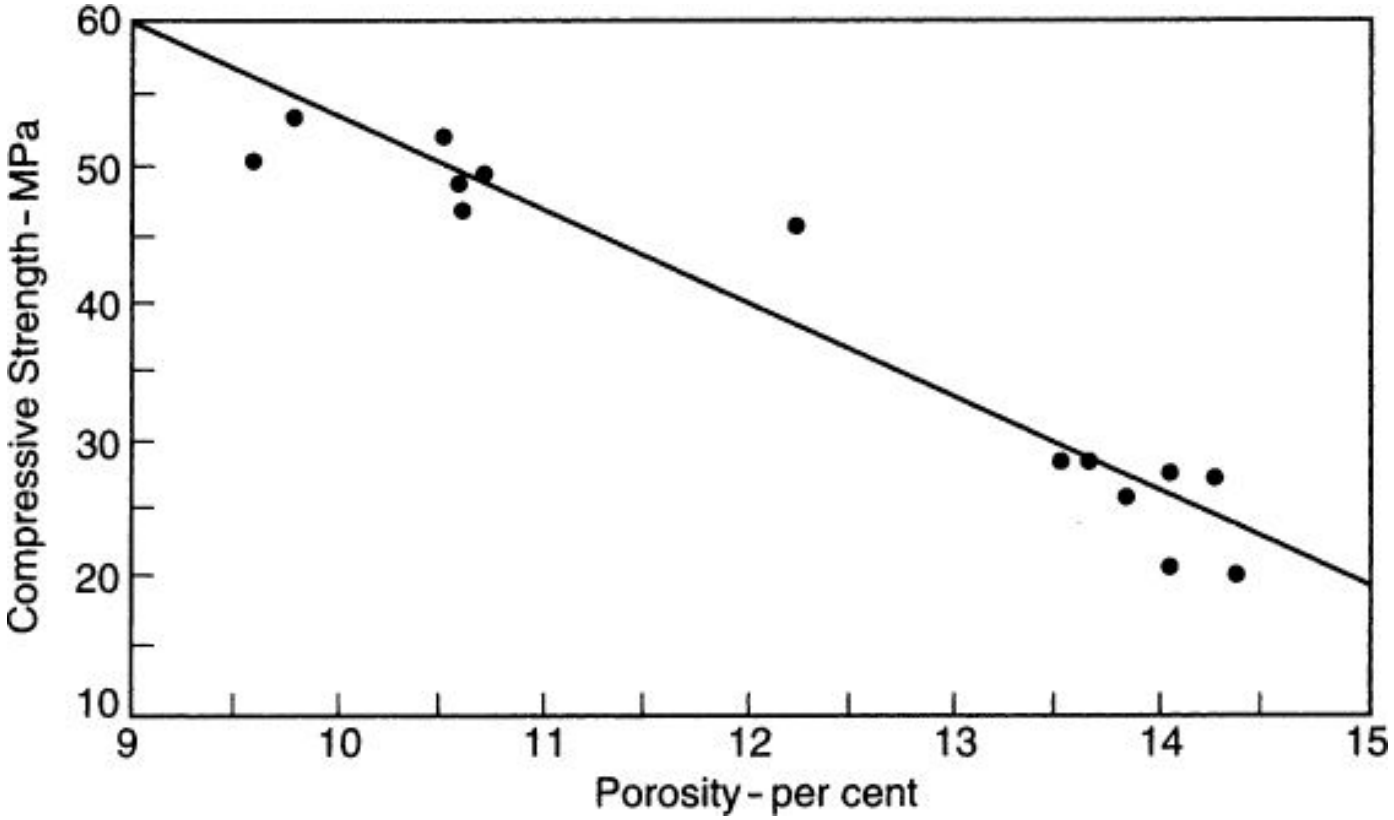


Fig. 6.15. Relation between compressive strength of mortar and porosity calculated from the volume of pores larger than 20 nm in diameter (based on ref. 6.66)

Although the pore size is, for convenience, expressed as a diameter, all the pores are by no means cylindrical or spherical in shape: the 'diameter' represents a sphere with the same ratio of volume to surface area as the totality of pores. It is only macropores, that is those whose diameter is larger than about 100 nm, that are approximately spherical. Figure 6.16 shows a diagrammat-

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ic representation of the various pores. This figure is an extension and modification of Fig. 1.13. The spherical pores originate from residual air bubbles or from imperfect cement particle packing but are not readily detected in porosimetry measurements because they are accessible only through connecting pores which have a narrow entrance (see Fig. 6.16).[6.70]

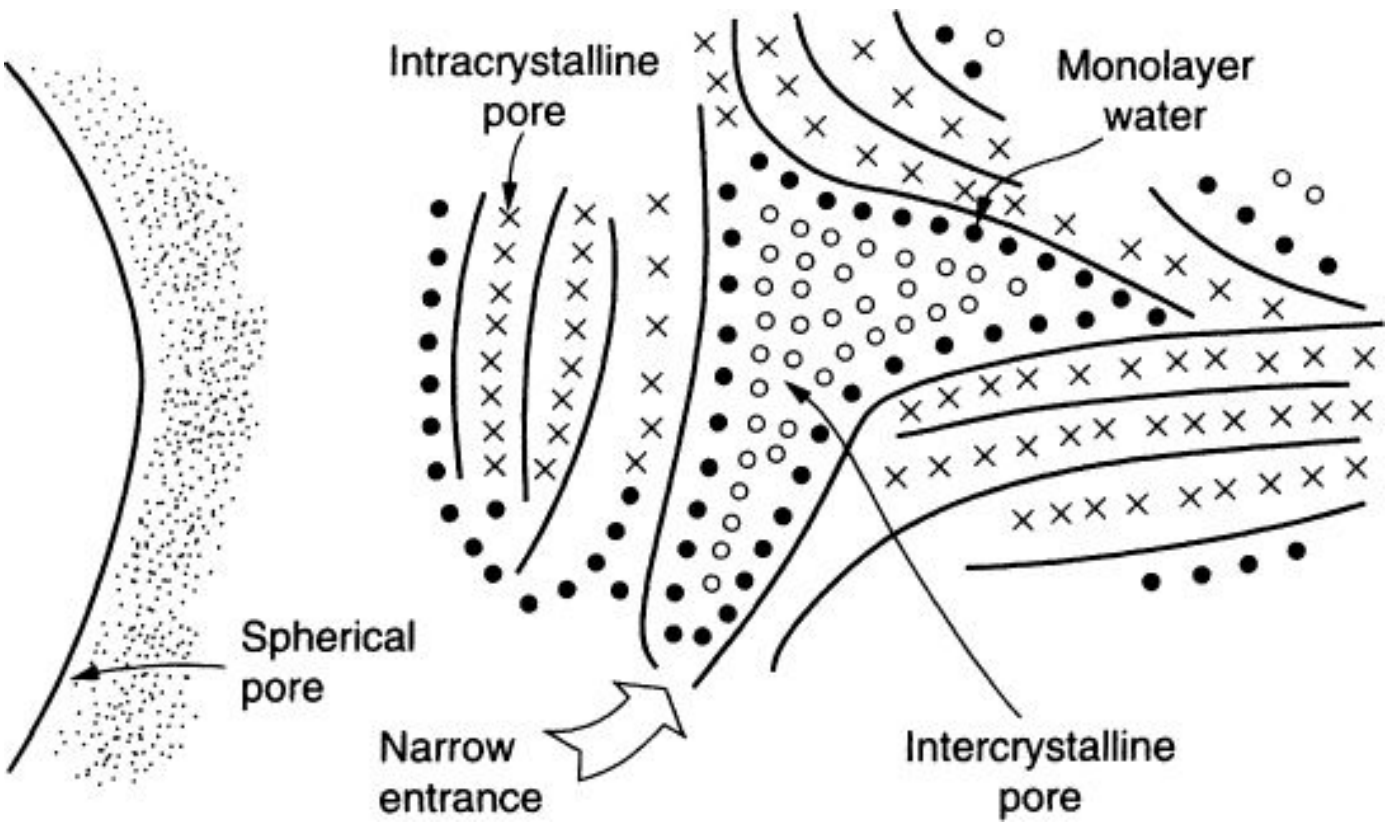


Fig. 6.16. Diagrammatic representation of the pore system in hydrated cement paste (based on Rahman's model in ref. 6.70)

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The dependence of the strength of hydrated cement paste on its porosity and on the pore size distribution is fundamental. Research papers occasionally consider a relation between strength and the gypsum content of the cement, but this is the outcome of the fact that the gypsum content affects the progress of hydration of cement and thus the pore distribution within the hydrated cement paste. The problem is, however, complicated by the fact that different methods of determination of porosity do not always yield the same values.[6.69] The main reason for this is that the process of porosimetry measurement, especially if it involves removal or addition of water, affects the structure of the hydrated cement paste.[6.67] The use of mercury intrusion in studies of the pore system in the cement paste is discussed by Cook and Hover.[6.114] This method assumes that pores become narrower with depth while, in fact, some pores have a constricted entrance; this distorts the value of porosity measured by mercury intrusion porosimetry.[6.115]

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As pointed out earlier, most of the experimental work on porosity of hydrated cement paste has been performed on specimens of neat cement paste or of mortar. In concrete, the pore characteristics of the hydrated cement are somewhat different because of the influence of coarse aggregate particles on the cement paste in their neighbourhood. Winslow and Liu[6.68] found that, with the same paste composition and at the same degree of hydration, the presence of coarse aggregate results in an increased porosity; even the presence of fine aggregate has a similar, but smaller, effect. The difference in porosity between concrete and neat cement paste, at the same water/cement ratio, increases with the progress of hydration and arises from the presence in concrete of some pores larger than those which can exist in neat cement paste.

Cement compacts

Cement compacts are manufactured by application of very high pressure with simultaneous high

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temperature. They do not, therefore, come under the heading of concrete, but are of interest in elucidating the role of porosity in strength because a porosity as low as 1 per cent can be achieved.[6.34]

One of the strongest cement-based materials reported[6.62] to have been produced had a water/cement ratio of 0.08: when compacted, it had a strength of 345 MPa (50 000 psi). Application of pressure of 340 MPa (49 500 psi) and temperature of 250 °C (480 °F) has resulted in compacts with a compressive strength of about 660 MPa (95 000 psi) and a tensile splitting strength of 64 MPa (9300 psi).[6.34]

Extrapolation of an experimental relation between porosity and compressive strength of specimens of the individual compounds in Portland cement, at a water/solid ratio of 0.45, suggests that, at zero porosity, the strength is about 500 MPa.[6.65] This can be compared with the value calculated by Nielsen[6.59] who estimates the

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strength of hydrated cement paste at zero porosity to be 450 MPa.

These values, although not unique, represent the intrinsic strength of hardened Portland cement paste.

Influence of properties of coarse aggregate on strength

Although the relation between strength and the water/cement ratio is generally valid, it is not independent of other factors. One of these is discussed in this section.

Vertical cracking in a specimen subjected to uniaxial compression starts under a load equal to 50 to 75 per cent of the ultimate load. This has been determined from measurements of the velocity of sound transmitted through the concrete,[6.22] and also using ultrasonic pulse velocity techniques.[6.23] The stress at which the cracks develop depends largely on the properties of the coarse aggregate: smooth gravel leads to cracking

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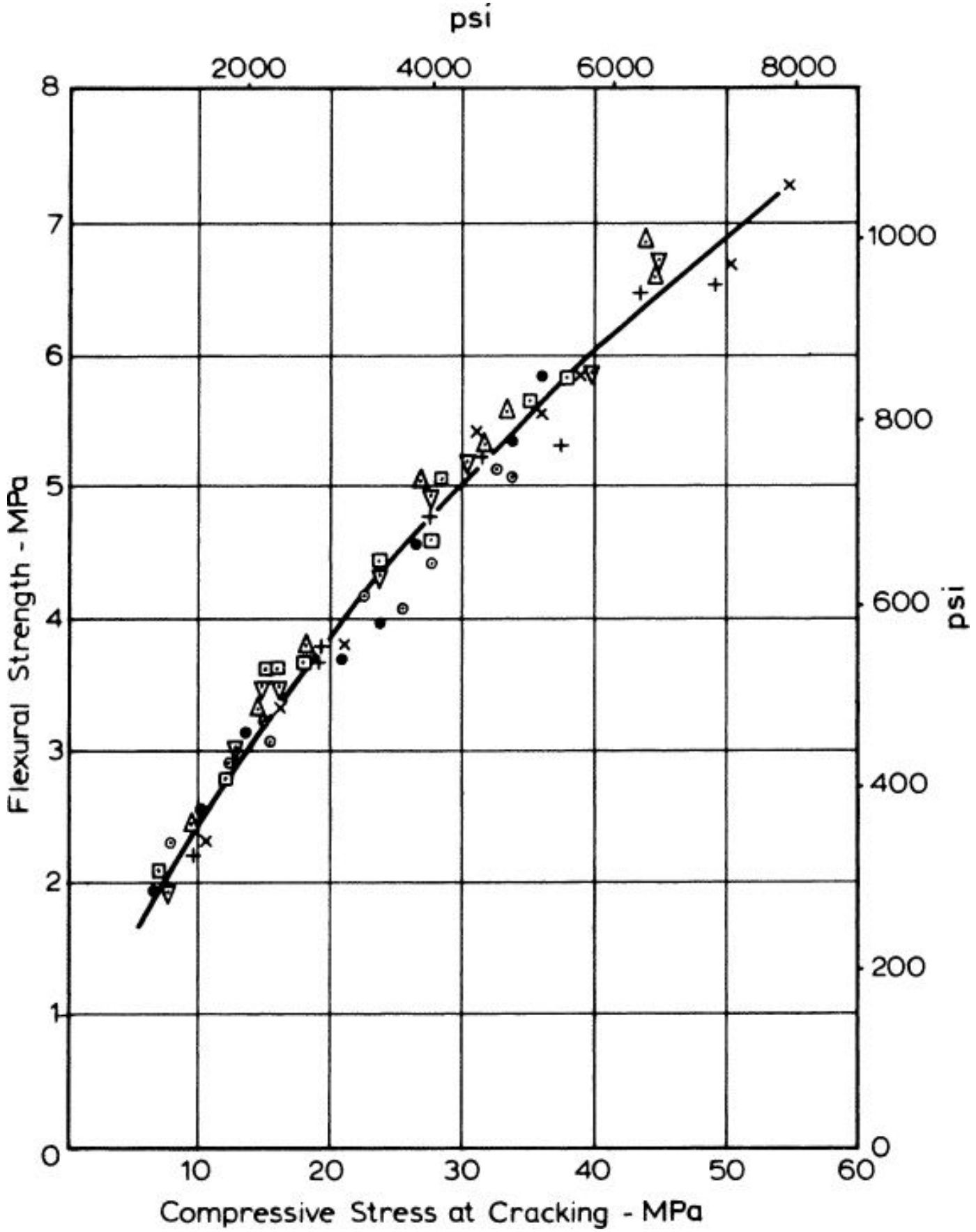
at lower stresses than rough and angular crushed rock, probably because mechanical bond is influenced by the surface properties and, to a certain degree, by the shape of the coarse aggregate.[6.19]

The properties of aggregate affect thus the cracking load, as distinct from the ultimate load, in compression and the flexural strength in the same manner, so that the relation between the two quantities is independent of the type of aggregate used. Figure 6.17 shows Jones and Kaplan's[6.19] results, each symbol representing a different type of coarse aggregate. On the other hand, the relation between the flexural and compressive *strengths* depends on the type of coarse aggregate used (see Fig. 6.18) because (except in high strength concrete) the properties of aggregate, especially its shape and surface texture, affect the ultimate strength in compression very much less than the strength in tension or the cracking load in compression. This behaviour was confirmed by Knab.[6.71] In experimental concrete, entirely smooth coarse aggregate led to a lower compress-

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ive strength, typically by 10 per cent, than when roughened.[6.38]

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Fig. 6.17. Relation between flexural strength and compressive stress at cracking for concretes made with different coarse aggregates[6.19] (Crown copyright)

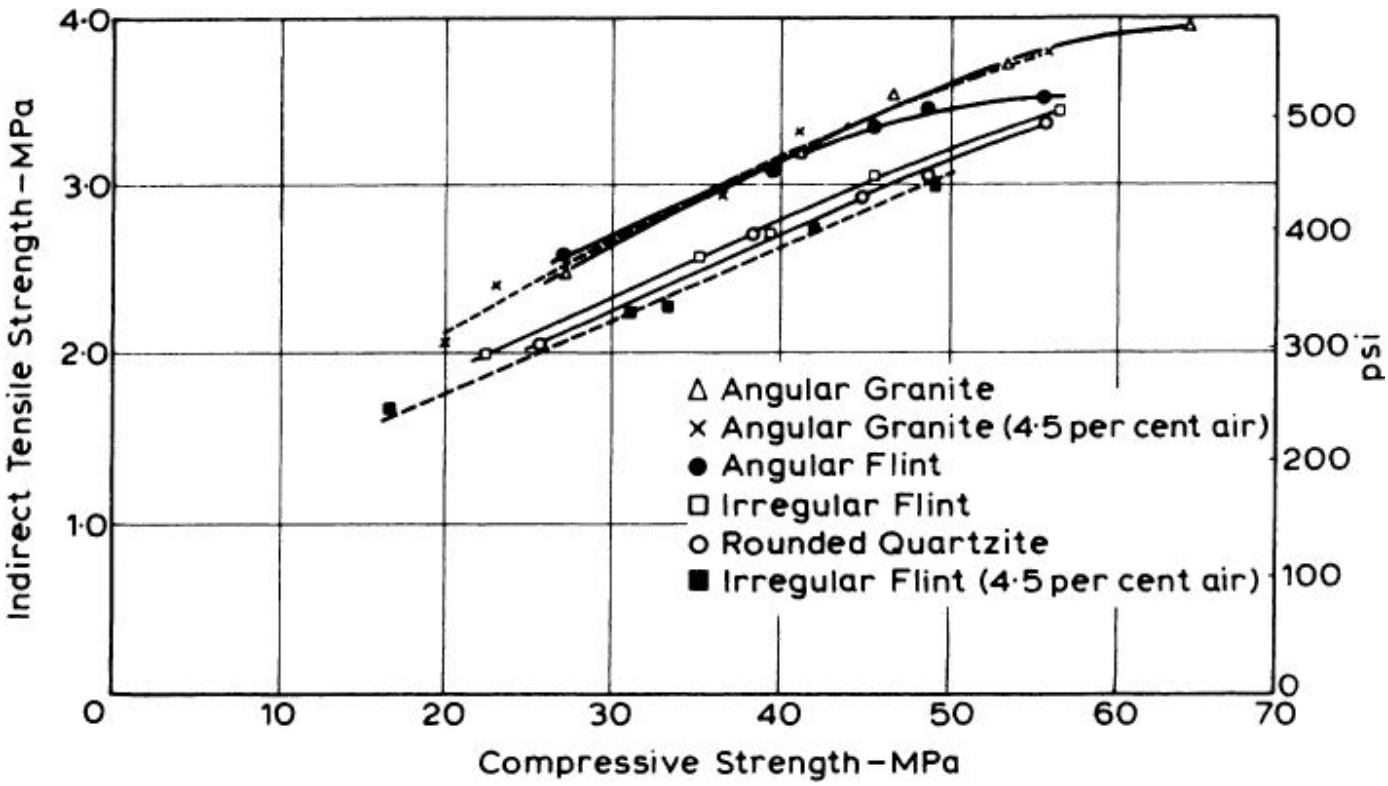


Fig. 6.18. Relation between compressive strength and indirect tensile strength for concretes of constant workability made with various aggregates (water/cement ratio between 0.33 and 0.68, aggregate/cement ratio between 2.8 and 10.1)[6.39] (Crown copyright)

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The influence of the type of coarse aggregate on the strength of concrete varies in magnitude and depends on the water/cement ratio of the mix. For water/cement ratios below 0.4, the use of crushed aggregate has resulted in strengths up to 38 per cent higher than when gravel is used. The behaviour at a water/cement ratio of 0.5 is shown in Fig. 6.19.[6.39] With an increase in the water/cement ratio, the influence of aggregate falls off, presumably because the strength of the hydrated cement paste itself becomes paramount and, at a water/cement ratio of 0.65, no difference in the strengths of concretes made with crushed rock and gravel has been observed.[6.24]

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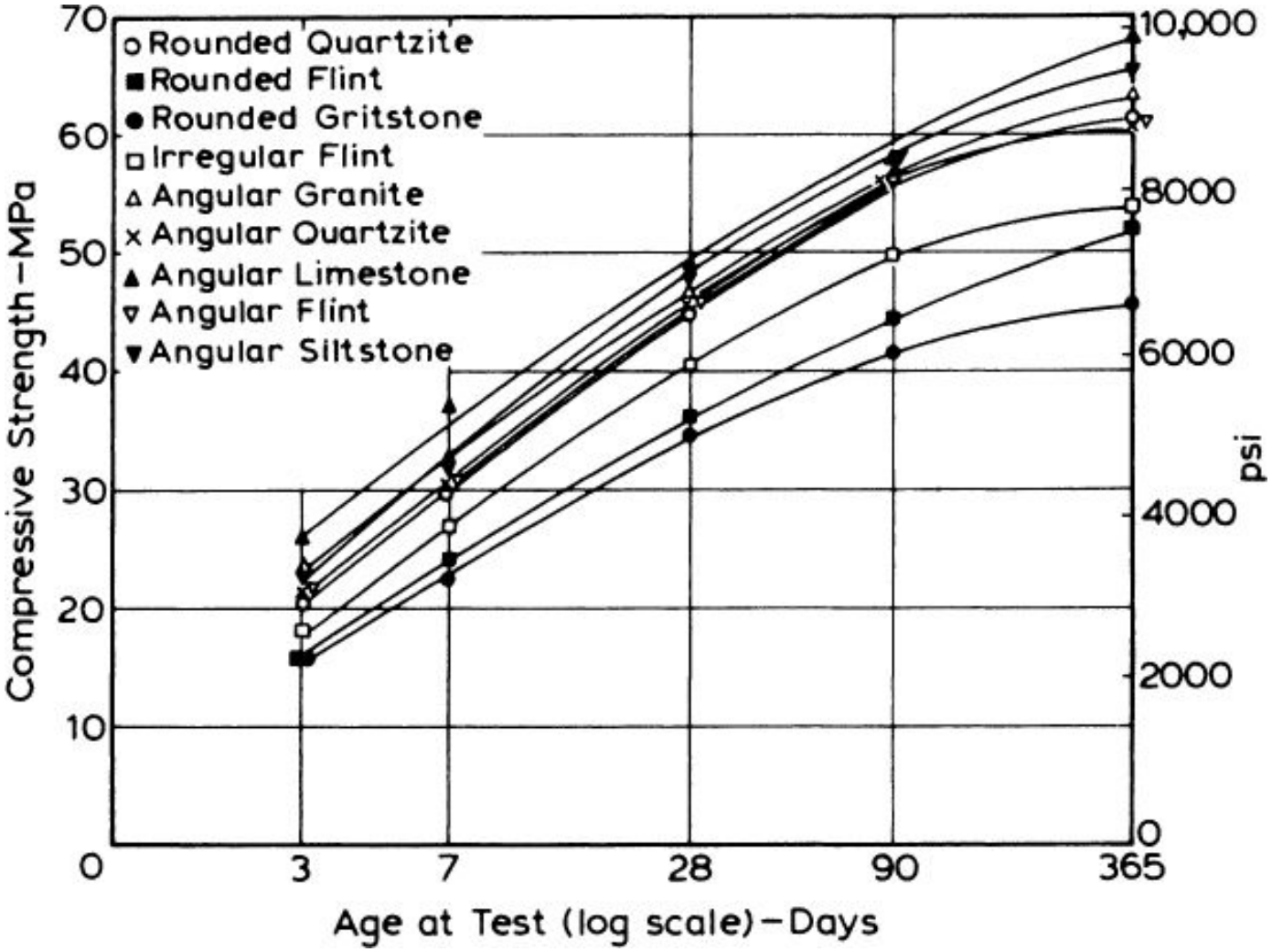


Fig. 6.19. Relation between compressive strength and age for concretes made with various aggregates (water/cement ratio = 0.5)[6.39] (Crown copyright)

The influence of aggregate on flexural strength seems to depend also on the moisture condition of the concrete at the time of test.[6.60]

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The shape and surface texture of coarse aggregate affect also the impact strength of concrete, the influence being qualitatively the same as on the flexural strength[6.61] (see p. 117).

Kaplan[6.25] observed that the flexural strength of concrete is generally lower than the flexural strength of corresponding mortar. Mortar would thus seem to set the upper limit to the flexural strength of concrete and the presence of the coarse aggregate generally reduces this strength. On the other hand, the compressive strength of concrete is higher than that of mortar, which, according to Kaplan, indicates that the mechanical interlocking of the coarse aggregate contributes to the strength of concrete in compression. This behaviour has not, however, been confirmed to apply generally, and the question of the influence of aggregate on strength is considered further in the next section. At this stage, it is useful to note that coarse aggregate particles act as crack arresters so that, under an increasing load, another crack is likely to open. Failure is, there-

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fore, gradual and, even in tension, there exists a descending part of the stress–strain curve.

Influence of aggregate/cement ratio on strength

The anomalous behaviour of extremely rich mixes with respect to strength was mentioned on p. 270, but the richness of the mix affects the strength of all medium- and high-strength concretes, i.e. those with a strength of about 35 MPa (5000 psi) or more. There is no doubt that the aggregate/cement ratio, is only a secondary factor in the strength of concrete but it has been found that, for a constant water/cement ratio, a leaner mix leads to a higher strength[6.12] (see Fig. 6.20).

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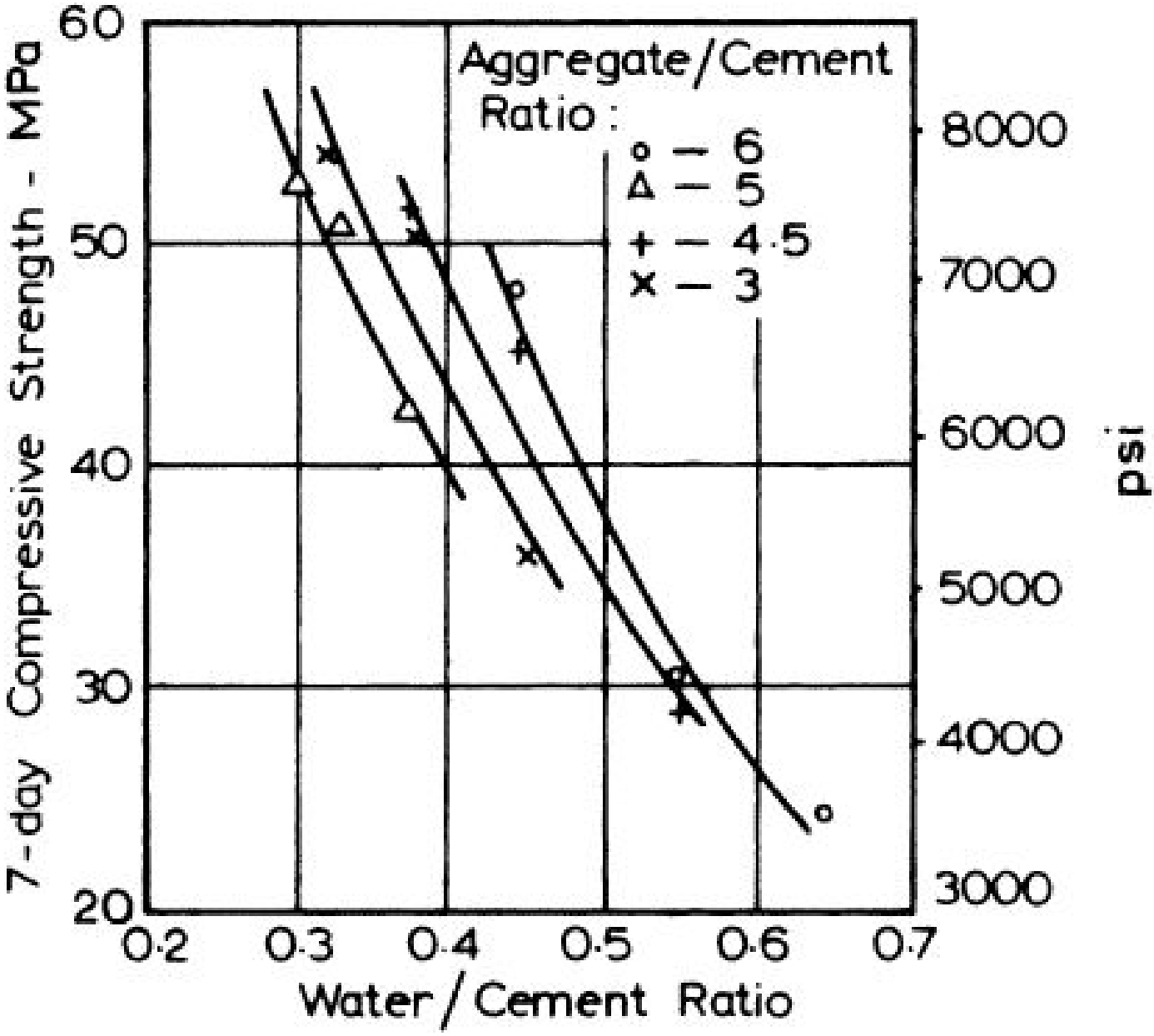


Fig. 6.20. Influence of the aggregate/cement ratio on strength of concrete[6.13]

The reasons for this behaviour are not clear. In certain cases, some water may be absorbed by the aggregate: a larger amount of aggregate absorbs a greater quantity of water, the effective water/

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cement ratio being thus reduced. In other cases, a higher aggregate content would lead to lower shrinkage and lower bleeding, and therefore to less damage to the bond between the aggregate and the cement paste; likewise, the thermal changes caused by the heat of hydration of cement would be smaller.[6.80] The most likely explanation, however, lies in the fact that the total water content per cubic metre of concrete is lower in a leaner mix than in a rich one. As a result, in a leaner mix, the voids form a smaller fraction of the total volume of concrete, and it is these voids that have an adverse effect on strength.

Studies on the influence of aggregate content on the strength of concrete with a given quality of cement paste indicate that, when the volume of aggregate (as a percentage of the total volume) is increased from zero to 20, there is a gradual decrease in compressive strength, but between 40 and 80 per cent there is an increase.[6.40] The pattern of behaviour is shown in Fig. 6.21. The reasons for this effect are not clear, but it is the same

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at various water/cement ratios.[6.41] The influence of the volume of aggregate on tensile strength is broadly similar[6.40] (Fig. 6.22).

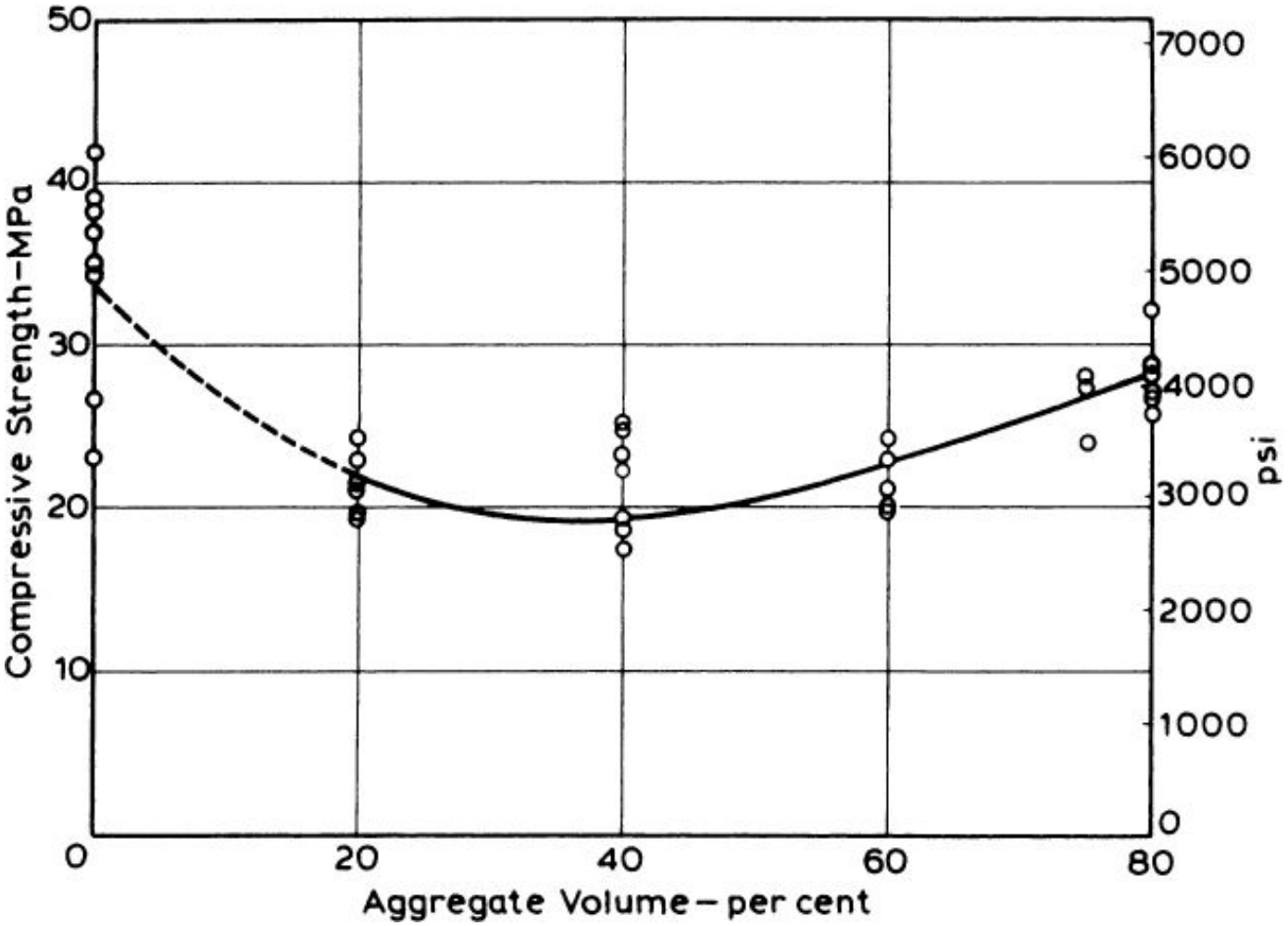


Fig. 6.21. Relation[6.40] between the compressive strength of cylinders (100 mm diameter, 300 mm in length) and volume of aggregate at a constant water/cement ratio of 0.50

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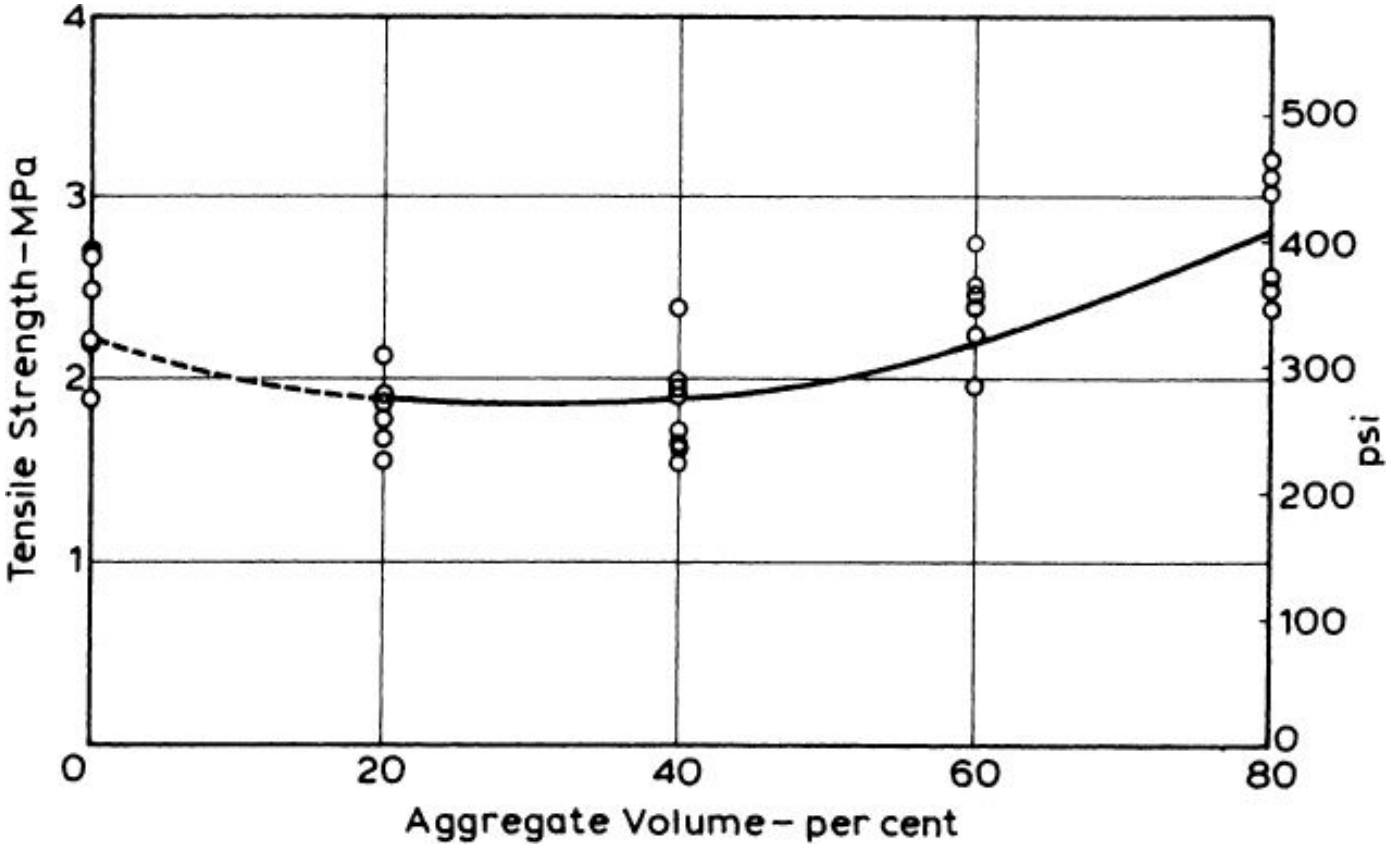


Fig. 6.22. Relation[6.40] between direct tensile strength and volume of aggregate at a constant water/cement ratio of 0.50

These effects are smaller in cubes than in cylinders or prisms. In consequence, the ratio of cylinder strength to cube strength (cf. p. 596) decreases as the volume of aggregate increases from zero to 40 per cent.[6.45] The explanation lies probably in the greater influence of the aggregate on the crack pattern when the end effect of platens is absent (see p. 587).

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Nature of strength of concrete

The paramount influence of voids in concrete on its strength has been repeatedly mentioned, and it should be possible to relate this factor to the actual mechanism of failure. For this purpose, concrete is considered to be a brittle material, even though it exhibits a small amount of plastic action, as fracture under static loading takes place at a moderately low total strain; a strain of 0.001 to 0.005 at failure has been suggested as the limit of brittle behaviour. High strength concrete is more brittle than normal strength concrete but there is no quantitative method of expressing the brittleness of concrete whose behaviour in practice falls between the brittle and the ductile types.

Strength in tension

The actual (technical) strength of hydrated cement paste or of similar brittle materials such as stone is very much lower than the theoretical strength estimated on the basis of molecular cohesion, and calculated from the surface energy of

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a solid assumed to be perfectly homogeneous and flawless. The theoretical strength has been estimated to be as high as 10.5 GPa (1.5×10^6 psi).

This discrepancy can be explained by the presence of flaws postulated by Griffith.[6.17] These flaws lead to high stress concentrations in the material under load so that a very high stress is reached in very small volumes of the specimen with a consequent microscopic fracture, while the average (nominal) stress in the whole specimen is comparatively low. The flaws vary in size, and it is only the few largest ones that cause failure: the strength of a specimen is thus a problem of statistical probability, and the size of the specimen affects the probable nominal stress at which failure is observed.

Hydrated cement paste is known to contain numerous discontinuities – pores, microcracks and voids – but the exact mechanism through which they affect the strength is not known. The voids themselves need not act as flaws, but the flaws may be cracks in individual crystals associ-

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ated with the voids[6.14] or caused by shrinkage or poor bond. This situation is not surprising in view of the heterogeneous nature of concrete and of the method of combining the various phases of this composite material into a single whole. Alford *et al.*[6.81] confirmed that the pores in the cement paste are not the only possible critical flaws. In unsegregated concrete, the voids are distributed in a random manner,[6.15] which is a condition necessary for the application of Griffith's hypothesis. While we do not know the exact mechanism of rupture of concrete, this is probably related to the bond within the hydrated cement paste and between the paste and the aggregate.

Griffith's hypothesis postulates microscopic failure at the location of a flaw, and it is usually assumed that the 'volume unit' containing the weakest flaw determines the strength of the entire specimen. This statement implies that any crack will spread throughout the section of the specimen subjected to the given stress or, in other words, an event taking place in an element is

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identified with the same event taking place in the body as a whole.

This behaviour can be met with only under a uniform stress distribution, with the additional proviso that the 'second weakest' flaw is not strong enough to resist a stress of $n/(n-1)$ times the stress at which the weakest flaw failed, where n is the number of elements in the section under load, each element containing one flaw.

Whereas local fracture starts at a point and is governed by the conditions at that point, the knowledge of stress at the most highly stressed point in the body is not sufficient to predict failure. It is necessary to know also the stress distribution in a volume sufficiently extended round this point because the deformational response within the material, particularly near failure, depends on the behaviour and state of the material surrounding the critical point, and the possibility of spreading of failure is strongly affected by this state. This would explain, for instance, why the maximum fibre stresses in flexure specimens at

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the instant of incipient failure are higher than the strength determined in uniform direct tension: in the latter case, the propagation of fracture is not blocked by the surrounding material. Some actual data on the relation between the strength in flexure and in splitting tension are given in Fig. 12.8.

We can see then that, in a given specimen, different stresses will produce fracture at different points, but it is not possible physically to test the strength of an individual element without altering its condition in relation to the rest of the body. If the strength of a specimen is governed by the weakest element in it, the problem becomes that of the proverbial weakest link in a chain. In statistical terms, we have to determine the least value (i.e. the strength of the most effective flaw) in a sample of size n, where n is the number of flaws in the specimen. The chain analogy may not be quite correct because, in concrete, the links may be arranged in parallel as well as in series, but computations on the basis of the weakest link assumption yield results of the correct order. It fol-

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lows that the strength of a brittle material such as concrete cannot be described by an average value only: an indication of the variability of strength must be given, as well as information about the size and shape of the specimens. These factors are discussed in Chapter 12.

Cracking and failure in compression

Griffith's hypothesis applies to failure under the action of a tensile force but it can be extended to fracture under bi- and triaxial stress and also under uniaxial compression. Even when two principal stresses are compressive, the stress along the edge of the flaw is tensile at some points, so that fracture can take place. Orowan[6.16] calculated the maximum tensile stress at the tip of the flaw of the most dangerous orientation relative to the principal stress axes as a function of the two principal stresses P and Q. The fracture criteria are represented graphically in Fig. 6.23, where K is the tensile strength in direct tension. Fracture occurs under a combination of P and Q such that

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the point representing the state of stress crosses the curve outwards onto the shaded side.

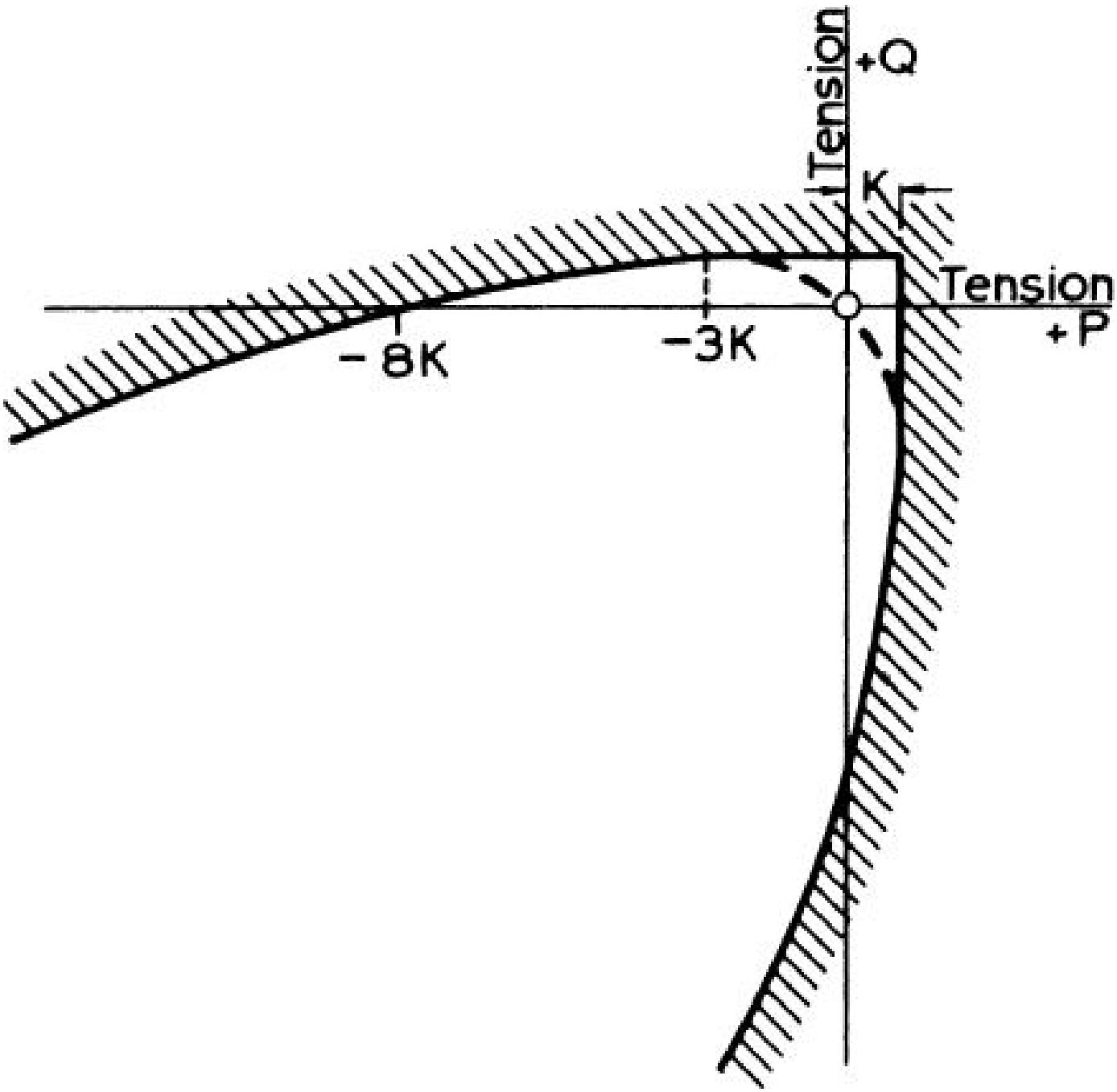


Fig. 6.23. Orowan's criteria of fracture under biaxial stress[6.16]

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From Fig. 6.23, it can be seen that fracture can occur when uniaxial compression is applied; this has in fact been observed in tests on concrete compression test specimens.[6.18] The nominal compressive strength in this case is $8K$, i.e. 8 times the tensile strength determined in a direct tension test. This figure is in good agreement with the observed values of the ratio of the compressive to tensile strengths of concrete. There are, however, difficulties in reconciling certain aspects of Griffith's hypothesis with the observed direction of cracks in compression specimens. It is possible, though, that failure in such a specimen is governed by the lateral strain induced by Poisson's ratio. The value of Poisson's ratio for concrete is such that, for elements sufficiently removed from the platens of the testing machine, the resulting lateral strain can exceed the ultimate tensile strain of concrete. Failure occurs then by splitting at right angles to the direction of the load, as in the splitting test (see p. 600), and this has been frequently observed, especially in

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specimens whose height is greater than their breadth.[6.18] The view that concrete fails by tensile splitting when subjected to uniaxial or bi-axial compression was confirmed by Yin *et al*.[6.86]

There are strong indications, first developed in ref. 6.14, that it is not a limiting strain but a limiting tensile strain that determines the strength of concrete under static loading: this is usually assumed to be between 100×10^{-6} and 200×10^{-6}. The failure criterion of limiting tensile strain is supported by an analysis advanced by Lowe.[6.36] It has been found that, at the point of initial cracking, the strain on the tension face of a beam in flexure and the lateral tensile strain in a cylinder in uniaxial compression are of similar magnitude.[6.21] The tensile strain in a beam at cracking is:

$$\frac{\text{tensile stress at cracking}}{E}$$

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where E is the modulus of elasticity of concrete over the linear range of deformation. Now, the lateral strain in a compression specimen when cracking is first observed is:

$$\frac{\mu \times \text{compressive stress at cracking}}{E}$$

where μ is the static Poisson's ratio, and E is the same as above. From the observed equality of the two strains it would appear that:

$$\mu = \frac{\text{tensile stress at cracking in flexure}}{\text{compressive stress at cracking in a compression specimen}}.$$

Poisson's ratio varies generally between about 0.15 for high strength concrete and 0.22 for low strength concrete (see p. 421), and it is significant that the ratio of the nominal tensile and compressive strengths for different concretes varies in a similar manner and between approximately the same limits. There is thus a possible connection between the ratio of nominal strengths and Poisson's ratio, and there are good grounds for suggesting that the mechanism producing the initial cracks in uniaxial compression and in flex-

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ure tension is the same.[6.19] The nature of this mechanism has not been established. It is probable that cracking is due to local breakdowns in bond between the cement and the aggregate.[6.20] However, the basic mechanism of compressive failure of concrete has not been reliably established and even the definition of failure of concrete is not obvious. One view is to associate failure with the so-called discontinuity point, defined as the point at which the volumetric strain stops decreasing and Poisson's ratio starts to increase sharply.[6.52,6.53] At this stage, extensive mortar cracking starts to develop (see p. 300). This is the beginning of instability, and sustained loading above this point will lead to failure. The lateral tensile strain at discontinuity depends on the level of axial compression and is greater for stronger concrete; Carino and Slate[6.53] observed an average value of about 300×10^{-6} at a stress of 7.5 MPa (1100 psi). It should be noted, however, that other workers[6.119] reported that hydrated cement

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paste is damaged progressively and without the discontinuity point being a significant feature.

The ultimate failure under the action of a uniaxial compression is either a tensile failure of cement crystals or of bond in a direction perpendicular to the applied load, or is a collapse caused by the development of inclined shear planes.[6.20] It is probable that ultimate strain is the criterion of failure, but the level of strain varies with the strength of concrete: the higher the strength the lower the ultimate strain. While actual values depend on the method of test, typical values are as given in Table 6.1.

Table 6.1. Typical Values of Compressive Strain at Failure

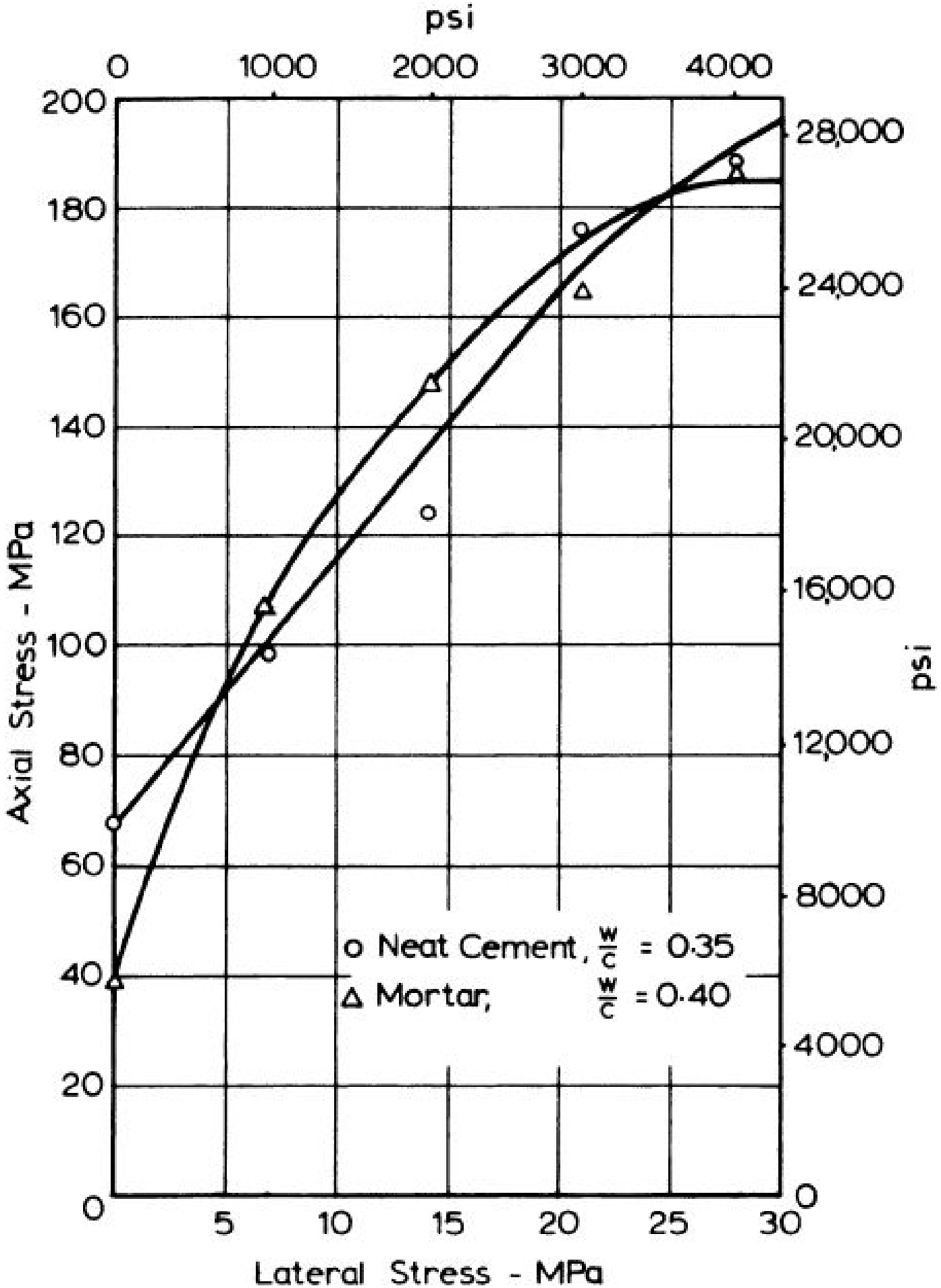
Nominal compressive strength		*Maximum strain at failure* 10^{-3}
MPa	*psi*	
7	1000	4.5
14	2000	4
35	5000	3
70	10 000	2

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Failure under multiaxial stress

Under triaxial compression, when the lateral stresses are high, failure must take place by crushing: the mechanism is, therefore, different from that described above, the behaviour of concrete changing from brittle to ductile. An increase in lateral compression increases the axial load that can be sustained, as shown, for instance, in Fig. 6.24.[6.26] With very high lateral stresses, extremely high strengths have been recorded[6.11] (Fig. 6.25). It should be noted that, if the development of pore water pressure in concrete is limited by allowing the displaced pore water to escape through the loading platens, then the apparent strength is higher.[6.75] Thus, in practice, a possible development of pore pressure is of importance.[6.84]

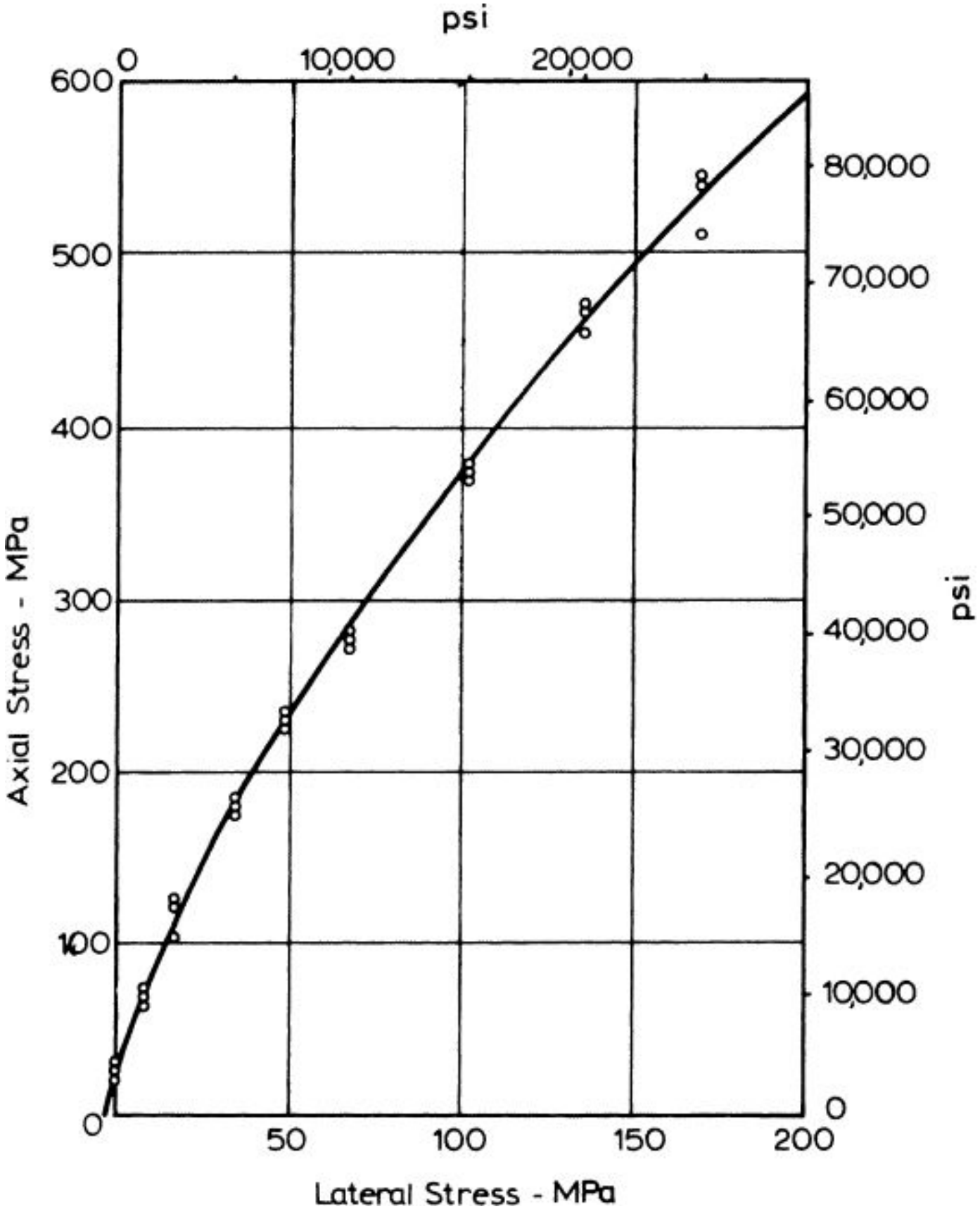
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Fig. 6.24. Influence of lateral stress on the axial stress at failure of neat cement paste and of mortar[6.26]

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Fig. 6.25. Influence of high lateral stress on the axial stress at failure of concrete[6.11]

A confining lateral stress of 520 MPa has been reported to lead to an axial stress of 1200 MPa.[6.82] If the lateral compressive stress increased progressively with the increase in the axial stress, even higher values of the axial stress can be reached: 2080 MPa has been reached, coupled with a large decrease in porosity.[6.82]

A lateral tensile stress has a similar influence but, of course, in the opposite direction.[6.11] This behaviour agrees well with the theoretical considerations on the previous page.

In practice, failure of concrete takes place over a range of stresses rather than as an instantaneous phenomenon, so that ultimate failure is a function of the type of loading.[6.19] This is of especial interest when repeated loading is applied – a condition frequently met with in practice. Fatigue strength of concrete is considered in Chapter 7.

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A general biaxial stress interaction curve is shown in Fig. 6.26.[6.78] A large interaction is observed when there is a considerable frictional restraint at the platens but, when the end restraint of the specimen is effectively eliminated (e.g. by the use of steel brush platens, see p. 589), the effect is much smaller. It can be seen from Fig. 6.26, that, under a biaxial stress $\sigma_1 = \sigma_3$, the strength is only 16 per cent higher than in uniaxial compression; biaxial tensile strength is no different from uniaxial tensile strength.[6.78] These findings were confirmed by other workers.[6.9,6.54,6.86] However, some differences were observed due to the variation in the rate of loading and the type of coarse aggregate in the concrete.[6.86] Experimental data on interaction are plotted in Fig. 6.27; these were obtained with steel brush platen loading and by the use of fluid membranes and solid platens.[6.46] Some contradictory data of other investigators can be explained by the use of uncertain end restraints.

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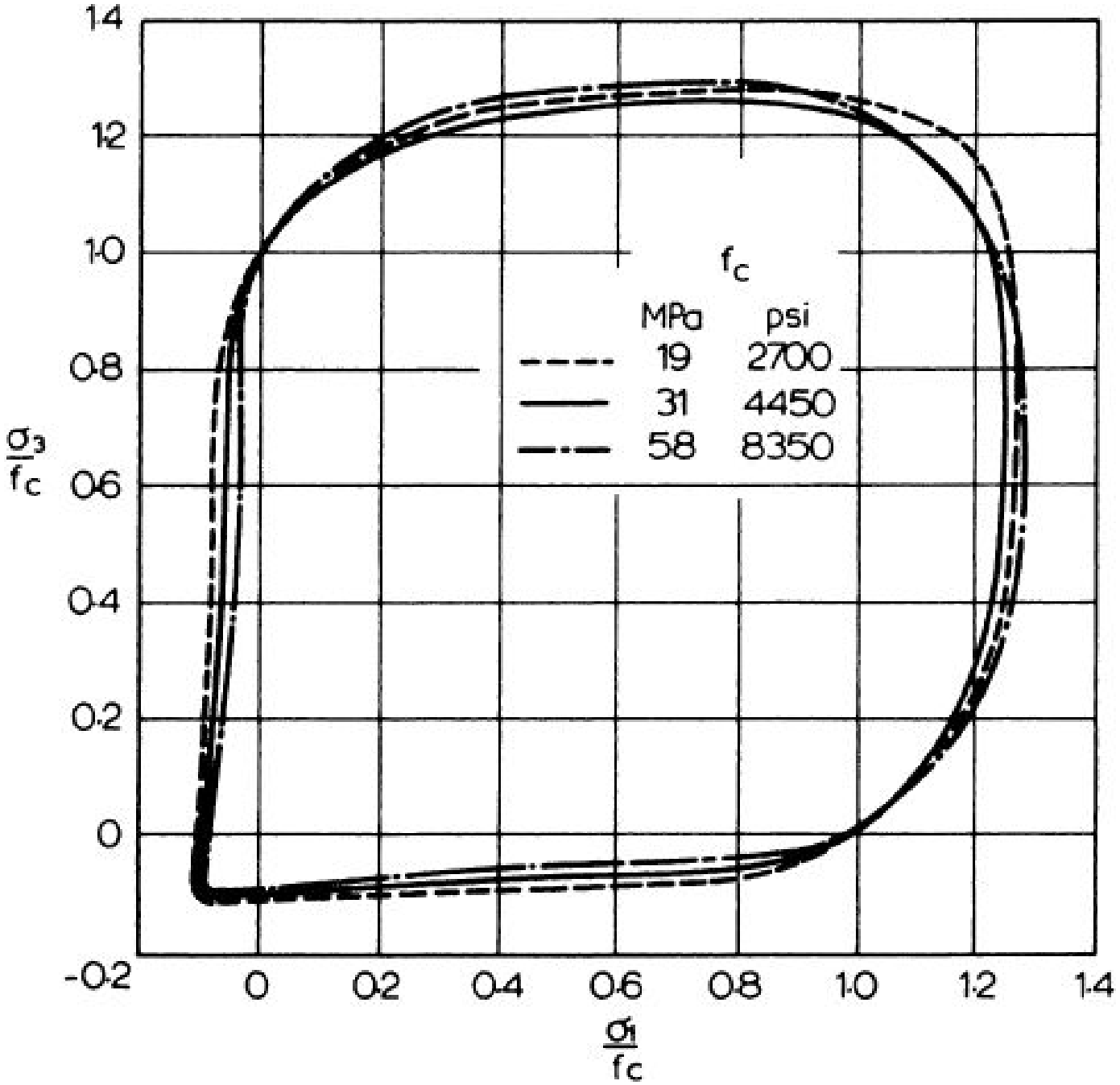


Fig. 6.26. Interaction curve for biaxial stress when the end restraint is effectively eliminated[6.78] (σ_1 and σ_3 are the biaxial stresses applied)

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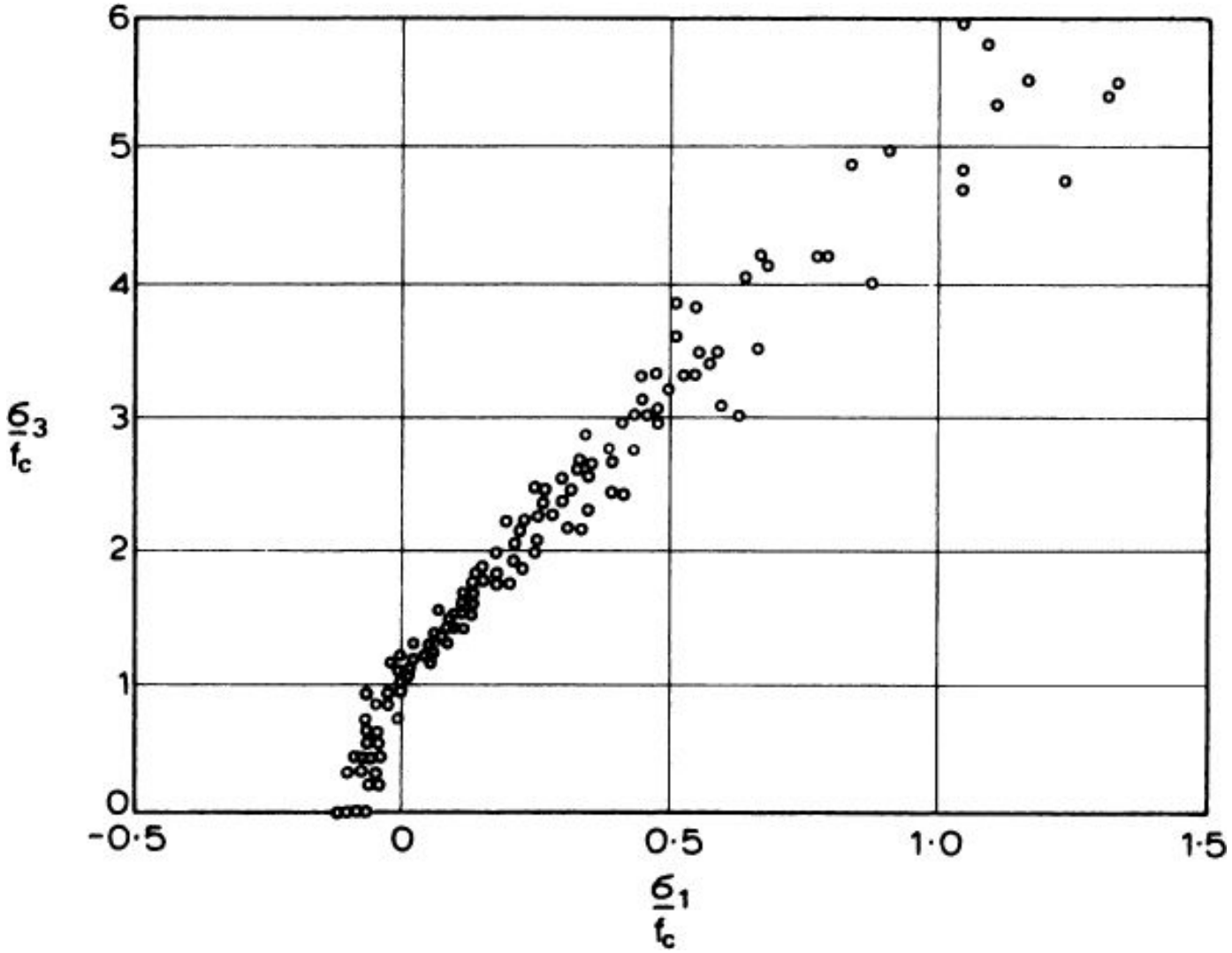


Fig. 6.27. Strength of concrete under multiaxial stress as measured by various investigators. Wet or air-dried concrete[6.46] (f_c = compressive strength)

The level of uniaxial compressive strength virtually does not affect the shape of the curve or the magnitude of the values given by it;[6.78] the prism strength range tested was 19 to 58 MPa (2700

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to 8350 psi) and both the water/cement ratio and cement content varied widely. However, in compression–tension and in biaxial tension, the relative strength at any particular biaxial stress combination decreases as the level of uniaxial compressive strength increases.[6.78] This accords with the general observation that the ratio of uniaxial tensile strength to uniaxial compressive strength decreases as the compressive strength level rises (see p. 311); in these tests, the ratio was 0.11, 0.09 and 0.08 at a uniaxial compressive strength level of 19, 31 and 58 MPa (2700, 4450 and 8350 psi) respectively.[6.78]

Generally, triaxial compression increases the strength of weaker or leaner concrete relatively more than that of stronger or richer concrete.[6.47] For the range of conventional concretes, Hobbs[6.47] found that, under triaxial compression, the major principal stress at failure, σ_1, can be expressed, on average, as:

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$$\frac{\sigma_1}{f_{cyl}} = 1 + 4.8\frac{\sigma_3}{f_{cyl}}$$

where σ_3 = minor principal stress, and

f_{cyl} = cylinder strength.

The limited information on lightweight aggregate concrete suggests that the influence of σ_3 is not as large as with normal aggregates;[6.46] therefore, the coefficient 4.8 in the above equation can be reduced to about 3.2.

The combined strength results for concretes in triaxial compression and in biaxial compression plus tension, may be represented[6.47] by the equation:

$$\frac{\sigma_1}{f_{cyl}} = \left(1 + \frac{\sigma_3}{f_t}\right)^n \quad (1)$$

where $f_t = 0.018 f_{cyl} + 2.3$ = tensile strength, and (2)

$$n = \frac{7.7}{f_{cyl}} + 0.4 \quad (3)$$

all values being averages in MPa, and compression being taken as positive.

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The values given in Eqs (2) and (3) apply to conventional concretes only, and not to neat cement pastes or mortars.

Substituting equations (2) and (3) into equation (1), but using the lower bound, and not average, values, yields the failure criterion for conventional concretes:

$$\frac{\sigma_1}{f_{cyl}} = \left(1 + \frac{\sigma_3}{0.014 f_{cyl} + 2.16}\right)^{\frac{7.1}{f_{cyl}} + 0.38}$$

This equation is plotted in Fig. 6.28 for various values of the cylinder strength, f_{cyl}. The generality of this equation should not be overestimated because, as Hobbs[6.47] points out, the tensile strength and compressive strength of concrete are not equally affected by the aggregate type and grading and by the direction of the applied stress relative to the direction of casting. In each case, the tensile strength is more sensitive. It should also be noted that the intermediate principal stress, σ_2, affects the value of σ_1.[6.85]

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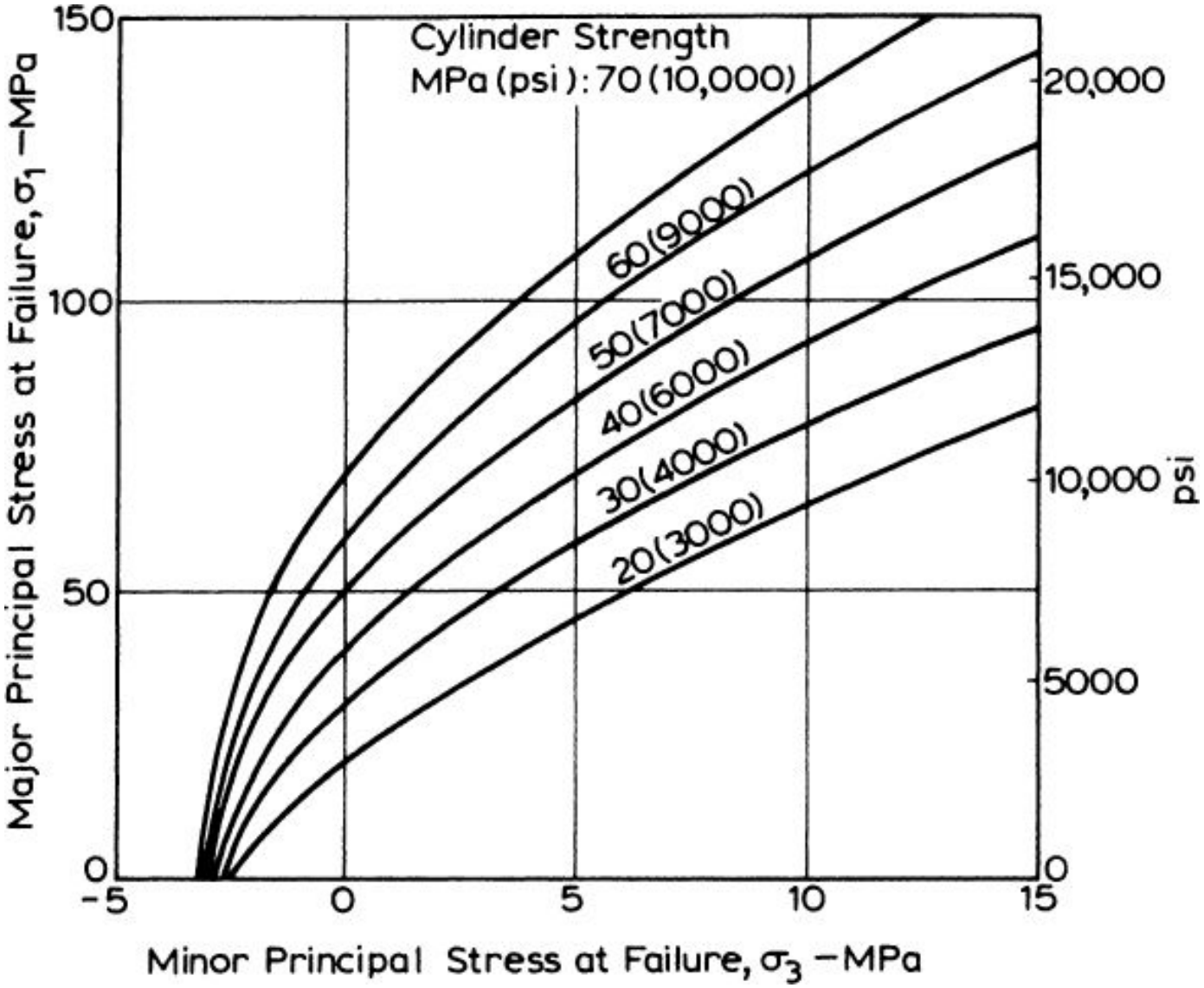


Fig. 6.28. Failure stresses in concrete under biaxial stress[6.47]

The preceding discussion has shown that, while strength of concrete is an inherent property of the material, as measured in practice it is also a function of the stress system which is acting. Mather[6.77] pointed out that, ideally, it should be

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possible to express the failure criteria under all possible stress combinations by a single stress parameter, such as strength in uniaxial tension. However, such a solution has not yet been found.

Berg[6.56] developed an equation of strength for concrete whose parameters are: the stress at the initiation of crack propagation, the splitting (tensile) strength, and the uniaxial compressive strength. This equation can be used for an analytical evaluation of the failure of concrete under combined states of stress but it ceases to apply when the tensile strength cannot be reached. Other approaches[6.79] also have somewhat limited validity.

Full understanding of the failure behaviour of concrete requires consideration of fracture energy, that is the energy absorbed in a unit area of crack surface. This is a subject of study of fracture mechanics, dealt with in specialist publications, e.g. refs 6.87 and 6.88. However, fracture mechanics has so far not succeeded in developing

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material parameters which can adequately quantify the resistance of concrete to cracking.

Microcracking

Because failure of concrete is the consequence of cracking, it is useful to consider this topic in some detail. In this section, only microcracking is considered. More general aspects of cracking will be discussed in Chapter 10 as this requires a prior consideration of the stress–strain relation of concrete.

Investigations have shown that very fine cracks at the interface between coarse aggregate and cement paste exist, in fact, even prior to application of the load on concrete.[6.76] They are probably due to the inevitable differences in mechanical properties between the coarse aggregate and the hydrated cement paste, coupled with shrinkage or thermal movement. Microcracking has been observed not only in normal strength concrete but also in wet-cured concrete with a water/cement ratio as low as 0.25, which

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had never been subjected to loading.[6.92] According to Slate and Hover,[6.91] pre-loading microcracks are largely responsible for the low tensile strength of concrete.

Microcracks have not been universally defined in terms of size, but an upper limit of 0.1 mm has been suggested;[6.91] this is the smallest size which can typically be detected by the naked eye. For engineering purposes, a lower limit can be taken as the smallest crack which can be observed using an optical microscope. As an increasing load is being applied, these microcracks remain stable up to about 30 per cent, or more, of the ultimate load and then begin to increase in length, width, and number. The overall stress under which they develop is sensitive to the water/cement ratio of the paste. This is the stage of slow crack propagation.

Upon further increase in load, up to between 70 and 90 per cent of the ultimate strength, cracks open through the mortar (cement paste and fine aggregate); they bridge the bond cracks so that a

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continuous crack pattern is formed.[6.76] This is the fast crack propagation stage. The stress level at the onset of this stage is higher in high strength concrete than in normal concrete.[6.90] The increase in the cumulative length of the microcracks is large; this was measured using neutron radiography.[6.116] However, high strength concrete exhibits a lower cumulative length of microcracks than normal strength concrete.[6.90]

The onset of the fast crack propagation stage corresponds to the discontinuity point in the volumetric strain (referred to on p. 422). If the load is sustained, failure may take place with time. This occurs both in normal strength and in high strength concrete.[6.90]

Interesting results of measurement of crack length are shown in Fig. 6.29.[6.37] It can be seen that there was very little increase in the total length between the beginning of loading and a stress equal to about 0.85 of the prism strength.[6.37] A further increase in stress resulted

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in a large increase in the total length of cracks. At a stress/strength ratio of about 0.95, not only interface (bond) cracks but also mortar cracks were present, and many cracks tended to become oriented roughly parallel to the direction of the applied load. Once the specimen reached the descending part of the stress/strain curve the rate of increase in the crack length and width became large.

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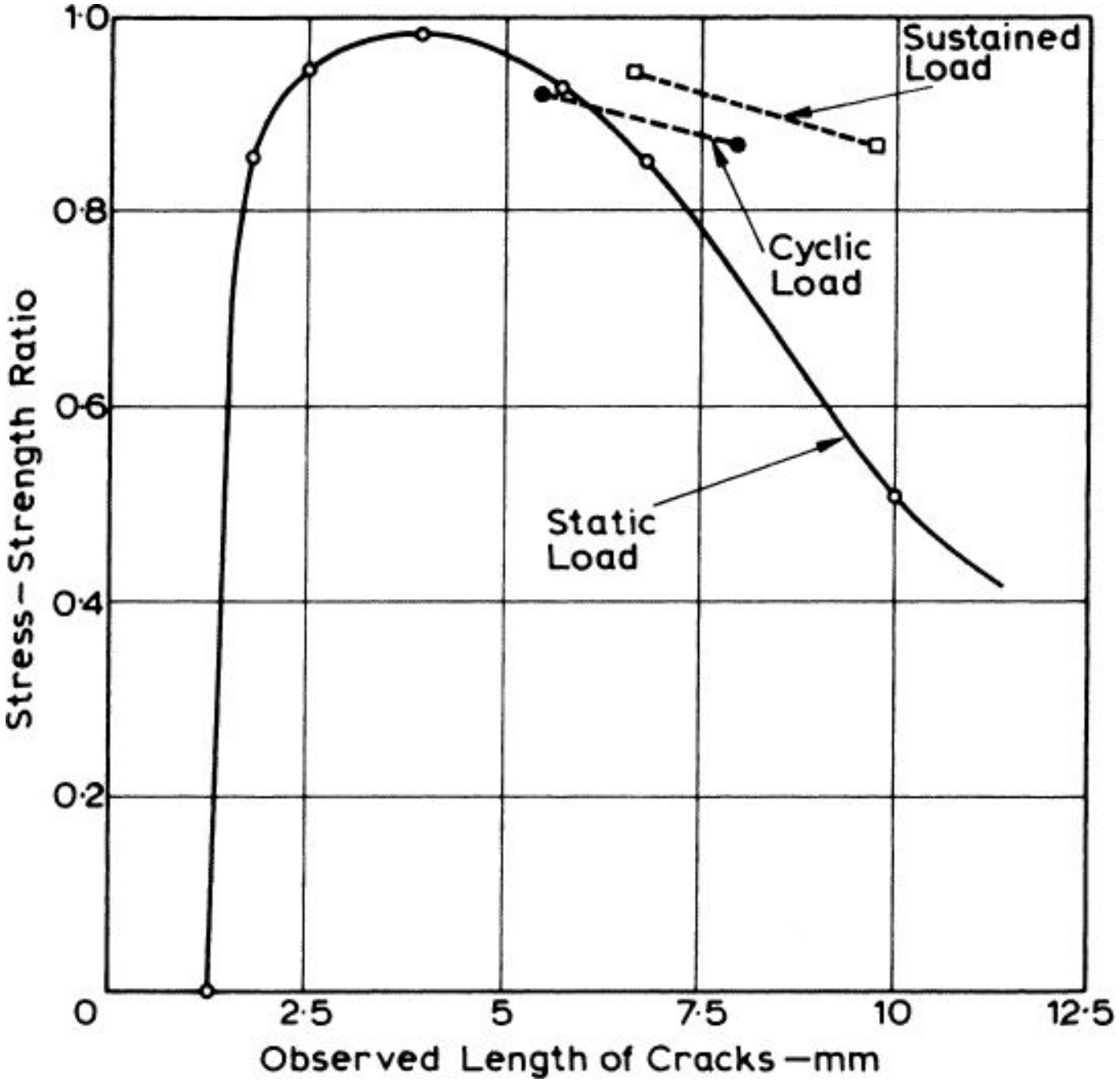


Fig. 6.29. Relation between the observed length of cracks in an area of 100 mm^2 and the stress/strength ratio in compression (based on prisms)[6.37]

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Figure 6.29 also shows the crack development under a cyclic stress alternating between zero and 0.85 of the prism strength. Immediately prior to failure, the cracks became longer and wider. Likewise, sustained loading at a stress/strength ratio of 0.85 led to an increase in cracking prior to failure.[6.37]

The preceding discussion has shown that microcracking is a general feature of concrete. As long as the cracks are stable, their presence is not harmful. Paradoxically, while the interface between coarse aggregate and the hydrated cement paste is the locus of early microcracks, it is the presence of coarse aggregate particles that prevents the opening of a single wide crack: these particles act as microcrack arrestors. The heterogeneity of concrete is thus beneficial. The aggregate–paste bond surfaces form all the possible angles with the direction of the external force. As a result, the local stress varies substantially above and below the nominal applied stress. The

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aggregate–paste interface will be discussed in the next section.

The existence of submicrocracks, defined as cracks which can be detected using a scanning electron microscope at a magnification of at least 1250, has been reported.[6.111] This is not surprising because, in concrete, there are discontinuities at any level, however small. There is no evidence, however, that submicrocracks are a factor in the strength of concrete.

Aggregate–cement paste interface

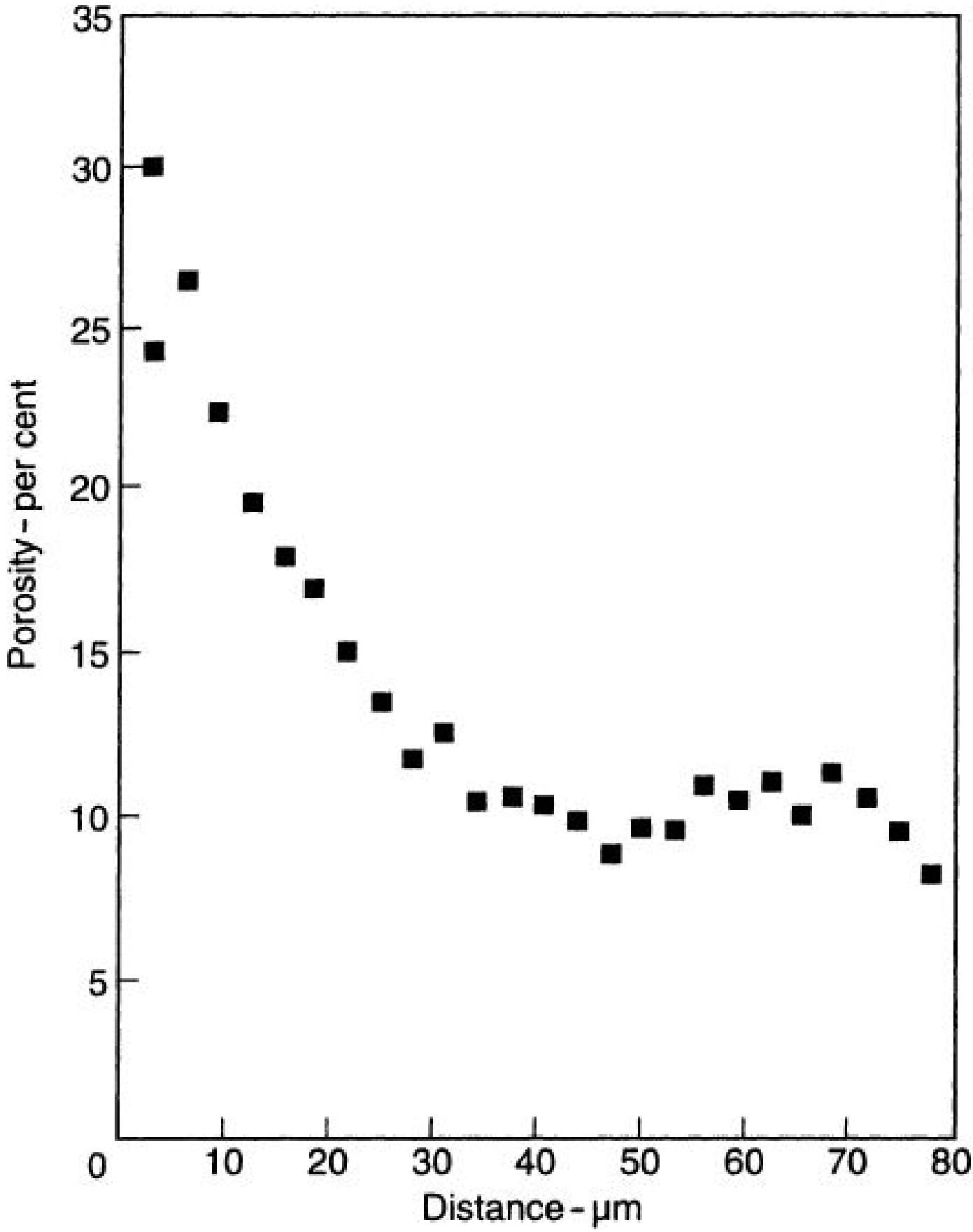
The observation that microcracking is initiated at the interface between coarse aggregate and the surrounding mortar and that, at failure, the crack pattern includes the interface, points to the importance of this part of the concrete. It is therefore necessary to understand the properties and behaviour of the *interface zone*, sometimes called the *transition zone*.

The first fact to note is that the microstructure of the hydrated cement paste in the immediate vi-

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cinity of coarse aggregate particles differs from that of the bulk of the cement paste. The main reason for this is that, during mixing, dry cement particles are unable to become closely packed against the relatively large particles of aggregate. This situation is similar to the 'wall-effect' at the surface of cast concrete surfaces (see p. 611) although on a much smaller scale. There is thus less cement present to hydrate and fill the original voids. In consequence, the interface zone has a much higher porosity than the hydrated cement paste further away from the coarse aggregate[6.94] (see Fig. 6.30). The influence of porosity on strength, discussed earlier in this chapter, explains the weakness of the interface zone.

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Fig. 6.30. Variation in porosity of hydrated cement paste with distance from the surface of an aggregate particle (based on ref. 6.94)

The microstructure of the interface zone is as follows. The surface of the aggregate is covered with a layer of oriented crystalline $Ca(OH)_2$, about 0.5 μm thick, behind which there is a layer of C-S-H of about the same thickness. This is referred to as a duplex film. Moving further away from the aggregate, there is the main interface zone, some 50 μm thick, containing products of hydration of cement with larger crystals of $Ca(OH)_2$ but without any unhydrated cement.[6.57]

The significance of the above distribution is twofold. First, the complete hydration of cement indicates that the water/cement ratio at the interface is higher than elsewhere. Second, the presence of large crystals of $Ca(OH)_2$ indicates that the porosity at the interface is higher than elsewhere; this confirms the 'wall effect' referred to earlier.

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The strength of the interface zone can increase with time in consequence of a secondary reaction between the $Ca(OH)_2$ present there and pozzolana. Silica fume, which is very much finer than cement particles, is particularly effective. This topic is discussed in Chapter 13.

Although the interface zone of primary interest is that at the surface of coarse aggregate particles, such a zone is also formed around the fine aggregate particles.[6.93] Here, the thickness of the interface zone is smaller but the surface effects originating from the fine particles interfere with those of the coarse aggregate and thus affect the overall extent of the interface zone.[6.93]

The mineralogical characteristics of the fine aggregate affect the microstructure of the transition zone: in the case of limestone, there is chemical reaction between the limestone and the cement paste and, consequently, a dense interface zone is formed.[6.95]

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As far as lightweight aggregate is concerned, if it has a dense outer layer, then the situation at the interface is the same as with normal weight aggregate.[6.89] However, lightweight aggregate with a more porous outer layer, which encourages the migration of mobile ions towards it,[6.96] leads to the formation of a more dense interface zone and also to improved mechanical interlocking of the aggregate particles and the hydrated cement paste.[6.89]

The study of the interface zone in actual concrete is difficult. Consequently, experiments on the interface between a single rock particle and cement paste have been resorted to. However, the results of such tests may be misleading as they do not include the effects of the interference of other coarse aggregate particles[6.94] or even of fine aggregate. Moreover, the laboratory-made artefact of a single particle covered by cement paste has not undergone the process of mixing, in which shearing action influences the microstructure of the cement paste at the time of setting. Further-

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more, in actual concrete, bleeding may result in water-filled voids on the underside of coarse aggregate particles, and it is at this type of interface that massive crystals of $Ca(OH)_2$ have been observed. More generally, the interface between the cement paste and the coarse aggregate is a zone of stress concentrations arising from the difference in the modulus of elasticity and the Poisson's ratio of the two materials.

Effect of age on strength of concrete

The relation between the water/cement ratio and the strength of concrete applies to one type of cement and one age only, and also assumes wet-curing conditions. On the other hand, the strength versus gel/space ratio relationship has a more general application because the amount of gel present in the cement paste at any time is itself a function of age and type of cement. The latter relation thus allows for the fact that different cements require a different length of time to produce the same quantity of gel.

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The rate of gain of strength of different cements was discussed in Chapter 2, and Figs 2.1 and 2.2 show typical strength–time curves. The influence of the curing conditions on the development of strength is considered in Chapter 7, but here we are concerned with the practical problem of strength of concrete at different ages.

In concrete practice, the strength of concrete is traditionally characterized by the 28-day value, and some other properties of concrete are often referred to the 28-day strength. There is no scientific significance in the choice of the age of 28 days; it is simply that early cements gained strength slowly and it was necessary to base the strength description on concrete in which a significant hydration of cement had already taken place. The specific choice of a multiple of weeks was, in all likelihood, made so that testing, like placing, would fall on a working day. In modern Portland cements, the rate of hydration is much greater than in the past, both because they have a much higher fineness and because they have a

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higher C_3S content. This is, however, not necessarily the case with all blended cements.

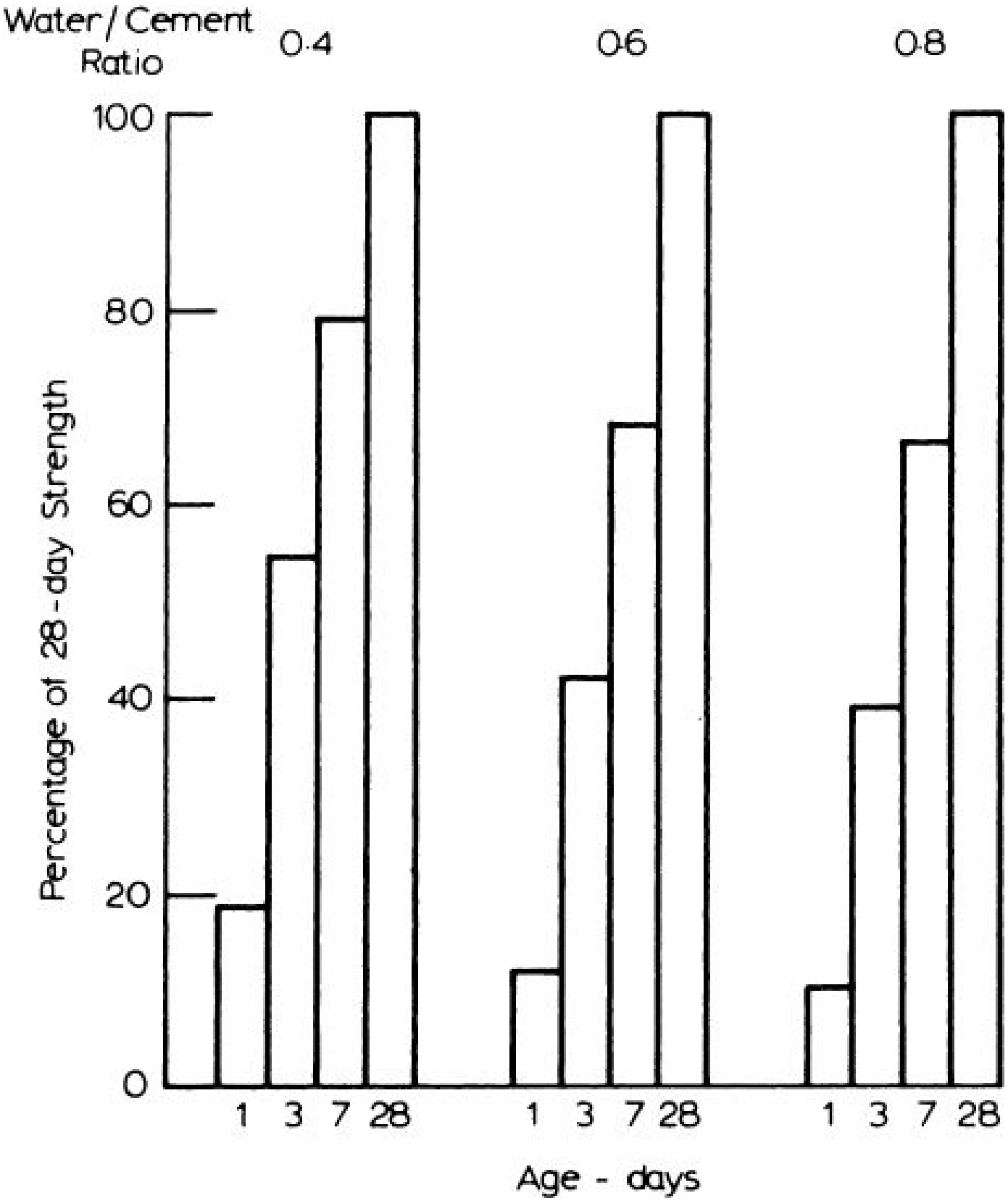
It is arguable that a shorter period than 28 days could be used for the characterization of strength, but the age of 28 days seems to have acquired an immutable position. Thus, compliance with the specification is almost invariably laid down in terms of the 28-day strength. If, for some reason, the 28-day strength is to be estimated from the strength determined at an earlier age, say 7 days, then the relation between the 28-day and the 7-day strengths has to be established experimentally for the given mix. For this reason, the various expressions for the ratio of the two strengths are no longer thought to be reliable, and they will not be discussed. The consequences of the change in the strength-development characteristics which took place in the 1970s will be discussed on p. 336.

Not only the properties of cement but the water/cement ratio also affect the rate of gain of strength of concrete. Mixes with a low water/

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cement ratio gain strength, expressed as a percentage of long-term strength, more rapidly than mixes with higher water/cement ratios[6.83] (Fig. 6.31). This is because in the former case the cement grains are closer to one another and a continuous system of gel is established more rapidly. It should be noted that in a hot climate the early strength gain is high and the ratio of the 28-day to 7-day strengths tends to be lower than in cooler weather. This is also the case with some lightweight aggregate concretes.

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Fig. 6.31. Relative gain of strength with time in concretes with different water/cement ratios, made with ordinary Portland cement[6.83]

Knowledge of the strength–time relation is of importance when a structure is to be put into use, that is, subjected to full loading, at a later age: in such a case, the gain in the strength after the age of 28 days can be taken into account in design. In some other situations, for example in precast or in prestressed concrete, or when early removal of formwork is required, the strength at an early age needs to be known.

Data on the development of strength of concretes made with water/cement ratios of 0.40, 0.53, and 0.71 are shown in Fig. 6.32 for concretes made with Type I cement in 1948 are continuously kept wet.[6.117]

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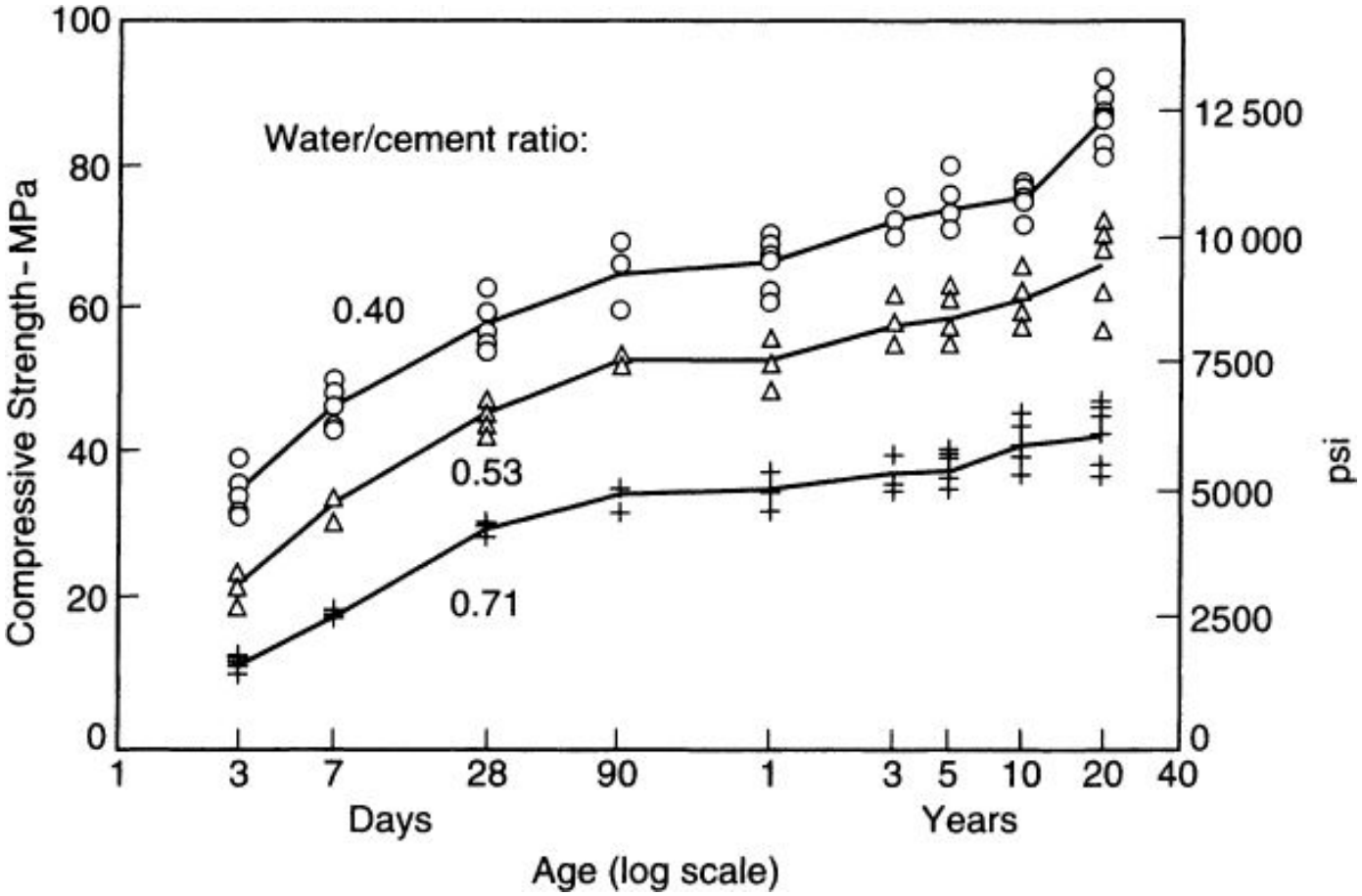


Fig. 6.32. Development of strength of concrete (determined on 150 mm (6 in.) modified cubes) over a period of 20 years; storage under moist conditions[6.117]

As far as the really long-term strength is concerned, American Portland cements made at the beginning of the century (which had a high C_2S content and a low specific surface) led to an increase in the strength of concrete stored outdoors which was proportional to the logarithm of age

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up to 50 years. The 50-year strength was typically 2.4 times the 28-day strength. However, cements made since the 1930s (with a lower C_2S content and a higher specific surface) reach their peak strength between 10 and 25 years, and thereafter undergo some retrogression of strength.[6.48] German Portland cements made in 1941, when used in concrete stored outdoors, led after 30 years to a strength 2.3 times the 28-day strength. The relative increase in strength was greater at higher water/cement ratios. By comparison, Portland blast-furnace cement led to a 3.1-fold increase.[6.49]

Maturity of concrete

The fact that the strength of concrete increases with the progress of hydration of cement, coupled with the fact that the rate of hydration of cement increases with an increase in temperature, leads to the proposition that strength can be expressed as a function of the time–temperature combination. The influence of a steady temperature on the development of strength is shown in Fig. 6.33,

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obtained from tests on specimens cast, sealed and cured at the indicated temperatures.[6.11] The effect of the temperature at the time of setting, with further storage at some other temperature, is considered on p. 362.

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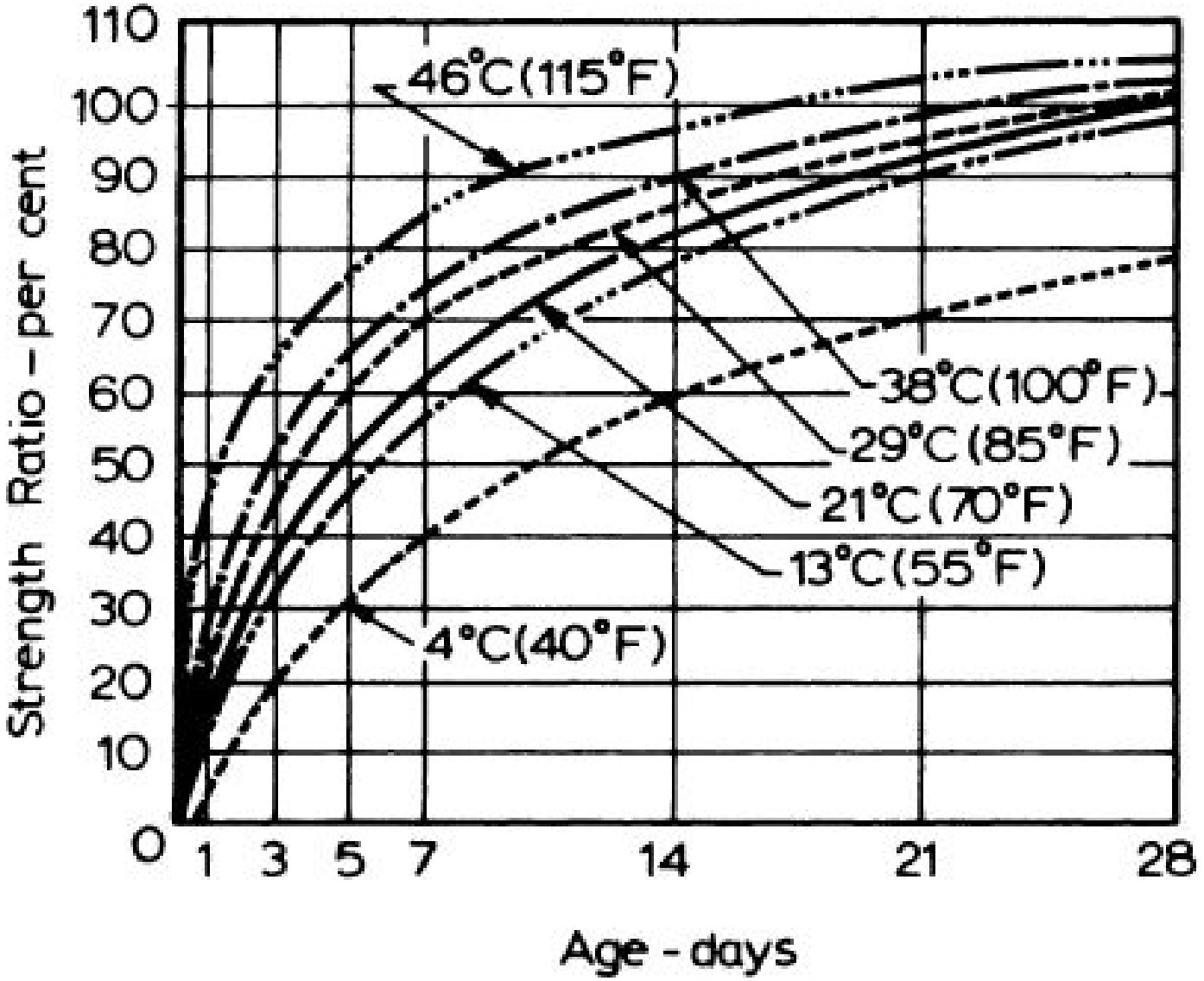


Fig. 6.33. Ratio of strength of concrete cured at different temperatures to the 28-day strength of concrete cured at 21 °C (70 °F) (water/cement ratio = 0.50; the specimens were cast, sealed, and cured at the indicated temperature)[6.11]

As the strength of concrete depends on both age and temperature, we can say that strength is

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a function of ∑(time interval × temperature), and this summation is called maturity. The temperature is reckoned from a datum found experimentally to be between –12 and –10 °C (11 and 14 °F). This is because at temperatures below the freezing point of water and down to about –12 °C (11 °F) concrete shows a small increase in strength with time, but the low temperature must not be applied, of course, until after the concrete has set and gained sufficient strength to resist damage due to the action of frost; a 'waiting period' of 24 hours is usually required. Below –12 °C (11 °F) concrete does not appear to gain strength with time.

The datum temperature generally used is –10 °C (11 °F). The appropriateness of this value for ages up to 28 days[6.101] and for temperatures in the range of 0 to 20 °C (32 to 68 °F) has been confirmed; for higher temperatures, a higher datum may be appropriate.[6.100] ASTM C 1074-04 describes a method of determination of the datum temperature.

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Maturity is measured in degree C-hours (degree F-hours) or degree C-days (degree F-days). Figures 6.34 and 6.35 show that compressive and tensile strengths plotted against the logarithm of maturity give a straight line.[6.50] It is, therefore, possible to express strength S_2 at any maturity as a percentage of strength of concrete at any other maturity S_1; the latter is often taken as 19 800 °C h (35 600 °F h), being the maturity of concrete cured at 18 °C (64 °F) for 28 days. This ratio of strengths, expressed as a percentage, can then be written as:

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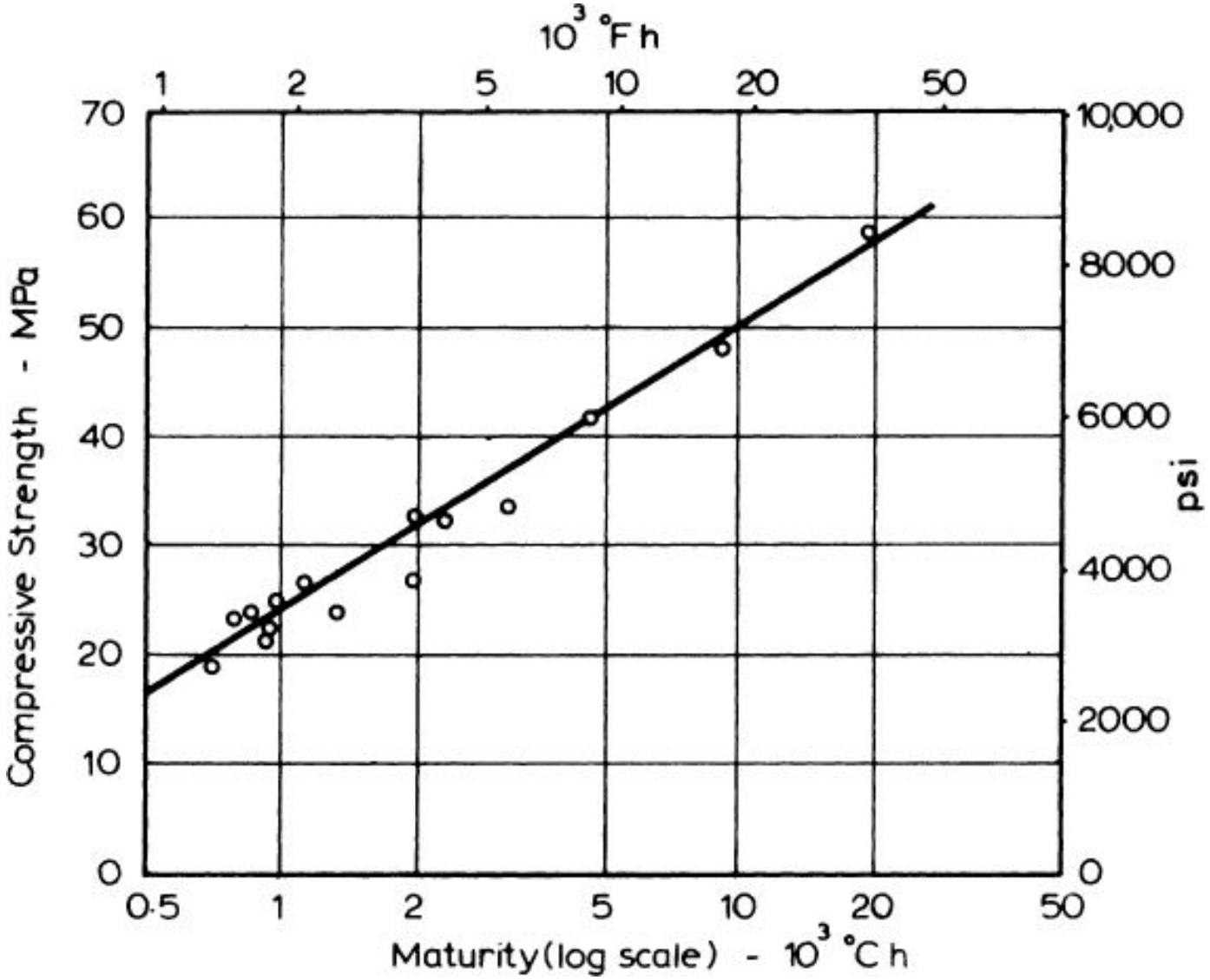


Fig. 6.34. Relation between logarithm of maturity and compressive strength of cubes[6.42]

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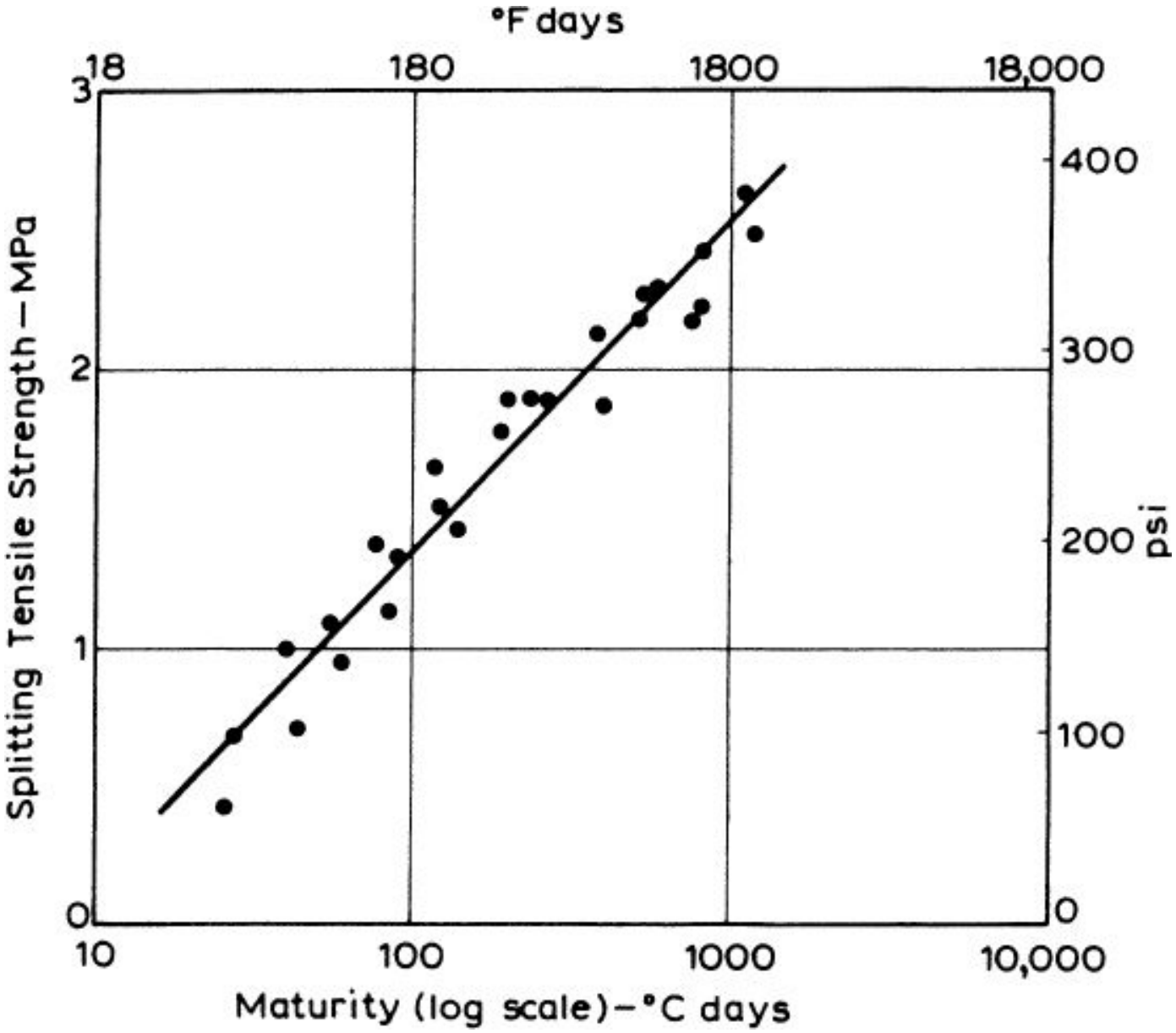


Fig. 6.35. Relation between logarithm of maturity and splitting strength (tests carried out at 2, 13, and 23 °C (35, 55, and 73 °F) up to 42 days)[6.50]

$$S_1/S_2 = A + B \log_{10}(\text{maturity} \times 10^{-3}).$$

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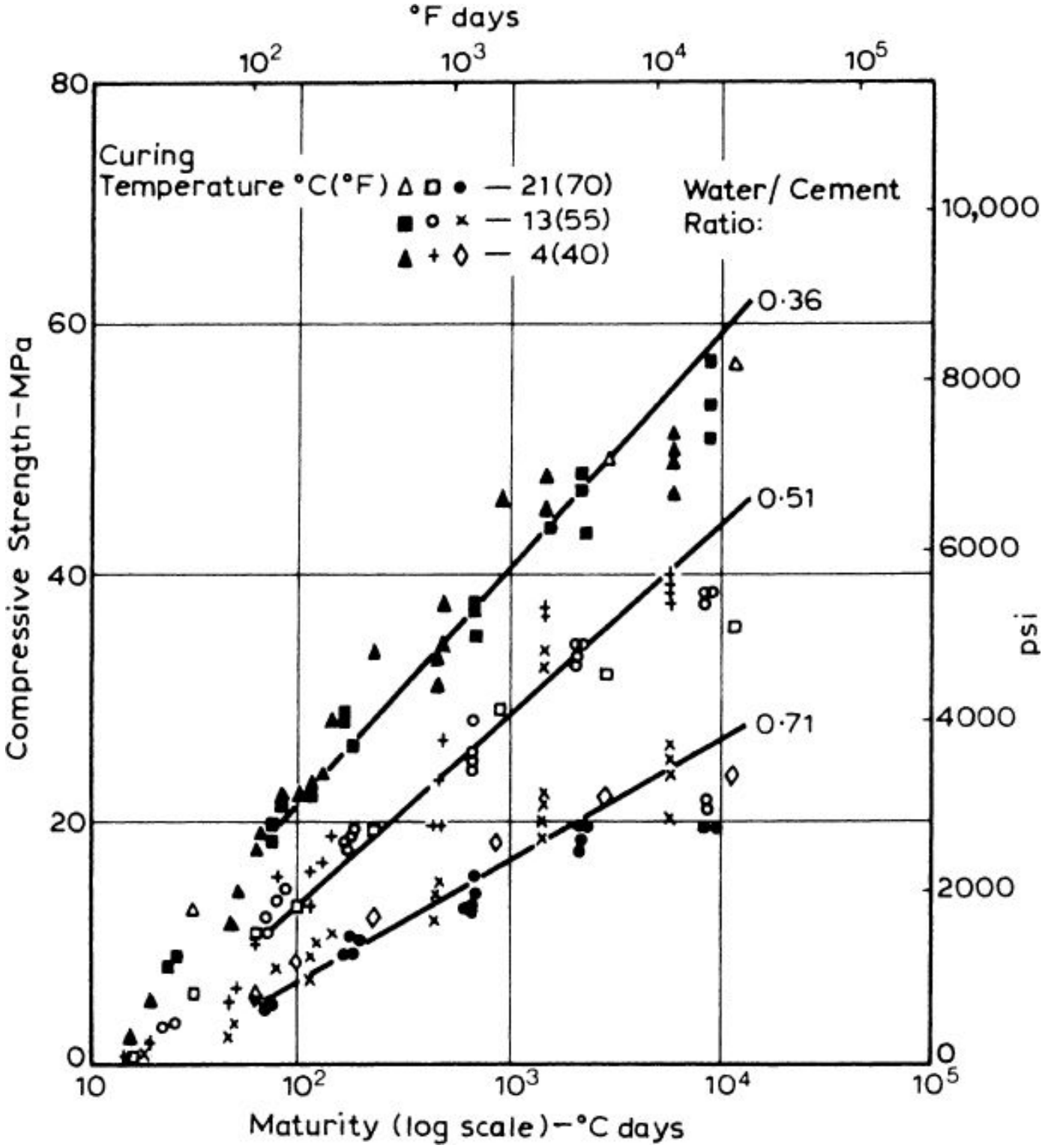
The values of the coefficients *A* and *B* depend on the level of strength of the concrete, that is on the water/cement ratio; those suggested by Plowman[6.42] are given in Table 6.2.

Table 6.2. Plowman's Coefficients for the Maturity Equation[6.42]

Strength after 28 days at 18 °C (64 °F) (maturity of 19 800 °C h (35 600 °F h))		*Coefficient*		
		B	*A*	
MPa	*psi*		*for units °C h*	*for units °F h*
<17	<2500	68	10	−7
17–35	2500–5000	61	21	6
35–52	5000–7500	54	32	18
52–69	7500–10 000	46.5	42	30

From Fig. 6.36, it can be seen that the linearity of the relation between strength and the logarithm of maturity applies only above a certain minimum maturity. The same figure shows that the relation depends on the water/cement ratio; it also depends on the type of cement used, especially if blended.

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Fig. 6.36. Relation between compressive strength of ordinary Portland (Type I) cement concrete and maturity for the data of Gruenwald[6.51] as treated by Lew and Reichard[6.55]

Furthermore, the early temperature also affects the precise strength–maturity relation, including its shape.[6.43] In particular, the effects of a period of exposure to a higher temperature are not the same when this occurs immediately after casting or later in the life of the concrete. Specifically, early high temperature leads to a lower strength for a given total maturity than when heating is delayed for at least a week or is absent. Concrete stored at 60 to 80 °C (140 to 170 °F) was found to have a long-term strength of about 70 per cent of the strength of concrete stored at 20 °C (68 °F), but the long-term strength was reached faster at the higher temperature.[6.102] The influence of initial temperature on the late-age strength at a given maturity was confirmed by Carino.[6.99] This is of interest in connection with

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steam curing. The general topic of influence of temperature on strength is considered in Chapter 8.

The fact that the original strength–maturity relation is not applicable over a wide range of conditions has encouraged some investigators to develop ‘improved’ maturity functions. Some of these are indeed improvements, but at the expense of introducing complications in the development and use of the functions. Other modified maturity functions offer improved prediction of strength in one range of ages and temperatures but, in some other range, the prediction fits less well. One approach uses the conversion of a curing interval at any temperature to an equivalent interval at the reference temperature, usually 20 °C (68 °F). The concept used is that of equivalent age, that is, the age at the reference temperature at which the same proportion of the ultimate strength is reached as would occur at other temperatures.[6.97]

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Despite these criticisms and developments in laboratory methods, it is reasonable to maintain that the original maturity function, as proposed by Plowman,[6.42] is a useful tool for use in practice; ASTM Standards C 918-07 and C 1074-04 are helpful in this respect.

ASTM C 918-07 makes the important point that there is no simple relation between the strength of the concrete in the structure and the strength of companion specimens, however closely they are intended to simulate the concrete in situ; only an indication can be obtained. In view of this, ASTM C 918-07 considers that the use of the maturity equation, developed from tests on standard compressive strength test specimens, is as good a method of estimating the potential strength of concrete at any required age as direct strength determination. The compressive strength specimens must be tested at ages from not less than 24 hours and extending up to the age at which the strength estimate is required, usually 28 days. The maturity relation is established from

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a plot of strength versus the logarithm of maturity. The slope of this line, b, makes it possible to estimate the strength S_2 at maturity m_2, from the strength S_1 at maturity m_1, using the equation:

$$S_2 = S_1 + b(\log m_2 - \log m_1).$$

Clearly, the relation applies only to concrete of the given composition.

If it is intended to estimate the strength of concrete with a known temperature history, ASTM C 1074-04 provides for the development and use of a maturity function. This is of value when a decision has to be taken on the removal of formwork and falsework (shoring) or on post-tensioning in prestressed concrete or on termination of cold-weather protection.

Maturity meters are available commercially; they are clock-driven temperature gauges, inserted into concrete, which integrate the temperature of the concrete with respect to time and give a read-out in degree C-hours. The use of such

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meters removes the uncertainty about strength in the periods of variable temperature (which can occur accidentally even in a precast concrete factory) as the meters determine the actual temperature of the concrete and can be located in temperature-sensitive parts of the concrete.[6.98]

The maturity equation should be used for wet-cured concrete only.[6.44] Attempts to allow for the relative humidity under other conditions of storage have been made[6.101] but they are unlikely to be of value as the effect of ambient relative humidity depends on the size and shape of the concrete element.

Relation between compressive and tensile strengths

The compressive strength of concrete is its property commonly considered in structural design but for some purposes the tensile strength is of interest; examples of these are the design of highway and airfield slabs, shear strength, and res-

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istance to cracking. From the discussion of the nature of strength of concrete, it would be expected that the two types of strength are closely related. This is indeed the case but there is no direct proportionality, the ratio of the two strengths depending on the general level of strength of the concrete. In other words, as the compressive strength, f_c, increases, the tensile strength, f_t, also increases but at a decreasing rate.

A number of factors affect the relation between the two strengths. The beneficial effect of crushed coarse aggregate on flexural strength was discussed on p. 287, but it seems that the properties of fine aggregate also influence the f_t/f_c ratio.[6.27] The ratio is furthermore affected by the grading of the aggregate.[6.28] This is probably due to the different magnitude of the wall effect in beams and in compression specimens: their surface/volume ratios are dissimilar so that different quantities of mortar are required for full compaction.

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Age is also a factor in the relation between f_t and f_c: beyond about one month, the tensile strength increases more slowly than the compressive strength so that the ratio f_t/f_c decreases with time.[6.29,6.103] This is in agreement with the general tendency of the ratio to decrease with an increase in f_c.

The tensile strength of concrete can be measured by radically different tests, namely flexure, direct tension, and splitting, and the resulting values of strength are not the same, as discussed in Chapter 12. Consequently, the numerical value of the ratio of the tensile strength to the compressive strength is also not the same. Incidentally, the value of the compressive strength is also not unique but is affected by the shape of the test specimen (see Chapter 12). For these reasons, in expressing the ratio of the tensile to compressive strengths, the test method must be explicitly stated. An example of the relation between the splitting strength and the compressive strength of standard cylinders, obtained by Oluokun[6.106]

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from a wide range of tests by different investigators, is shown in Fig. 6.37. If the value of the flexural strength is of interest, a factor relating the splitting strength to flexural strength needs to be applied.[6.104]

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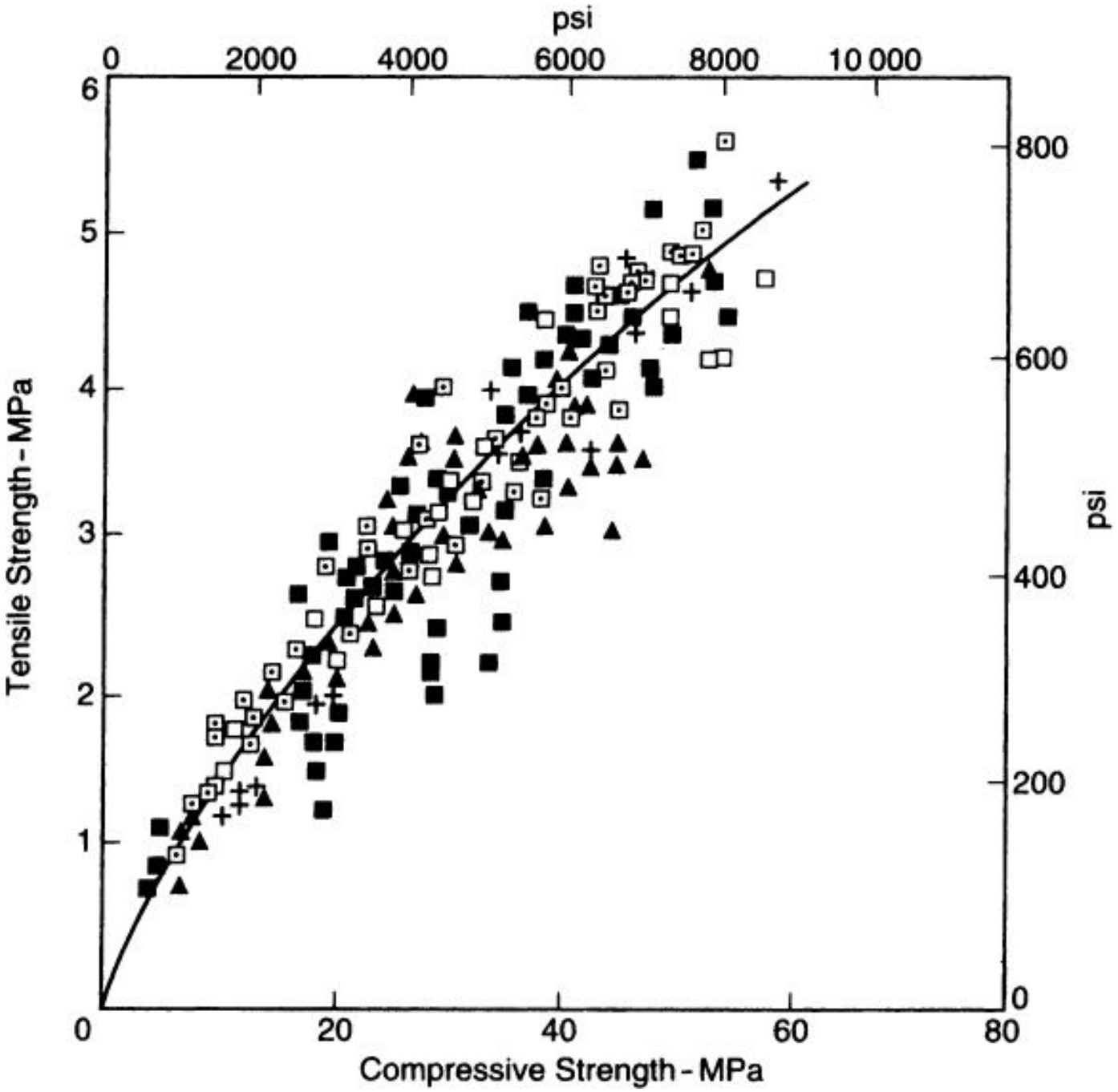


Fig. 6.37. Relation between splitting tensile strength and compressive strength (measured on standard cylinders) for tests by various investigators (collated by Oluokun)[6.106]

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The tensile strength of concrete is more sensitive to inadequate curing than the compressive strength,[6.30] possibly because the effects of non-uniform shrinkage of flexure test beams are very serious. Thus, air-cured concrete has a lower f_t/f_c ratio than concrete cured in water and tested wet. Air entrainment affects the f_t/f_c ratio because the presence of air lowers the compressive strength of concrete more than the tensile strength, particularly in the case of rich and strong mixes.[6.30] The influence of incomplete compaction is similar to that of entrained air.[6.31]

Lightweight concrete conforms broadly to the pattern of the relation between f_t and f_c for ordinary concrete. At very low strengths (say, 2 MPa (300 psi)) the ratio f_t/f_c can be as high as 0.3, but at higher strengths it is the same as for ordinary concrete. However, drying reduces the ratio by some 20 per cent so that in the design of lightweight concrete a reduced value of f_t/f_c is used.

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A number of empirical formulae connecting f_t and f_c have been suggested, many of them of the type:

$$f_t = k(f_c)^n$$

where k and n are coefficients. Values of n between $\frac{1}{2}$ and $\frac{3}{4}$ have been suggested. The former value is used by the American Concrete Institute, but Gardner and Poon[6.120] found a value nearer the latter, cylinders being used in both cases. Probably the best fit overall is given by the expression:

$$f_t = 0.3(f_c)^{2/3}$$

where f_t is the splitting strength, and f_c is the compressive strength of cylinders, both in mega-pascals. If the stress is expressed in pounds per square inch, the coefficient 0.3 is replaced by 1.7. The above expression was suggested by Raphael.[6.110] A modification by Oluokun[6.106] is:

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$$f_t = 0.2(f_c)^{0.7}$$

where the strengths are in megapascals; the coefficient becomes 1.4 in pounds per square inch.

An expression used in the British Code of Practice BS 8007 : 1987 (superseded by BS EN 1992-3 : 2006 Eurocode 2) is similar, namely:

$$f_t = 0.12(f_c)^{0.7}$$

bearing in mind that the compressive strength is determined on cubes (in megapascals); f_t represents the direct tensile strength.

The differences between the various expressions are not large. What is important, however, is that the power exponent used in the ACI Building Code 318-02[6.118] is too low so that the splitting strength is overestimated at low compressive strengths and underestimated at high compressive strengths.[6.105]

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Bond between concrete and reinforcement

Since structural concrete is, in the vast majority of cases, used with steel reinforcement, the strength of bond between the two materials is of considerable importance with respect to structural behaviour, including cracking due to shrinkage and early thermal effects. Bond arises primarily from friction and adhesion between concrete and steel, and from mechanical interlocking in the case of deformed bars. Bond may also be beneficially affected by the shrinkage of concrete relative to the steel.

In a structure, the bond strength involves not only the properties of the concrete but also other factors. These include the geometry of the reinforcement and of the structure such as the thickness of cover to the reinforcement. The state of the surface of the steel is also a factor. The presence of rust on the surface of the steel, provided the rust is well connected to the underlying steel, improves bond of plain bars and does not impair

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the bond of deformed reinforcement.[6.108] Coating by galvanizing or by epoxy affects the bond strength.

These considerations put the subject of bond largely outside the scope of the present book except in so far as the properties of concrete influence the bond strength which, incidentally, is not easily determined.

The critical property is the tensile strength of concrete. For this reason, design formulae for bond strength usually express it as being proportional to the square root of compressive strength. As shown earlier, the tensile strength of concrete is proportional to a somewhat higher power of the compressive strength, say about 0.7; consequently, the expressions used in the various codes are not a correct representation of the indirect dependence of the bond strength on the compressive strength of concrete. Nevertheless, bond strength of deformed steel bars has been shown to increase with an increase in compressive strength, albeit at a decreasing rate, for concrete

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strengths up to about 95 MPa (14 000 psi).[6.107,6.109]

A rise in temperature reduces the bond strength of concrete: at 200 to 300 °C (400 to 570 °F) there may be a loss of one-half of the bond strength at room temperature.

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Chapter 7. Further aspects of hardened concrete

In the preceding chapter, we considered the main factors influencing the strength of concrete. Here, some further aspects of strength will be discussed, including fatigue and impact; this will be followed by a brief description of electrical and acoustic properties of concrete.

Curing of concrete

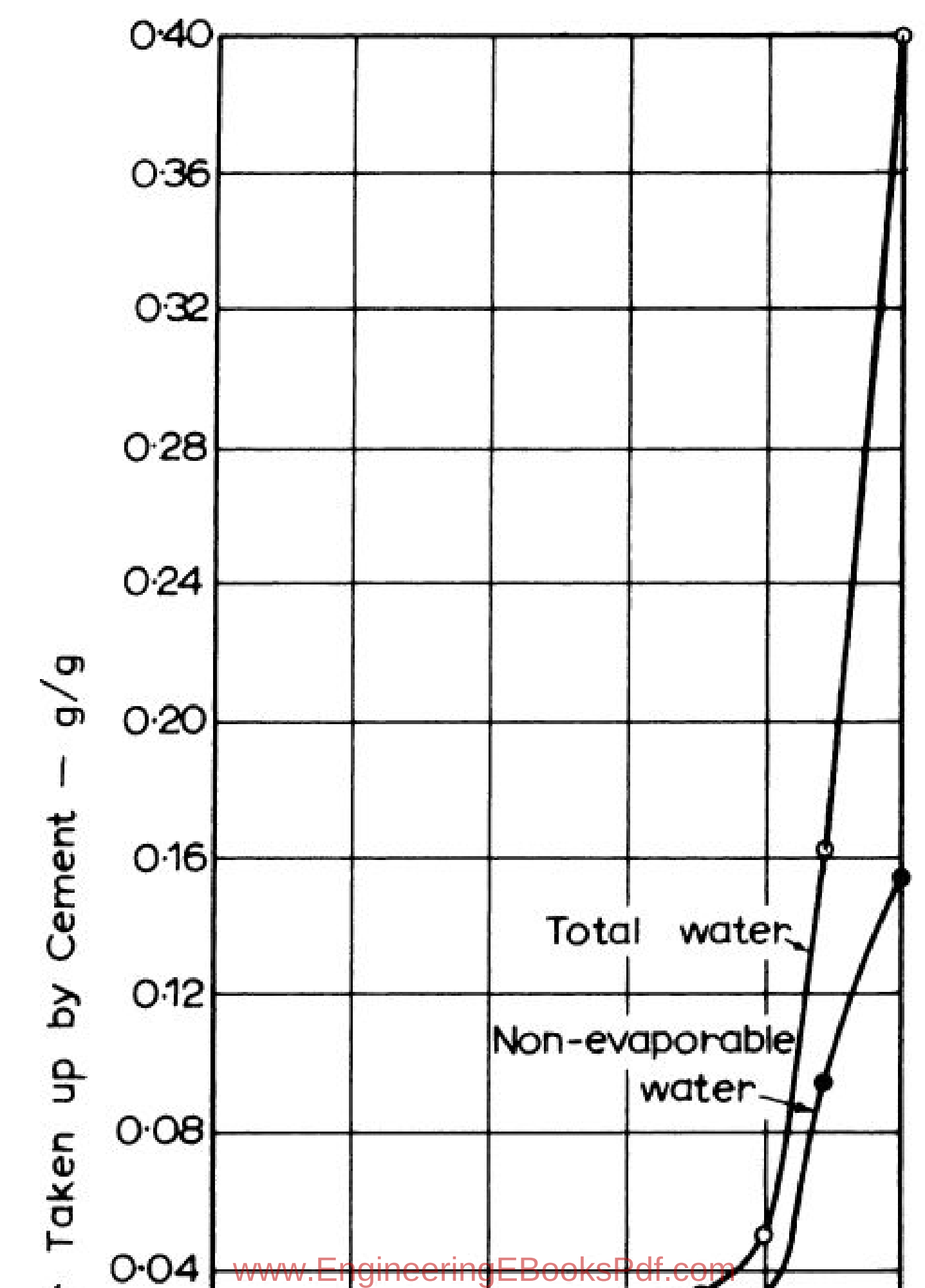
In order to obtain good concrete, the placing of an appropriate mix must be followed by curing in a suitable environment during the early stages of hardening. Curing is the name given to procedures used for promoting the hydration of cement, and consists of a control of temperature and of the moisture movement from and into the concrete. The temperature factor is dealt with in Chapter 8.

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More specifically, the object of curing is to keep concrete saturated, or as nearly saturated as possible, until the originally water-filled space in the fresh cement paste has been filled to the desired extent by the products of hydration of cement. In the case of site concrete, active curing stops nearly always long before the maximum possible hydration has taken place.

Powers[7.36] showed that hydration is greatly reduced when the relative humidity within the capillary pores drops below 80 per cent; this was confirmed by Patel *et al.*[7.3] Hydration at a maximum rate can proceed only under conditions of saturation. Figure 7.1 shows the degree of hydration of cement after six months' storage at different relative humidities, and it is clear that, below a vapour pressure of 0.8 of the saturation pressure, the degree of hydration is low, and is negligible below 0.3 of the saturation pressure.[7.36]

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Fig. 7.1. Water taken up by dry cement exposed for six months to different vapour pressures[7.36]

It follows that, for hydration to continue, the relative humidity inside the concrete has to be maintained at a minimum of 80 per cent. If the relative humidity of the ambient air is at least that high, there will be little movement of water between the concrete and the ambient air, and no active curing is needed to ensure continuing hydration. Strictly speaking, the preceding statement is valid only if no other factors intervene, e.g. there is no wind, there is no difference in temperature between the concrete and the air, and if the concrete is not exposed to solar radiation. In practice, therefore, active curing is unnecessary only in a very humid climate with a steady temperature. It is important to note that in many parts of the world the relative humidity falls below 80 per cent at some time during the day so that the belief in 'natural curing', just because the weather is wet, is unfounded.

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An indication of the influence on evaporation from the concrete surface, of temperature and relative humidity of the surrounding air, and of the wind velocity is given in Figs 7.2, 7.3, and 7.4, based on Lerch's[7.37] results. The difference between the temperatures of concrete and of air also affects the loss of water, as shown in Fig. 7.5. Thus, concrete saturated in day-time would lose water during a cold night, and this would also be the case with concrete cast in cold weather, even in saturated air. The examples quoted are merely typical as the actual loss of water depends on the surface/volume ratio of the specimen.[7.38]

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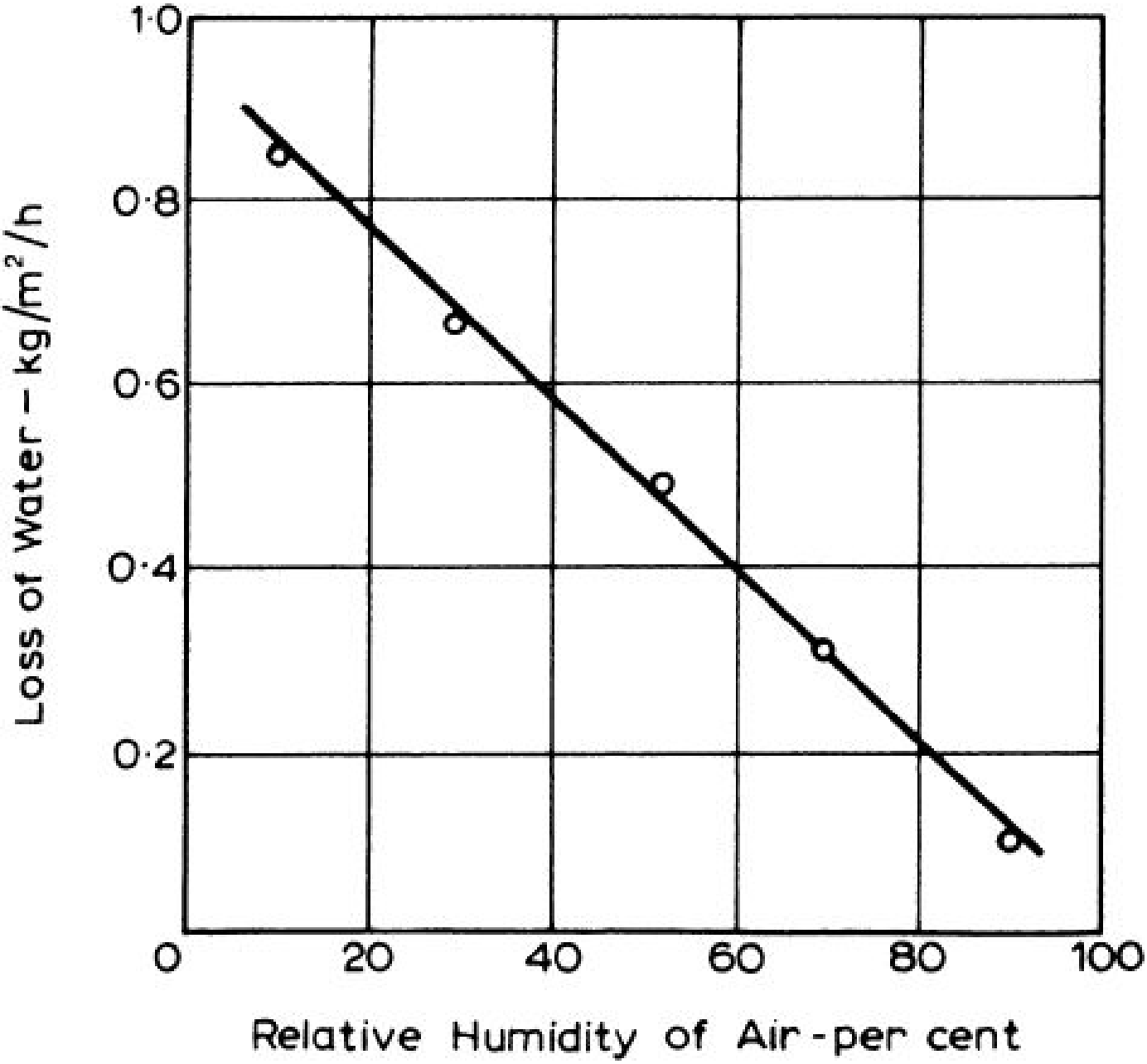


Fig. 7.2. Influence of relative humidity of air on the loss of water from concrete in the early stages after placing (air temperature 21 °C (70 °F); wind velocity 4.5 m/s (10 mph))

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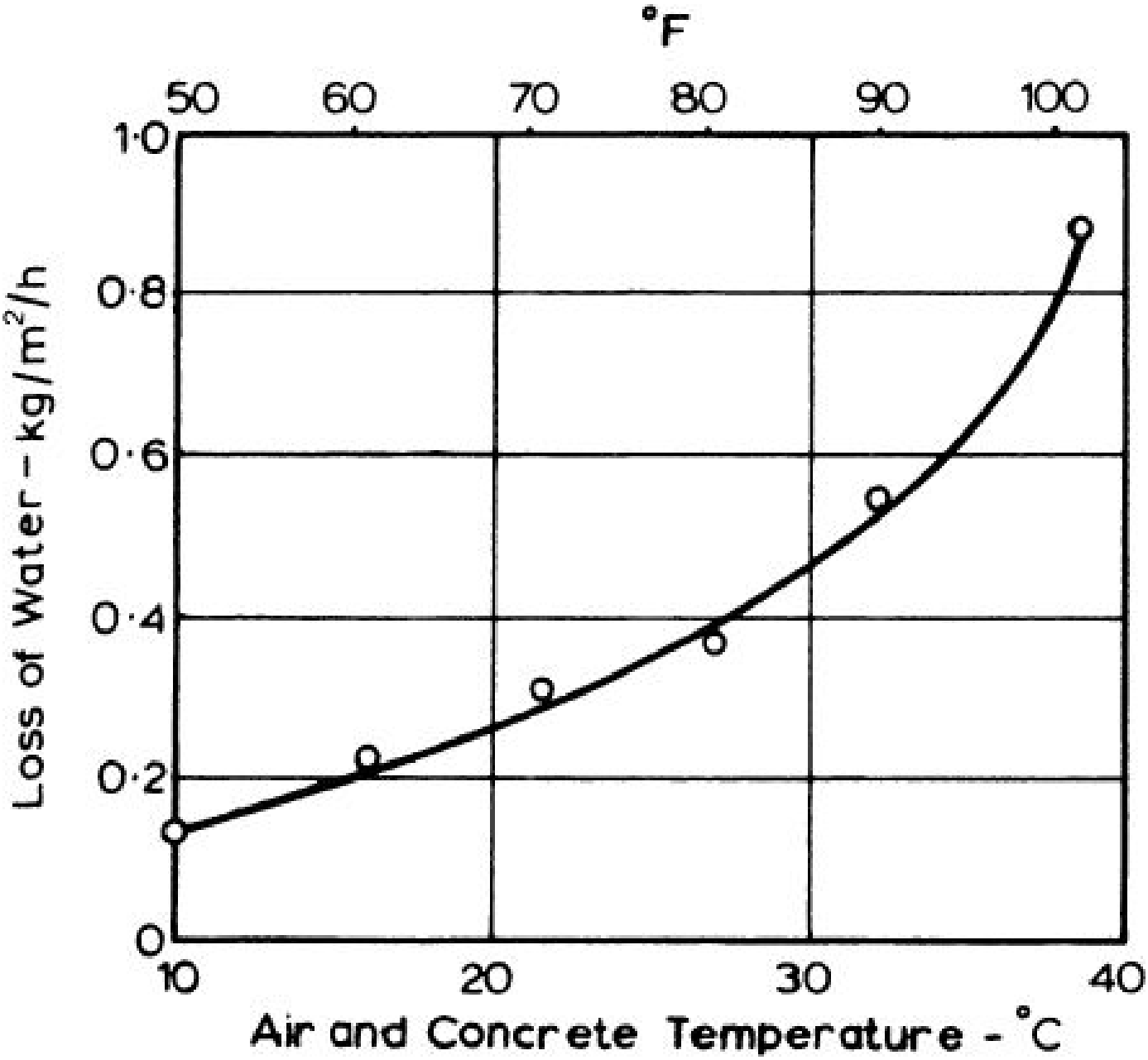


Fig. 7.3. Influence of temperature of air and concrete on the loss of water from concrete in the early stages after placing (relative humidity of air 70 per cent; wind velocity 4.5 m/s (10 mph))

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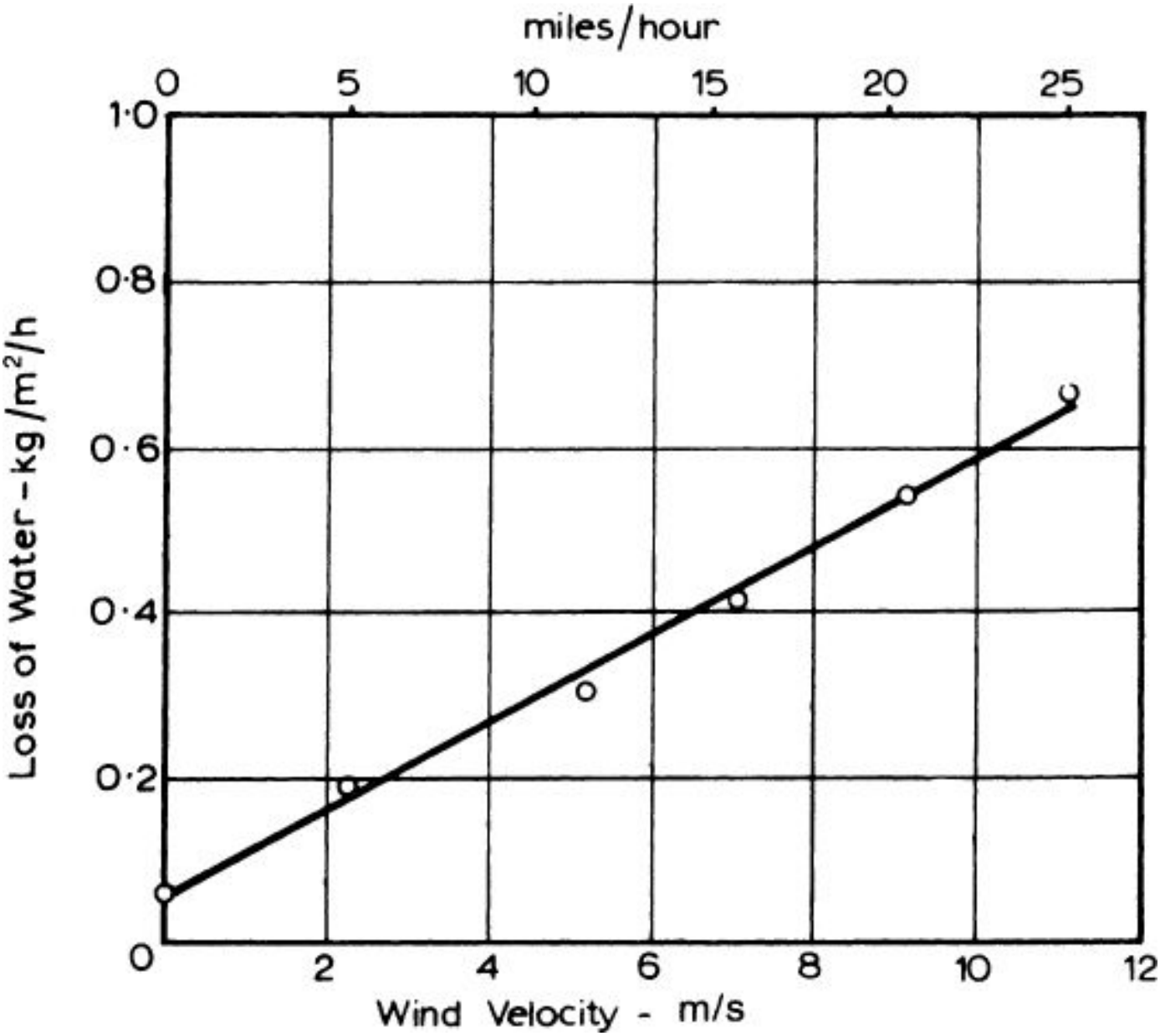


Fig. 7.4. Influence of wind velocity on the loss of water from concrete in the early stages after placing (relative humidity of air 70 per cent, temperature 21 °C (70 °F))

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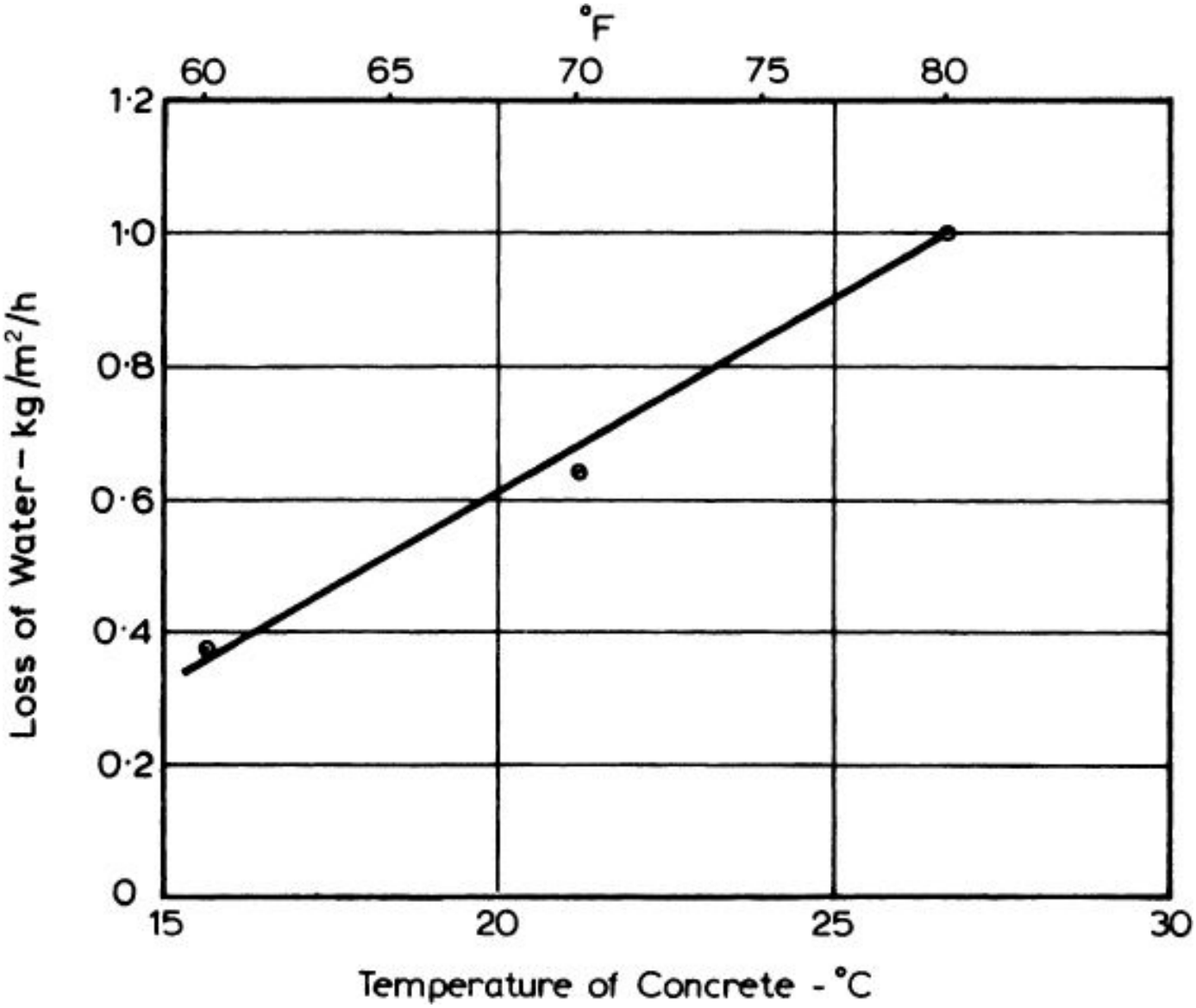


Fig. 7.5. Influence of temperature of concrete (at an air temperature of 4.5 °C (40 °F)) on the loss of water from concrete in the early stages after placing (relative humidity of air 100 per cent, wind velocity 4.5 m/s (10 mph))

Prevention of the loss of water from the concrete is of importance not only because the loss

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adversely affects the development of strength, but also because it leads to plastic shrinkage, increased permeability, and reduced resistance to abrasion.

From the preceding discussion, it could be inferred that, for hydration of cement to continue, it is sufficient to prevent the loss of moisture from the concrete. This is true only if the water/cement ratio of the concrete is sufficiently high for the quantity of the mix water to be adequate for hydration to continue. It was shown in Chapter 1 that hydration of cement can take place only in water-filled capillaries. This is why loss of water by evaporation from the capillaries must be prevented. Furthermore, water lost internally by self-desiccation (due to the chemical reactions of hydration of cement) has to be replaced by water from outside, i.e. ingress of water into the concrete must be made possible.

It may be recalled that hydration of a sealed specimen can proceed only if the amount of water present in the paste is at least twice that of the wa-

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ter already combined. Self-desiccation is thus of importance in mixes with water/cement ratios below about 0.5; for higher water/cement ratios, the rate of hydration of a sealed specimen equals that of a saturated specimen.[7.35] It should not be forgotten, however, that only half the water present in the paste can be used for chemical combination; this is so even if the total amount of water present is less than the water required for chemical combination.[7.36]

In view of the above, we can distinguish between curing needs in situations where, on the one hand, only loss of water from the concrete needs to be prevented and, on the other, situations where water ingress from outside is necessary for hydration to continue. The dividing line is approximately at a water/cement ratio of 0.5. With many modern concretes having a water/cement ratio below 0.5, the promotion of hydration by the ingress of water into concrete is desirable.

It should be added that concrete remote from the surface, that is at depth, is hardly subjected to

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moisture movement, which affects only an outer zone, typically 30 mm deep, but occasionally up to a depth of 50 mm. In reinforced concrete, this depth represents all or most of the depth of cover.

Thus, concrete in the interior of a structural member is generally unaffected by curing, so that curing is of little importance with respect to structural strength except in the case of very thin members. On the other hand, the properties of concrete in the outer zone are greatly influenced by curing; it is the concrete in this zone that is subject to weathering, carbonation, and abrasion, and the permeability of the outer-zone concrete has a paramount influence on the protection of steel reinforcement from corrosion (see Chapter 11).

An indication of the depth of the outer zone which is affected by curing can be obtained from Parrott's tests[7.2] on concrete with a water/cement ratio of 0.59 stored at 20 °C (68 °F) in air with a relative humidity of 60 per cent; he found the following periods for the relative humidity inside

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concrete to fall to 90 per cent: 12 days to a depth of 7.5 mm; 45 days to a depth of 15.5 mm; and 172 days to a depth of 33.5 mm. At lower water/cement ratios, which are common in modern concrete, these periods would be longer.

A reduction in the ambient relative humidity from 100 to 94 per cent was found greatly to increase the water absorption capacity of the concrete, this being an indication of the extent of the continuous large pore system in the concrete.[7.5] Curing at an external relative humidity below about 80 per cent was shown to result in a very large increase in the volume of pores larger than 37 nm, which are relevant to the durability of concrete.[7.3]

It follows from the preceding discussion that the effects of curing should be studied on the outer-zone concrete. However, traditionally, they are expressed in terms of the influence of curing on strength, that is on a comparison of the strength of the specimens stored in water (or in fog) with the strength of those stored under some

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other conditions for different periods; this is taken to demonstrate the effectiveness of curing and its beneficial effect. An example of this is shown in Fig. 7.6, obtained for concrete with a water/cement ratio of 0.50. The loss of strength due to inadequate curing is more pronounced in smaller specimens, but the loss is smaller in light-weight aggregate concrete.[7.55] Tensile and compressive strengths are affected in a similar manner; in both cases, richer mixes are slightly more susceptible.[7.56]

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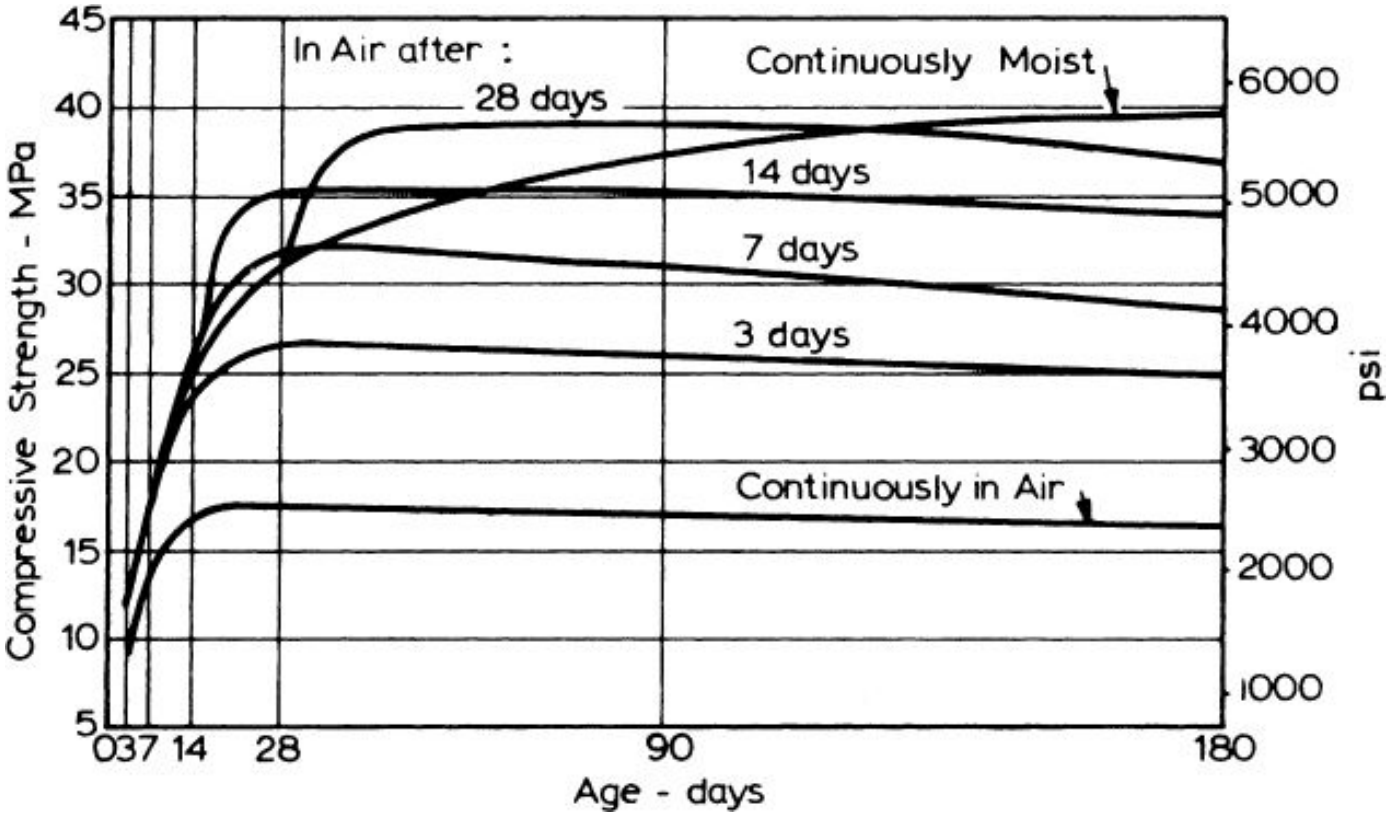


Fig. 7.6. Influence of moist curing on the strength of concrete with a water/cement ratio of 0.50[7.11]

The loss of strength at 28 days seems to be directly related to the loss of water which occurred during the first 3 days; the temperature (20 or 40 °C (68 or 104 °F)) has no effect[7.7] (see Fig. 7.7).

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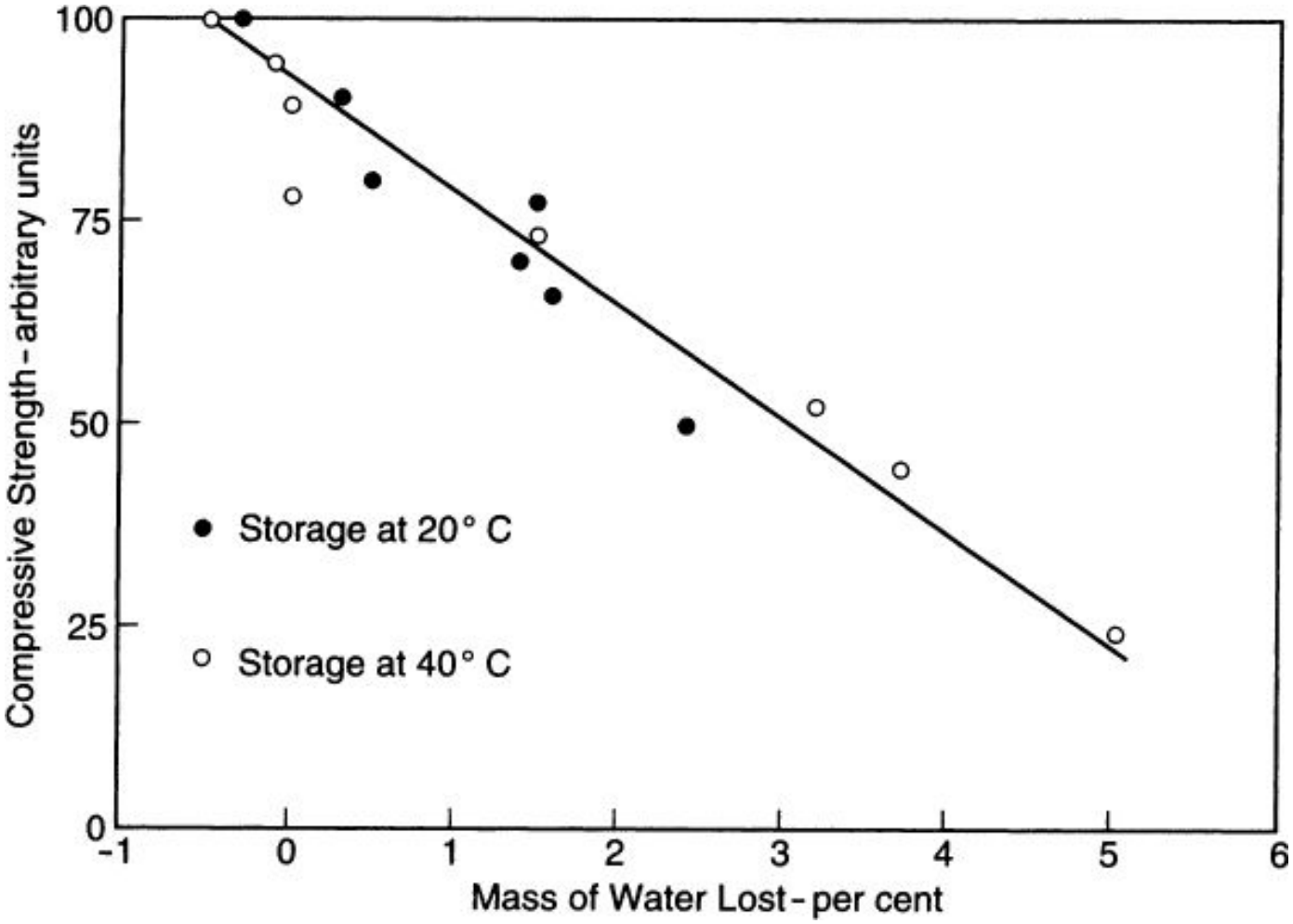


Fig. 7.7. Relation between the compressive strength of concrete at the age of 28 days and the loss of water (by mass of concrete) during the first 3 days (based on ref. 7.7)

The effect of inadequate curing on strength is greater at higher water/cement ratios and is also greater in concretes with a lower rate of development of strength.[7.29] Thus, the strength of concretes made with ordinary Portland (Type I) ce-

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ment is more affected by poor curing. Likewise, concretes containing fly ash or ground granulated blastfurnace slag are more affected than concretes made with Portland cement only.

It must be stressed that, for a satisfactory development of strength, it is not necessary for all the cement to hydrate and, indeed, this is only rarely achieved in practice: as shown in Chapter 6, the quality of concrete depends primarily on the gel/space ratio of the paste. If, however, the water-filled space in fresh concrete is greater than the volume that can be filled by the products of hydration, greater hydration will lead to a higher strength and a lower permeability.

Methods of curing

There are two broad categories of curing whose principles will now be considered, recognizing that the actual procedures used vary widely, depending on the conditions on site and on the size, shape, and position of the concrete member. The

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methods may be broadly described as *wet curing* and *membrane curing*, respectively.

The first method is that of providing water which can be imbibed by the concrete. This requires that the surface of the concrete is continuously in contact with water for a specified length of time, starting as soon as the surface of the concrete is no longer liable to damage. Such conditions can be achieved by continuous spraying or flooding (ponding), or by covering the concrete with wet sand or earth, sawdust or straw. Some care is required as staining may result. Periodically-wetted clean hessian (burlap) or cotton mats (thick and lapped) may be used, or alternatively an absorbent covering with access to water may be placed over the concrete. On inclined or vertical surfaces, soaking hoses can be used. A continuous supply of water is naturally more efficient than an intermittent one, and Fig. 7.8 compares the strength development of concrete cylinders whose top surface was flooded during the first 24 hours with that of cylinders

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covered with wet hessian.[7.77] The difference is apparent only at water/cement ratios below about 0.4 where self-desiccation results in a shortage of water within the concrete. It follows that for low water/cement ratios wet curing is highly desirable. Length of curing is dealt with in BS EN 13670-2009.

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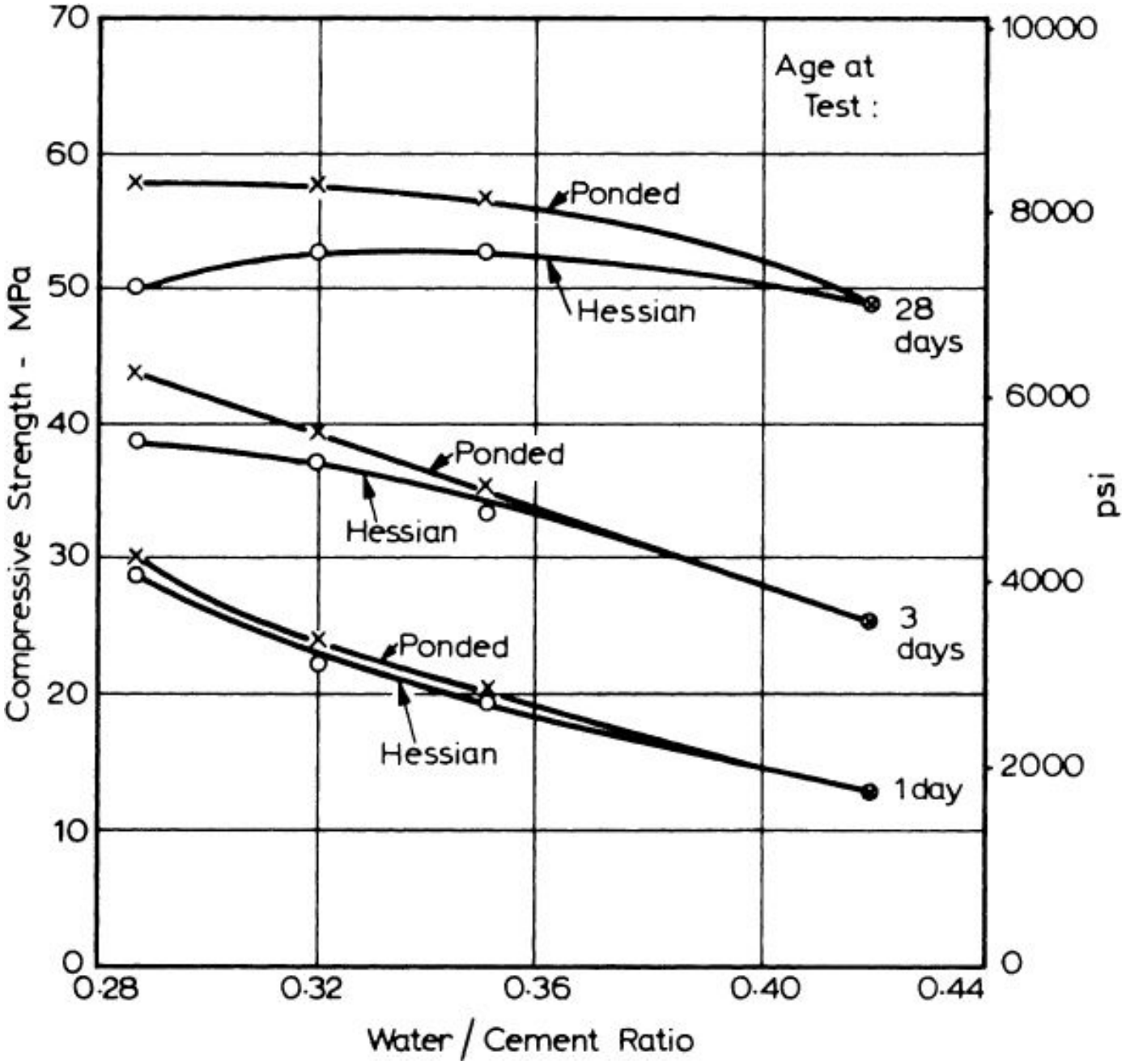


Fig. 7.8. Influence of curing conditions on strength of test cylinders[7.77]

As far as quality of the water used for curing is concerned, ideally it should be the same as mixing water (see p. 184). Sea water may lead to corrosion of reinforcement. Also, iron or organ-

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ic matter may cause staining, particularly if water flows slowly over the concrete and evaporates rapidly. In some cases, discoloration is of no significance.

Whether or not staining will take place cannot be stated on the basis of a chemical analysis and should be checked by a performance test. U.S. Army Corps of Engineers[7.40] recommends a preliminary test in which 300 ml of the water to be used for curing is evaporated from a slight depression, 100 mm (4 in.) in diameter, in the surface of a specimen of neat white cement or plaster of Paris. If the resulting colouring is not considered objectionable, a further test is performed. Here, 150 litres (40 U.S. gallons) of water are allowed to flow lengthwise over a 150 by 150 by 750 mm (6 by 6 by 30 in.) concrete beam with a channel-shaped top surface, placed at 15 to 20° to the horizontal; the rate of flow is 4 litres in 3 to 4 hours. Forced circulation of air and heating by electric lamps encourage evaporation and thus deposition of the residue. The test is again evalu-

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ated by observation only and, if necessary, an actual field test may be performed: a 2 m^2 (or 20 ft^2) slab is cured.

It is essential that curing water be free from substances that attack hardened concrete; these are discussed in Chapters 10 and 11.

The temperature of the water should not be much lower than that of the concrete in order to avoid thermal shock or steep temperature gradients; ACI 308-92 recommends a maximum difference of 11 °C (20 °F).[7.9]

The second method of curing relies on the prevention of loss of water from the surface of the concrete, without the possibility of external water ingressing into it. This could be called a water-barrier method. The techniques used include covering the surface of the concrete with overlapping polyethylene sheeting, laid flat, or with reinforced paper. The sheeting can be black, which is preferable in cold weather, or white, which has the advantage of reflection of solar ra-

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diation in hot weather. Paper with a white surface is also available. Sheeting can cause discoloration or mottling because of non-uniform condensation of water on the underside.

Another technique uses spray-applied curing compounds which form a membrane. The common ones are solutions of synthetic hydrocarbon resins in high-volatility solvents, sometimes including a fugitive bright-colour dye. The dye makes obvious the areas not properly sprayed. A white or alumina pigment can be included to reduce the solar heat gain; this is very effective. Other resin solutions are available: acrylic, vinyl or styrene butadiene, and chlorinated rubber. Wax emulsions can also be used, but they result in a slippery finish which is not easy to remove, whereas the hydrocarbon resins have poor adhesion to concrete and are degraded by ultraviolet light; both these features are desirable.

A specification for liquid membrane-forming curing compounds is given in ASTM C 309-07; and for sheet materials in ASTM C 171-07.

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A question often arises: which curing method or technique to use? For concrete with a water/cement ratio lower than about 0.5, and certainly lower than 0.4, wet curing should be used, but only if it can be applied thoroughly and continuously. If such an assurance is not possible, then membrane curing is preferable, but that, too, has to be well executed.

It is obvious that the membrane must be continuous and undamaged. The timing of spraying is also critical. The curing spray should be applied after bleeding has stopped bringing water to the surface of the concrete but before the surface has dried out: the optimum time is the instant when the free water on the surface of the concrete has disappeared so that the water sheen is no longer visible. However, if bleeding has not stopped, the curing membrane should not be applied even if the surface of the concrete appears dry in consequence of a high rate of evaporation. For this purpose, an evaporation rate of 1 kg/m^2 per hour can be taken as 'high'. The rate can be

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calculated using Figs 7.2 to 7.5, based on Lerch's results;[7.37] alternatively, a chart in ACI 308R-86, based on the same source[7.37] as these figures, can be used.

When a high rate of evaporation removes water faster than it is brought up by bleeding, Mather[7.6] recommends wetting the concrete and delaying the application of the curing compound until bleeding has ceased.

Some concretes, for instance those containing silica fume, exhibit no bleeding, in which case the curing membrane should be applied without delay. If the curing compound is applied to a surface which has dried out, the spray will penetrate into the concrete and prevent further hydration within the outer zone. Moreover, no effective continuous membrane would be formed.[7.6]

With slip-forming, where the form is effectively removed after several hours, the immediate application of curing is important if there are durability requirements, or for strength reasons in the

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case of thin members. On the other hand, ordinary formwork left in place is a means of preventing the loss of moisture from vertical surfaces. After it has been loosened, application of water is possible.

Tests on curing compounds

The efficiency of curing compounds in terms of the extent to which they permit the loss of water from the surface of a standard mortar can be determined by tests. British Standard BS 7542 : 1992 uses 1:3 mortar with a water/cement ratio of 0.44, and exposure to 38 °C (100 °F) and a relative humidity of 35 per cent for 72 hours; the percentage reduction in the loss of water compared with a specimen without a membrane is taken to represent the curing efficiency. The test method of ASTM C 156-09a is similar but the performance of the compound is expressed as a loss of water per unit area. The reproducibility of this test is said to be poor.[7.4]

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Neither the British nor the American test measures the quality of the cured concrete in the surface zone, which is what is of interest in practice but is not easy to determine. Various proposed other tests are too cumbersome for practical use or else interfere with the concrete being tested.

In tests, the surface of the mortar is level and finished with a float. In practice, the surface of the concrete may be coarsely brushed or tined (as in the case of highway slabs) and this affects the amount of curing compound necessary. Also, because a uniform and continuous membrane is more difficult to achieve under such circumstances, a good water-retaining performance in the test may fail to be matched in practice.

Length of curing

The period of curing required in practice cannot be prescribed in a simple manner: the relevant factors include the severity of the drying conditions and the expected durability requirements.

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As an example, the minimum periods of curing for external exposure, including freezing and thawing but not the use of de-icing agents and for exposure to aggressive chemicals are given in Table 7.1, derived from BS EN 206-1 : 2007. If concrete is to be subjected to abrasion, doubling of the curing periods is desirable. Minimum periods of curing are given in BS 8110-1 : 1997.

Table 7.1. Minimum Curing Times (in days) Recommended in BS EN 206-1 : 2007

Rate of gain of strength of concrete	*Rapid**			*Medium*			*Slow*		
Temperature of concrete, °C	5	10	15	5	10	15	5	10	15
Ambient conditions during curing									
No sun, rh ⩾ 80	2	2	1	3	3	2	3	3	2
Medium sun or medium wind or rh ⩾ 50	4	3	2	6	4	3	8	5	4
Strong sun or high wind or rh < 50	4	3	2	8	6	5	10	8	5

rh = relative humidity in per cent.
*Low water/cement ratio and rapid-hardening cement.

The requirements for striking formwork are governed by the strength of concrete. This can be estimated from the maturity of the concrete (see p. 306) or by tests on companion compress-

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ive strength specimens (see p. 584) or else by non-destructive tests. Guidance is given by Harrison.[7.8]

It was stated earlier that curing should start at the earliest possible instant and should be continuous. Occasionally, intermittent curing is applied, and it is useful to appreciate its effect. In the case of concrete with a low water/cement ratio, continuous curing at an early age is vital as partial hydration may make the capillaries discontinuous: on renewal of curing, water would not be able to enter the interior of the concrete and no further hydration would result. However, mixes with a high water/cement ratio always retain a large volume of capillaries so that curing can be effectively resumed at any time, but the earlier the better.

The preceding discussion has laid much stress on the importance of proper curing. Curing is always specified but rarely adequately executed. And yet, inadequate curing is responsible for a great many durability problems with concrete, es-

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pecially reinforced concrete. For this reason, the importance of curing cannot be overemphasized.

Autogenous healing

Fine cracks in fractured concrete, if allowed to close without tangential displacement, will heal completely under moist conditions. This is known as autogenous healing, and is due primarily to the hydration of the hitherto unhydrated cement which becomes exposed to water upon the opening of the cracks. Healing is also aided by the formation of insoluble calcium carbonate from the calcium hydroxide in hydrated cement if carbonation takes place. Some mechanical blocking of the cracks may also occur if very fine material is suspended in the water.

The maximum width of cracks which can undergo autogenous healing is estimated to be between 0.1 and 0.2 mm, and the necessary moist conditions include frequent periodic wetting as well as immersion,[7.28] but not fast-flowing water or high water pressure, which is not conducive

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to reducing the movement of water through the crack. The application of pressure across the crack assists in healing.

In young concrete, cracks 0.1 mm wide can heal after several days but 0.2 mm wide cracks require several weeks.[7.28] Generally, the younger the concrete, i.e. the more unhydrated cement it contains, the higher the re-gain of strength, but healing without a loss of strength has been observed at ages up to three years. It has been reported[7.31] that, even when healed, cracks represent a weakness zone in which renewed cracking may occur under future adverse conditions.

Means of encouraging autogenous healing by incorporation of suitable bacteria in the mix are being studied in laboratory tests (see p. 266).

Variability of strength of cement

Up to now we have not considered the strength of cement as a variable in the strength of concrete. By this we do not mean the differences in the

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strength-producing properties of cements of different types, but the variation between cements of nominally the same type: they vary fairly widely, and it is this variation that is considered in this section.

The strength requirements for cement were discussed in Chapter 2. Traditionally, only a minimum strength at certain ages has been prescribed, so that there should be no objection to a cement with a much higher strength. The cement manufacturers advance this argument forcefully and are unsympathetic to cement users who want to take economic advantage of the *actual* higher strength of cement and who complain when, at some instant, the strength margin above the specified minimum is substantially reduced.

One consequence of the absence of an upper limit on strength is that there is an overlap in the strengths of Type I and Type III cements: occasionally Type I cements have been found to have strengths as high as twice the specified minimum.[7.41]

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The absence of a maximum specified strength persists in most specifications. However, European Standard BS EN 197-1 : 2000, BS 12 : 1996. German standards (which pioneered this approach) prescribe a maximum strength for most of the cement classes at a value 20 MPa greater than the minimum. This range of strengths for a given class of cement is high, although probably justified economically for a mass product with a wide range of uses.

The variation in strength of cement is due largely to the lack of uniformity in the raw materials used in its manufacture, not only between different sources of supply, but also within a pit or quarry. Furthermore, differences in details of the processes of manufacture and, above all, the variation in the ash content of coal used to fire the kiln, contribute to the variation in the properties of commercial cements. This is not to deny that the modern manufacture of cement is a highly sophisticated process.

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Pioneer work in the variation in the strength of cement by Walker and Bloem[7.42] has contributed to the development of a test method for the evaluation of cement strength uniformity from a single source, ASTM C 917-05. This method uses the mortar cube strength test ASTM C 109-08 and relies on a moving average of five grab (spot) samples. An example of the variability in a single plant over a period of three years is given in Fig. 7.9. It can be seen that there was a reduction in variability between 1982 and 1984; the standard deviation[*] of the 7-day strength at the end of the period was 1.4 MPa (208 psi). Tests[7.14] at 87 United States cement plants, conducted in 1991, showed that 81 per cent of them had a standard deviation of 7-day strength lower than 2.10 MPa (300 psi); at 28 days, only 43 per cent of the plants had a standard deviation lower than 2.10 MPa. The increase in the standard deviation with age is typical of American cements[7.12] but not necessarily of cements made elsewhere.

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*The statistical terms are defined on p. 642.

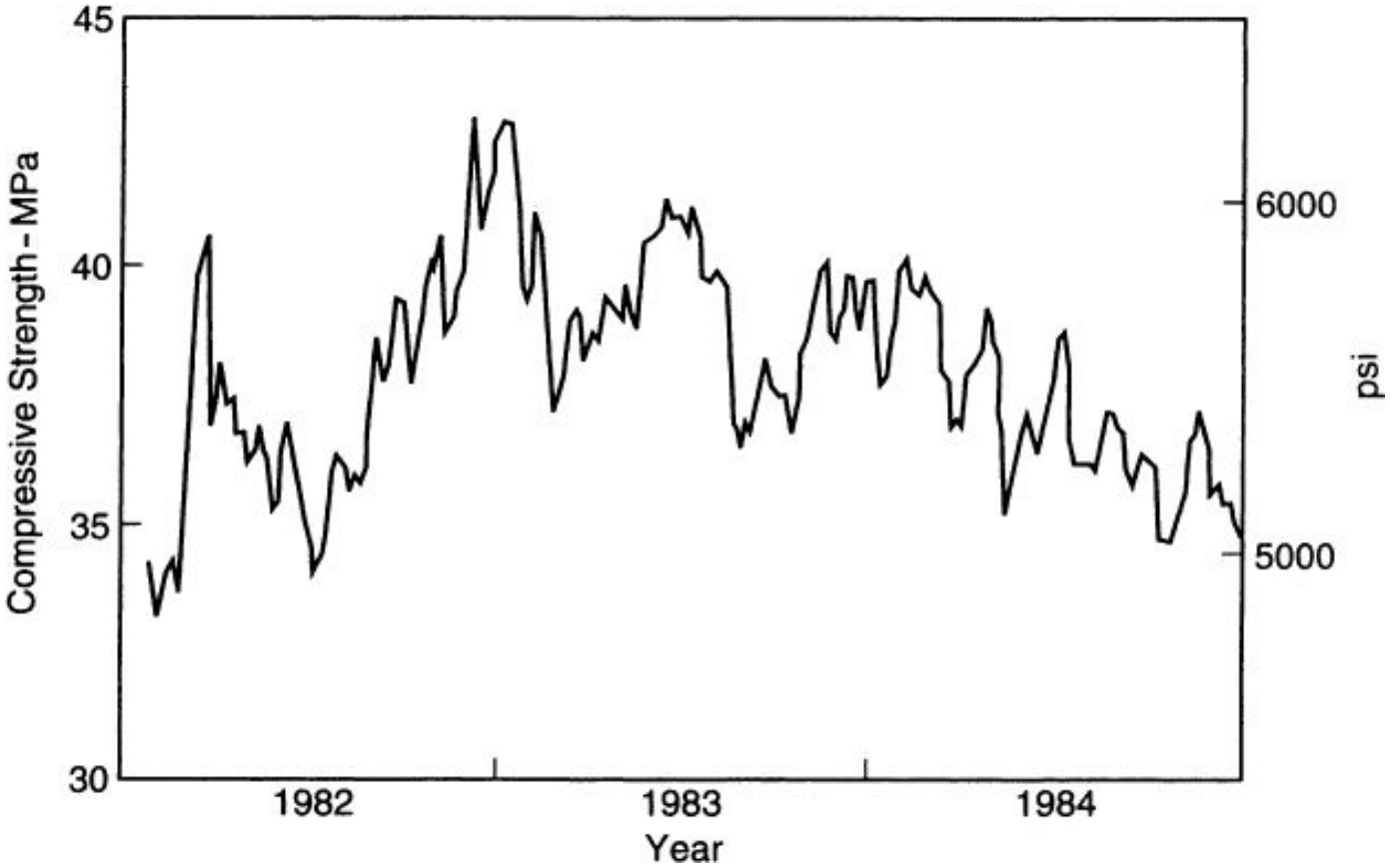


Fig. 7.9. Plot of moving averages of strength of five tests on 28-day mortar cubes (made to ASTM C 109) using cement from a single plant in the years 1982 to 1984 (based on ref. 7.13)

The large range of strengths of cement from a single plant in Fig. 7.9 should be noted: a range of 28-day strength of 7 MPa (1000 psi) in a period of a few months is not uncommon. Clearly, us-

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ing cement with a smaller and known variability would result in an economic advantage over relying on the minimum strength. There remains, however, the problem of the relatively poor precision of the ASTM C 109-08 mortar test used to measure the strength of cement. Nevertheless, large purchasers of cement can influence its variability by requiring testing to ASTM C 917-05 and agreeing appropriate limits.

It is important to be clear about the use of grab samples and of a moving average. Values of single grab samples might not be representative and would be unduly affected by testing errors. On the other hand, composite samples, which are obtained by putting together sub-samples from production during 24 hours, give an unduly smoothed-out result.

What is the relevance of the strength of cement to the strength of concrete made with that cement? It is rational to expect a direct influence[7.78] (see Fig. 7.10), even though many other factors also affect the strength of concrete. This

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relation between cement and concrete strengths may seem obvious, but in the past it was claimed[7.32] that there is no correlation between the strength of concrete and the strength of cement, as established by the cement manufacturers' testing, used in making that concrete.

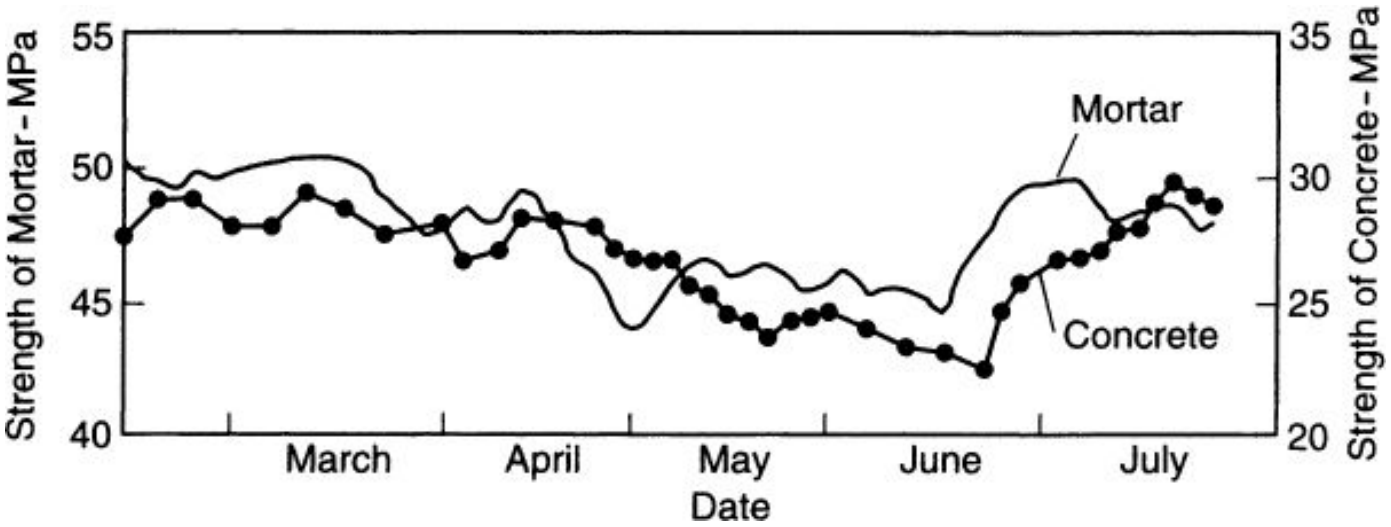


Fig. 7.10. Moving average strength of mortar cubes (made to ASTM C 109) and average strength of concrete cylinders at the age of 28 days in the period March to July 1980 (based on ref. [7.78]). *Note:* The ordinates for mortar and concrete are not the same; the two plots have been moved close to one another

This kind of argument misses the crucial point that a composite sample of cement obtained over

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a 24-hour period represents *average* properties of the thousands of tonnes of cement produced during that period. Inevitably, there are variations within that bulk of cement, only a very small part of which is used in making a given batch of concrete. At the same time, making concrete also introduces variability.

As an aside, it may be useful to make a comment about the use of cement manufacturers' test certificates in research. Often, the properties of cement such as chemical composition reported in the test certificate are used by the researcher as a test parameter. If the test certificate refers to the average of a 24-hour production, the properties as listed cannot be considered as necessarily applying to the actual cement used by the researcher. If they are so considered, spurious correlations with the property investigated may be found; alternatively, the experimental work may fail to show a real correlation through no fault of the researcher.[7.33]

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It must be realized that the use of admixtures would clearly upset the relationship between cement and concrete strengths because the precise influence of admixtures depends on the properties of the cement used, whereas the cement strength test uses a mortar without any admixtures.

With the introduction of performance-based specifications for cement, it is important to know more about its true strength characteristics, which must affect the strength of the concrete made with a given cement. The situation becomes more complicated when cement comes from different sources.

The variation in the strength of cement from different plants is obviously much larger than when a single plant is the supplier. Table 7.2 gives the data for 87 plants in the United States, tested in 1991;[7.14] the strengths are those of mortar cubes according to ASTM C 109-08. It must not be forgotten, however, that variation in cement accounts, at the most, for one-half of the

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variation in the strength of site test specimens; U.S. Bureau of Reclamation data[7.57] suggest a typical value of one-third. The variation in the strength of site cubes is discussed on p. 639. More recent studies on the variability of strength of cement can be found in ref 7.102.

Table 7.2. Strength of Cement Produced in 87 American Plants in 1991[7.14] (shown as a percentage of plants with average strength lower than indicated) (Copyright ASTM–reproduced with permission)

7-day strength psi	*Percentage*	*28-day strength psi*	*Percentage*
5800	100	7500	100
5600	99	7250	99
5400	98	7000	98
5200	97	6750	93
5000	93	6500	89
4800	78	6250	69
4600	53	6000	48
4400	23	5750	24
4200	7	5500	7
4000	0	5250	1
		5000	1
		4750	0

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Finally, it should be stressed that the variation in cement affects to the greatest extent the early strength of concrete, i.e. the strength most often determined by test but not necessarily the strength of greatest practical significance. Furthermore, strength is not the only important characteristic of concrete: from considerations of durability and permeability, a cement content in excess of that needed for strength may well be required, in which case the variability of cement becomes unimportant.

Changes in the properties of cement

In the preceding section, we considered the variation in the strength of cement produced in a single plant over a period of several months or a year. Some reference was also made to the differences in the strength of cements made in different plants during a single year. There is, in addition, a systematic change in the strength of cement with time. Indeed, there has been such a continuing change over many years in conse-

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quence of the improvement in the manufacture of cement[7.10,7.39] (see Fig. 7.11).

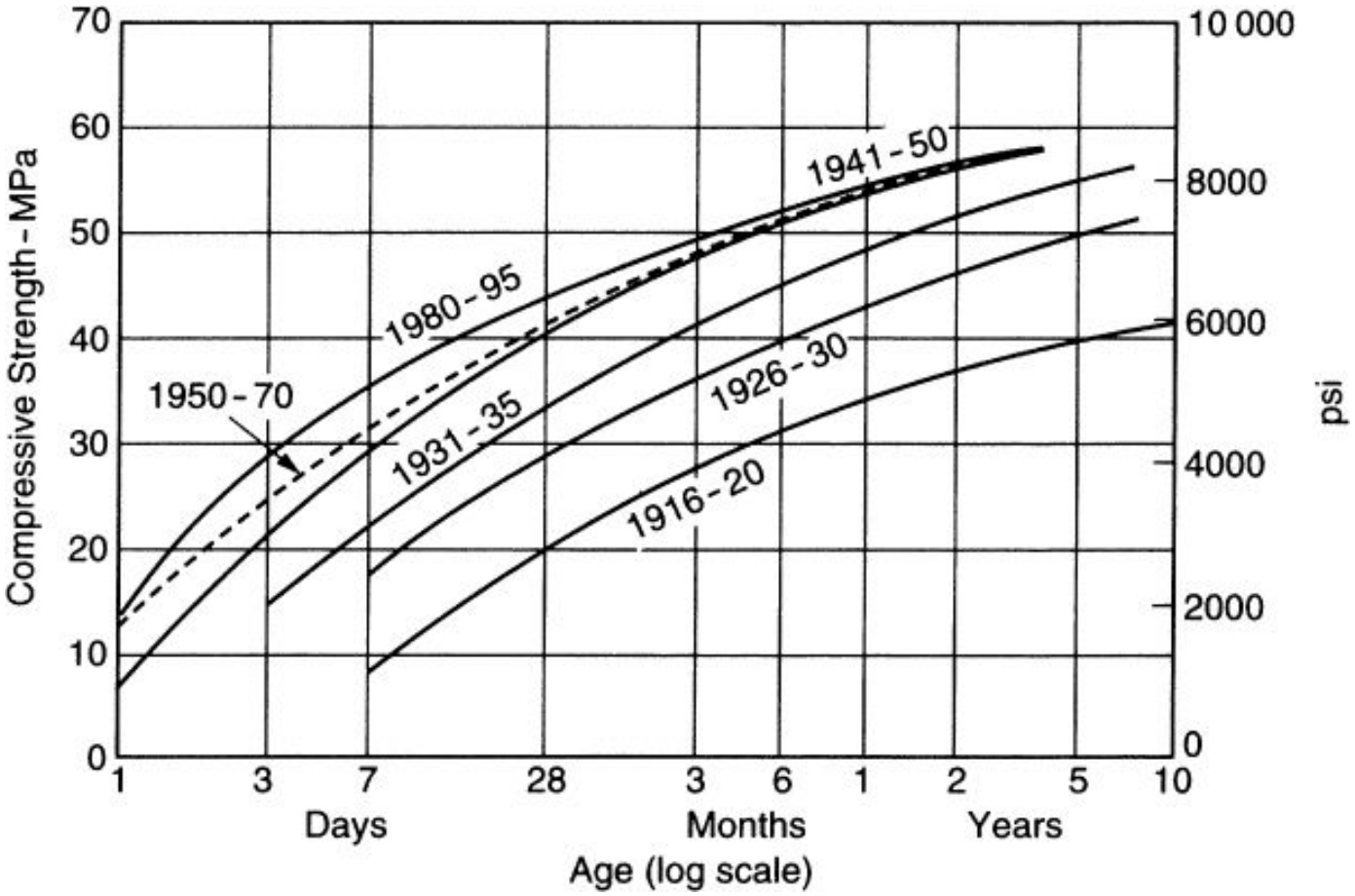


Fig. 7.11. Changes in gain of strength of cements with age between 1916 and the 1990s (measured on standard cylinders of concrete with a water/cement ratio of 0.53 (based on refs 7.10 and 7.39 and private data)

First of all, we can give an example[7.1] of the difference in the average properties of cements produced in 1923 and in 1937. Two series of

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tests spanning a 50-year life of concrete stored outdoors in Wisconsin, United States, gave data on the strength development. The 1923 concretes were made with cements with a high C_2S content and a low fineness: their compressive strength increased in proportion to the logarithm of age up to 25 or 50 years. The concretes made in 1937 used cements with a lower C_2S content and a high fineness: their compressive strength increased in proportion to the logarithm of age for about 10 years but, thereafter, decreased or remained constant.[7.1] This change in behaviour is mainly of historical interest but it helps to understand the differences in the behaviour of concretes of various ages.

A more recent change, namely that around the 1960s, merits particular attention because it has had far-reaching consequences for concrete production practice.

The changes in British cements are well documented[7.16,7.21] but they occurred in other countries as well. The change of greatest practical in-

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terest was the increase in the 28-day strength, and also in the 7-day strength, of mortar made with a fixed water/cement ratio. The main reason for this was a large increase in the average content of C_3S: from about 47 per cent in 1960 to about 54 per cent in the 1970s.[7.16] There was a corresponding decrease in the content of C_2S so that the total content of calcium silicates remained constant at 70 to 71 per cent. This change was made possible by improvements in the methods of manufacture of cement, but it was also driven by the benefits of using a 'stronger' cement as perceived by the users, namely: reduction in cement content for a given specified strength, earlier removal of formwork, and faster construction. Such benefits were, unfortunately, associated with disadvantages.

There was no significant change in the fineness of cement, which is not surprising because of the high cost of grinding clinker.[7.16,7.20]

The high rate of increase in strength up to 7 days and the rate of increase between 7 and 28

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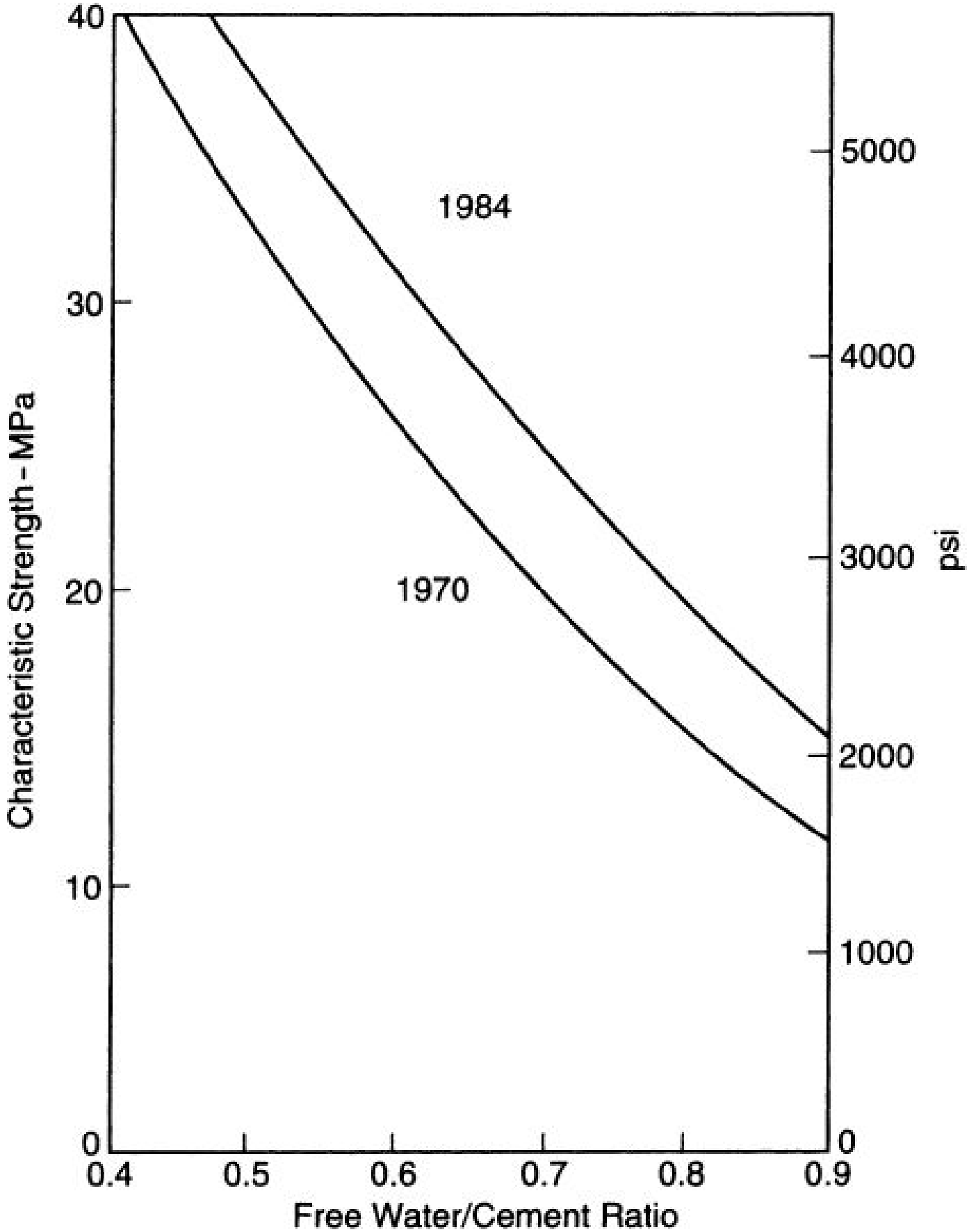
days have changed in consequence of the higher alkali content in modern cements as well as because of the change in the ratio of C_3S to C_2S. The ratio of the strength at 28 days to that at 7 days has decreased substantially. For concrete with a water/cement ratio of 0.6, a decrease in the 28- to 7-day strength ratio from about 1.6 prior to 1950 to about 1.3 in the 1980s was reported;[7.20] these figures are only examples of the behaviour of some British cements and are not necessarily generally valid. At lower water/cement ratios, the ratio of the 28-day strength to the 7-day strength is lower. Likewise, the increase in strength beyond the age of 28 days is much reduced when modern cements are used so that it should no longer be relied upon in the design of structures which will be subjected to full load only at an advanced age.

An example of the change in the 28-day strength of cement between 1970 and 1984 is shown in Fig. 7.12.[7.21] It can be seen that concrete with a characteristic cube strength (see p.

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732) of 32.5 MPa (4700 psi), which in 1970 required a water/cement ratio of 0.50, could be achieved in 1984, using a water/cement ratio of 0.57. Assuming that, for the workability to remain constant, the same water content of, say, 175 kg per cubic metre of concrete was maintained, it was possible to reduce the cement content from 350 to 307 kg/m^3.

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Fig. 7.12. Relation between the characteristic strength of concrete and water/cement ratio for concretes made in 1970 and in 1984; 20 mm ($\frac{3}{4}$ in.) maximum aggregate size, slump of 50 mm (2 in.) (based on ref. 7.21)

More generally, over the longer period between the 1950s and the 1980s, for concrete of a given strength and workability, it was possible to reduce the cement content by 60 to 100 kg per cubic metre of concrete and concomitantly to increase the water/cement ratio by between 0.09 and 0.13.[7.20]

While a higher 28-day strength of concrete at a given water/cement ratio could be economically exploited, there were consequential disadvantages. Concrete having the same 28-day strength as before (when the 'old' cements were used) can be made using a higher water/cement ratio and a lower cement content, as shown in the preceding paragraph. Both these concomitant changes result in concrete with a higher permeab-

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ility and therefore more liable to carbonation and penetration by aggressive agents, and generally of lower durability.

Moreover, the absence of a significant increase in strength beyond the age of 28 days[7.20,7.21] removed a long-term improvement in concrete which had reassured users in the past (even if such improvement was not taken into account in design).

The rapid early gain in strength also means that strengths adequate for removal of formwork are achieved earlier than was the case with the 'old' cements so that effective curing ceases at an early age.[7.17] The adverse consequences of this were discussed earlier in this chapter.

These consequences were not foreseen, partly because many concrete users were preoccupied with exploiting the high early strength properties of cement, and partly because the concrete specifications were couched predominantly in terms

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of a 28-day strength, which remained the same as it had been when the 'old' cements were used.

Although the above data refer to British cements, the changes have occurred worldwide, albeit not at the same time, the driving force being the modernization of cement plants. French figures may be of interest: between the mid-1960s and 1989, the average C_3S content of Portland cement increased from 42 to 58.4 per cent, with C_2S decreasing simultaneously from 28 to 13 per cent.[7.15]

The increase in the average 28-day strength appears to continue. In the United States, between 1977 and 1991, the strength of mortar made according to ASTM C 109-93 increased from 37.8 MPa (5470 psi) to 41.5 MPa (6020 psi).[7.14]

Fatigue strength of concrete

In Chapter 6, we considered only the strength of concrete under static loading. In many structures,

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however, repeated loading is applied. Typical of these are offshore structures subjected to wave and wind loading, bridges, road and airfield pavements, and railway sleepers (ties); the number of cycles of loading applied during the life of the structure may be as high as 10 million, and occasionally even 50 million.

When a material fails under a number of repeated loads, each smaller than the static compressive strength, failure in fatigue is said to take place. Both concrete and steel possess the characteristics of fatigue failure but, in this book, the behaviour of concrete alone is dealt with.

Let us consider a concrete specimen subjected to alternations of compressive stress between values σ_1 (≥ 0) and σ_h ($>\sigma_1$). The stress–strain curve varies with the number of load repetitions, changing from concave towards the strain axis (with a hysteresis loop on unloading) to a straight line, which shifts at a decreasing rate (i.e. there is some irrecoverable deformation) and eventually becomes concave towards the stress axis. The de-

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gree of this latter concavity is an indication of how near the concrete is to failure. Failure will, however, take place only above a certain limiting value of σ_h, known as *fatigue limit* or *endurance limit*. If σ_h is below the fatigue limit, the stress–strain curve will indefinitely remain straight, and failure in fatigue will not take place. The changes in the stress–strain curve with the number of applied cycles are illustrated in Fig. 7.13 for compressive loading and in Fig. 7.14 for direct tension.[7.94]

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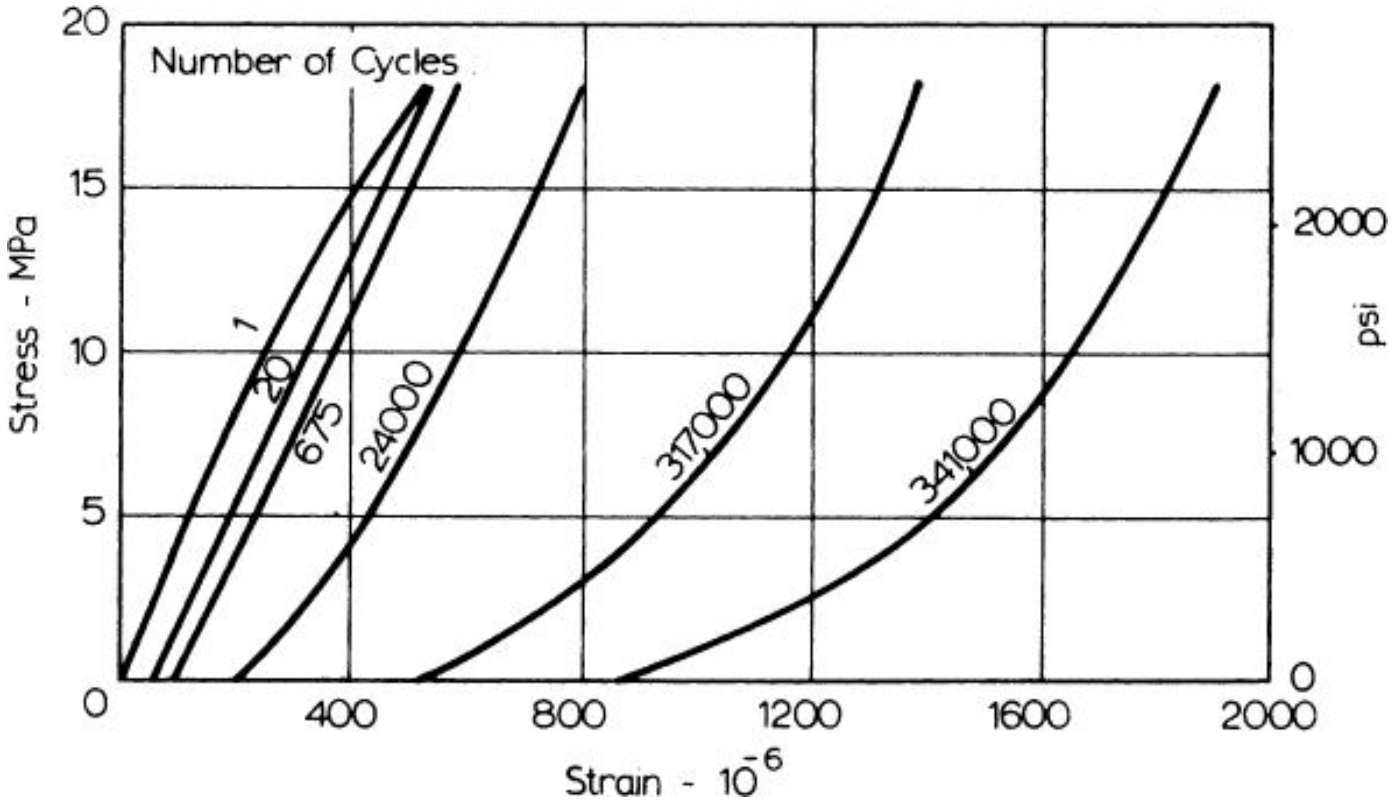


Fig. 7.13. Stress–strain relation of concrete under cyclic compressive loading

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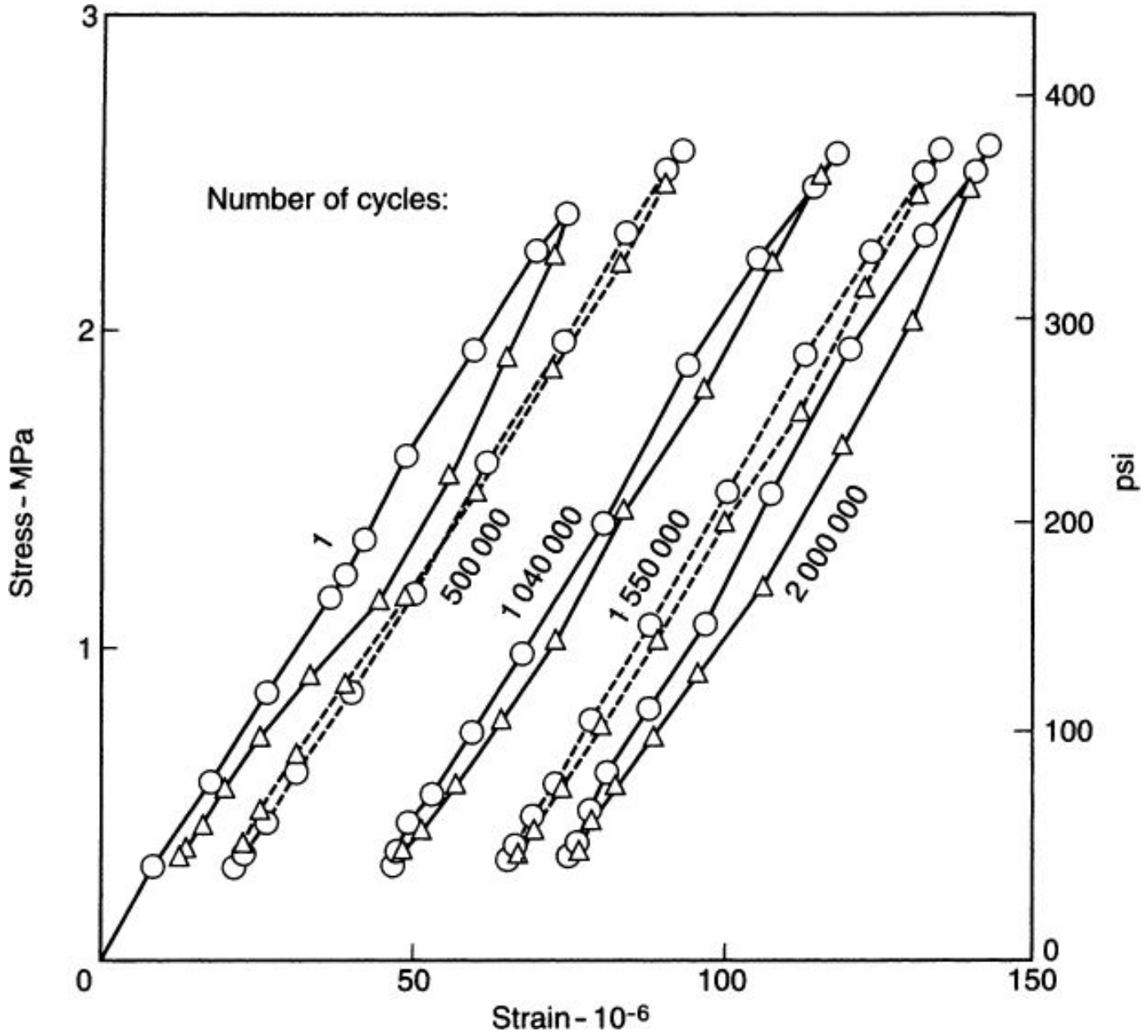


Fig. 7.14. Stress–strain relation of concrete under cyclic loading in direct tension (based on ref. 7.94)

The change in strain with the number of cycles of loading can be described as consisting of three phases.[7.83] In Phase 1, that is, the initiation phase,

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strain increases rapidly, but at a progressively decreasing rate, with the number of cycles of loading. In Phase 2, which represents the stable state, strain increases approximately linearly with the number of cycles. In Phase 3, which represents instability, strain increases at a progressively increasing rate until failure in fatigue takes place. An example of this behaviour is shown in Fig. 7.15.

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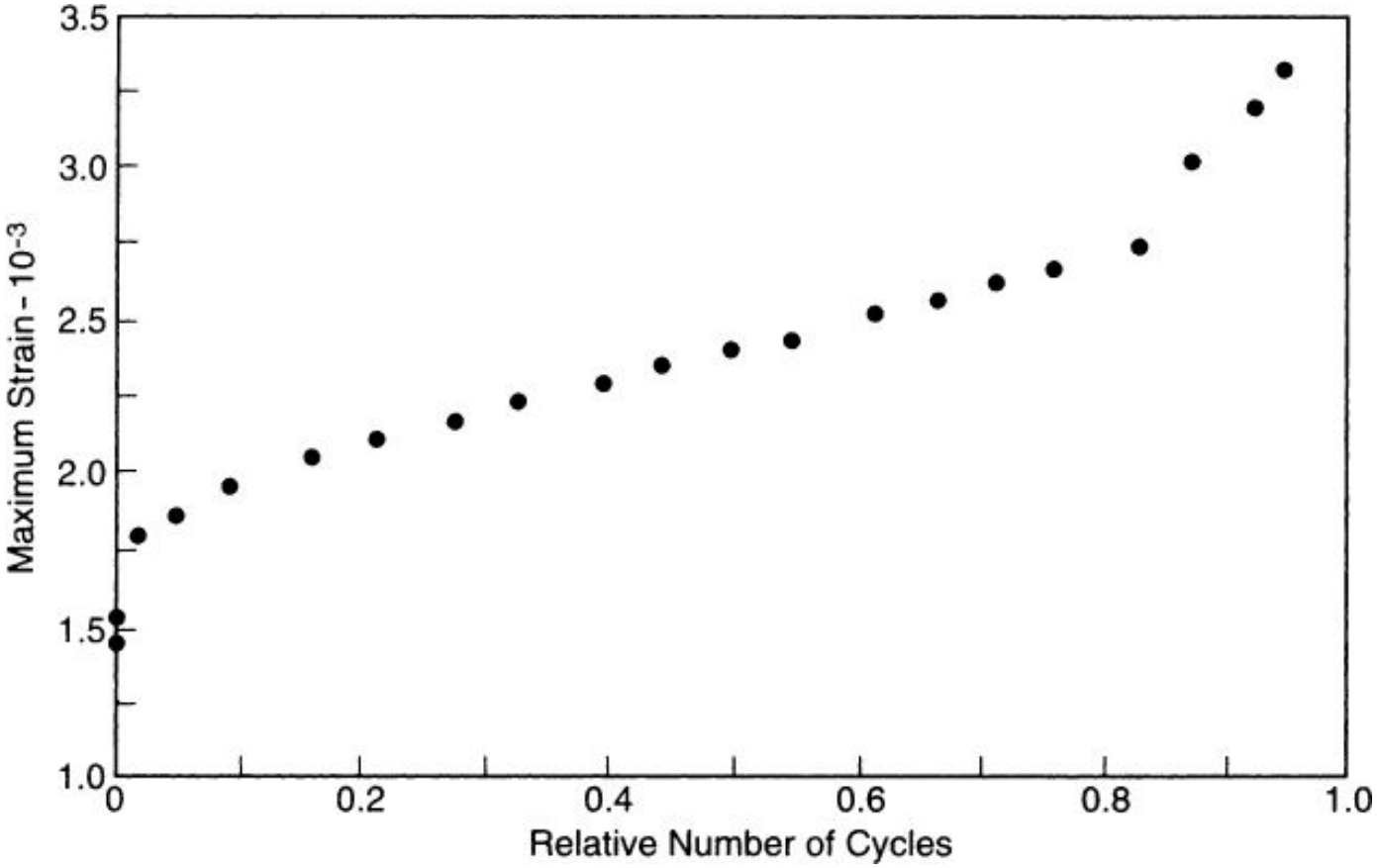


Fig. 7.15. Relation between strain and relative number of cycles of loading in compression, expressed as proportion of number of cycles to failure (maximum stress equal to 0.75 of the static strength; minimum stress equal to 0.05 of the static strength) (based on ref. 7.83)

If the stress–strain curve for unloading were also drawn in Fig. 7.13, a hysteresis loop in each cycle could be seen. The area of this loop decreases with each successive cycle and then eventually increases prior to fatigue failure.[7.43] There

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does not seem to be such an increase in specimens which do not fail in fatigue. If we plot the area of each successive hysteresis loop as a percentage of the area of the first loop, the variation with the number of cycles is as shown in Fig. 7.16.

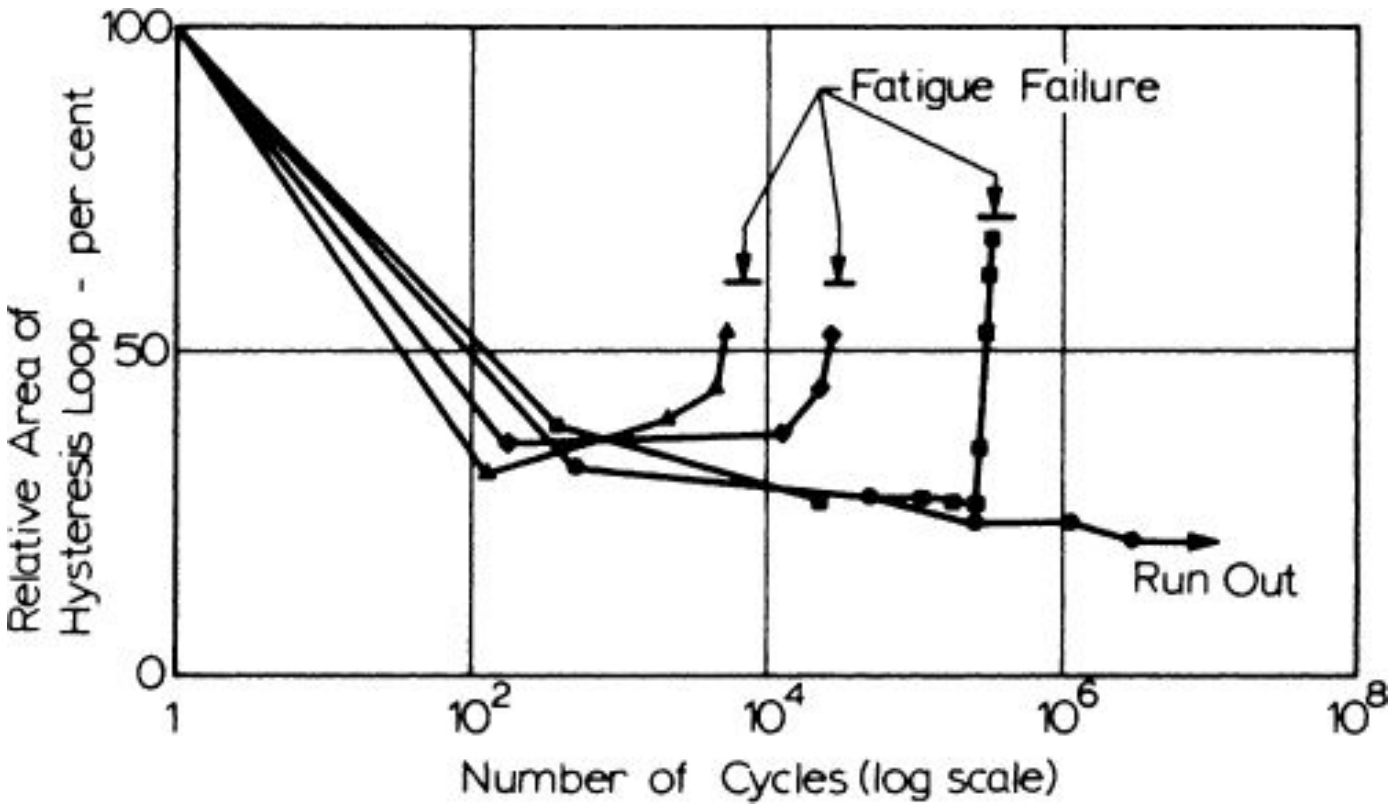


Fig. 7.16. Variation in the area of the hysteresis loop as a percentage of the first loop with the number of cycles[7.43]

The interest in the hysteresis loop arises from the fact that its area represents the irreversible

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energy of deformation, and is manifested by a rise in temperature of the specimen. The irreversible deformation involved is probably in the form of microcracking. Pulse velocity measurements have shown[7.43] that it is the development of cracks that is responsible for the change in behaviour near failure.

The strain at failure in fatigue is much larger than in static failure and can be as high as 4×10^{-3} after 13 million cycles at 3 Hz. Generally, the specimen with a longer fatigue life has a higher non-elastic strain at failure (Fig. 7.17).

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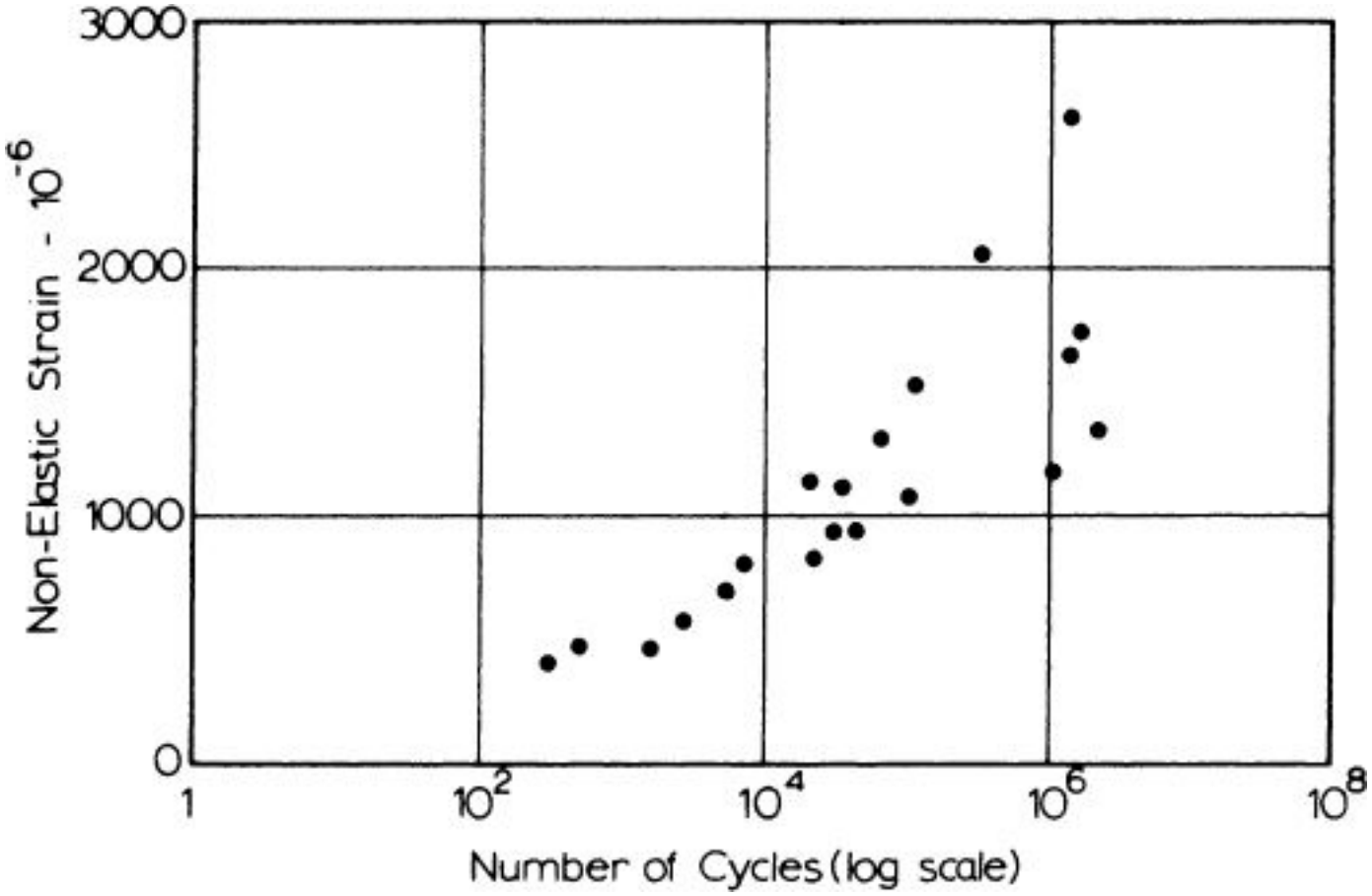


Fig. 7.17. Relation between non-elastic strain near failure and number of cycles at failure[7.43]

The elastic strain also increases progressively with cycling. This is shown in Fig. 7.18 by the reduction in the secant modulus of elasticity (see p. 414) with an increase in the percentage of the 'fatigue life' used up. This relation is independent of the level of stress in the fatigue test and is, there-

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fore, of interest in assessing the remaining fatigue life of a given concrete.

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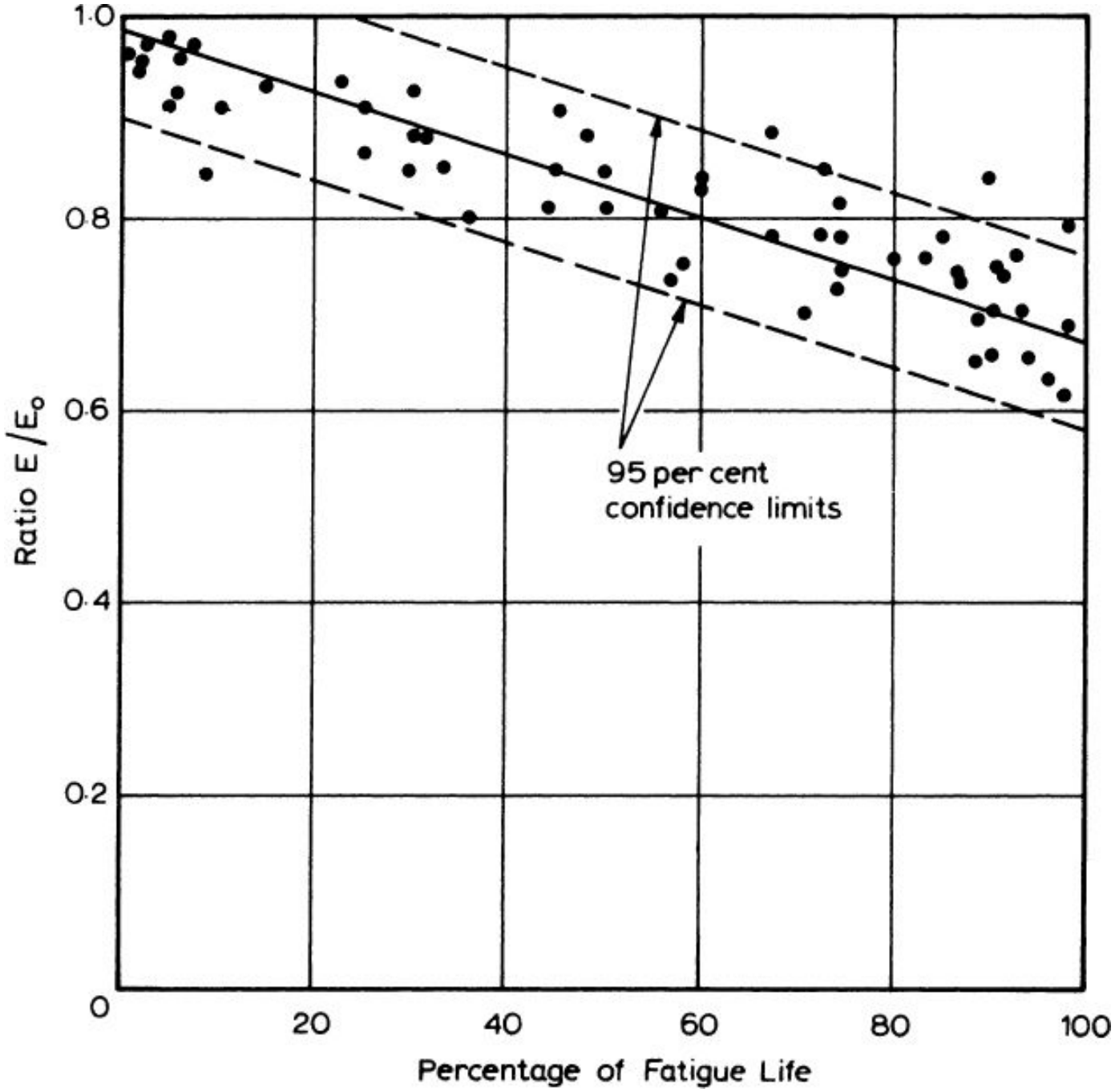


Fig. 7.18. Relation between the ratio of the secant modulus of elasticity at the given instant (E) to the modulus at the beginning of cycling (E_0) and the percentage of fatigue life used up[7.43]

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The lateral strain is also affected by the progress of cyclic loading, the Poisson's ratio decreasing progressively.

Cyclic loading below the fatigue limit improves the fatigue strength of concrete, i.e. concrete loaded a number of times below its fatigue limit will, when subsequently loaded above the limit, exhibit a higher fatigue strength than concrete which had never been subjected to the initial cycles. The former concrete also exhibits a higher static strength by some 5 to 15 per cent, but a value as high as 39 per cent has been reported.[7.85] It is probable that this increase in strength is due to a densification of concrete caused by the initial low-stress level cycling, in a manner similar to improvement in strength under moderate sustained loading.[7.45] This property is akin to strain hardening in metals, and is of particular interest because concrete under static loading is a strain-softening rather than strain-hardening material.

Strictly speaking, concrete does not appear to have a fatigue limit, i.e. a fatigue strength at an

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infinite number of cycles (except when stress reversal takes place). It is usual, therefore, to refer to fatigue strength at a very large number of cycles, such as 10 million, but for some sea structures an even higher number may be appropriate.

The fatigue strength can be represented by means of a modified Goodman diagram (see Fig. 7.19). The ordinate from a line at 45° through the origin shows the range of stress ($\sigma_h - \sigma_1$) for a given number of cycles; σ_1 is generally greater than zero (arising from the dead load) while σ_h is due to the dead plus live (transient) load. Thus, the range of stress that a given concrete can withstand a specified number of cycles can be read off the diagram. For a given σ_1, the number of cycles is very sensitive to the range of stress. For instance, an increase in range from 57.5 to 65 per cent of the ultimate static strength has been found to decrease the number of cycles by a factor of 40.[7.46]

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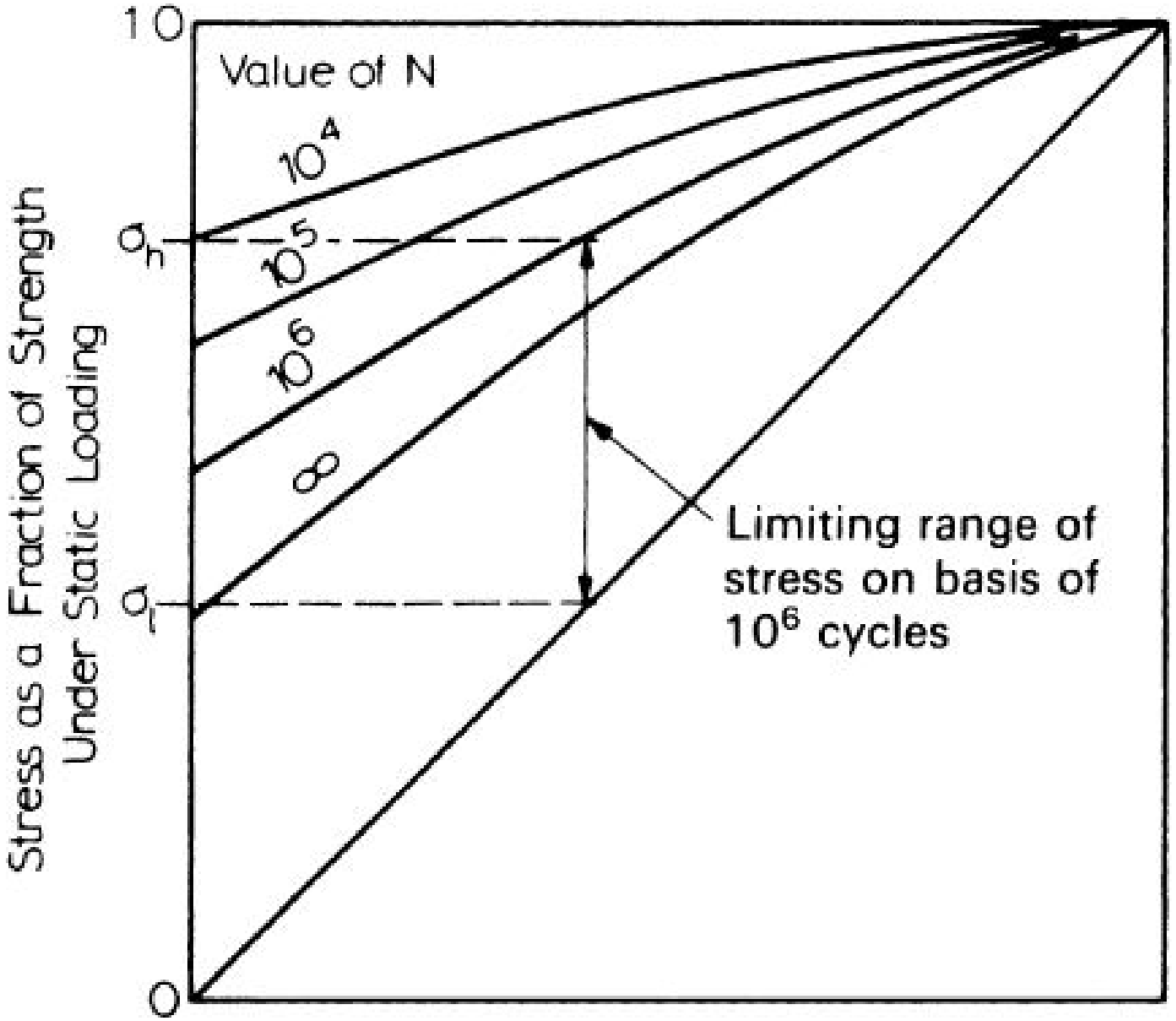


Fig. 7.19. Modified Goodman diagram for concrete in compression fatigue (*N* is number of cycles)

The modified Goodman diagram (see Fig. 7.19) shows that, for a constant range of stress, the higher the value of the minimum stress the lower the number of cycles that a given concrete

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can withstand. This is of significance in relation to the dead load of a concrete member which is to carry a transient load of a certain magnitude.

From the fact that the lines of Fig. 7.19 rise to the right, it can also be seen that the fatigue strength of concrete is lower the higher the ratio σ_h/σ_1.

The frequency of the alternating load, at least within the limits of 1.2 to 33 Hz, does not affect the resulting fatigue strength;[7.47] higher frequency is of little practical significance. This applies both in compression and in flexure, the similarity between fatigue behaviour in the two types of loading, as well as in splitting tension,[7.63] suggesting that the failure mechanism is the same.[7.48] In fact, the fatigue behaviour in flexure parallels closely that in compression (Fig. 7.20). The fatigue strength in flexure (for 10 million cycles) was found to be 55 per cent of the static strength;[7.84] values of 64 to 72 per cent have also been reported.[7.99] By comparison, in com-

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pression, the fatigue strength was reported to be between 60 and 64 per cent after the same number of cycles, but a value of 55 per cent has also been quoted.[7.85] Because of a high scatter in the fatigue test results, the application of the concept of probability of survival in fatigue has to be used in design.[7.95]

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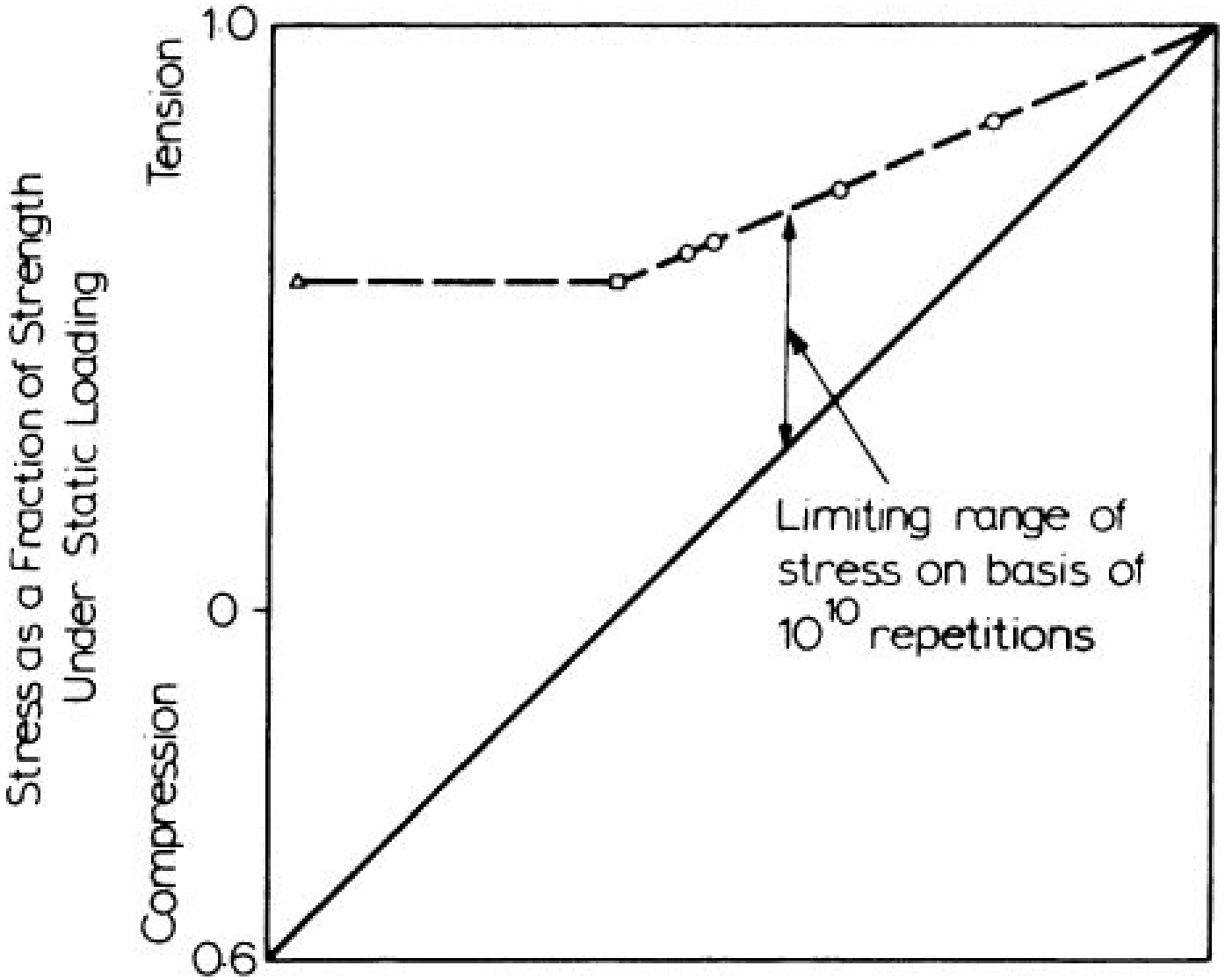


Fig. 7.20. Modified Goodman diagram for concrete in flexure fatigue[7.44]

Some tests have shown that lateral pressure increases the fatigue life of concrete, but not at very high stresses.[7.58] Generally, the pattern of fatigue behaviour of plate-shaped specimens in biaxial compression is broadly similar to that under uniaxial compression; compressive lateral stress

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of 0.2 and 0.5 of the axial stress was found to increase the fatigue life by up to 50 per cent compared with that under uniaxial compression.[7.87] An increase in the fatigue life of cubes under biaxial compression was also reported.[7.96] The reason for this is probably the fact that a compressive lateral stress restrains the development of microcracking which is responsible for fatigue failure. This observation is of interest as in many structural situations lateral compression is present.

Some tests have shown that the moisture condition of concrete prior to loading affects its fatigue strength in flexure: oven-dried specimens show the highest strength and partially dried ones the lowest; wet specimens are in between (Fig. 7.21). The explanation of this behaviour lies in differential strains induced by the moisture gradient.[7.59] The apparent effect is thus test-related. Submersion in water does not affect the fatigue life.[7.86]

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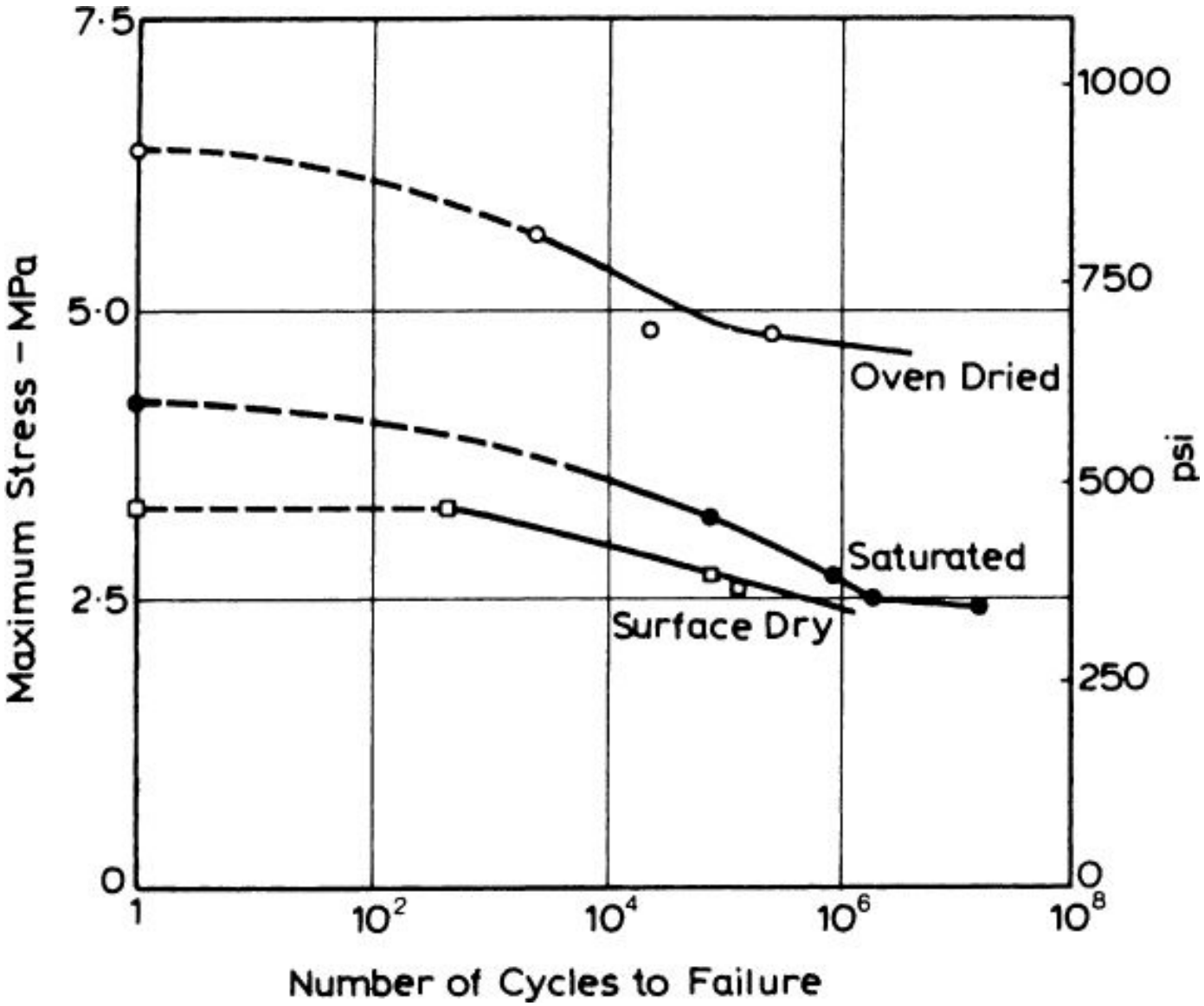


Fig. 7.21. Effect of moisture condition on fatigue performance of concrete specimens[7.59] (Crown copyright)

Generally speaking, the ratio of fatigue strength to static strength is independent of the water/cement ratio, the cement content, type of aggregate, and age at loading because these

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factors affect both the static and the fatigue strength in the same manner.

As strength increases with age, fatigue strength both in compression and in flexure also increases.[7.63] The important point is that, at a given number of cycles, fatigue failure occurs at the same fraction of ultimate strength, and is thus independent of the magnitude of this strength (both in compression and in splitting tension)[7.64] and of the age of concrete,[7.47] although some tests suggest an increase in fatigue life with age.[7.59] It can thus be seen that a single parameter is critical in fatigue failure. Murdock[7.47] expressed the view that the deterioration of bond between the hydrated cement paste and aggregate is responsible for this failure. Tests have shown that fatigue specimens had fewer broken aggregate particles than specimens which failed in a static test.[7.49] Thus, failure at the aggregate–paste interface is probably dominant in fatigue; in mortar, fatigue failure is believed to take place at the interface

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of the fine aggregate particles.[7.43] It is likely that a smaller maximum size of aggregate leads to a higher fatigue strength,[7.60] probably because of greater homogeneity of concrete.

Air-entrained concrete and lightweight aggregate concrete have the same fatigue behaviour as concrete made with ordinary aggregate,[7.50,7.61,7.86] although air entrainment may reduce the fatigue life in flexure.[7.98] Fatigue in concrete cylinders occurs in the same way as in large specimens subjected to low-frequency loading.[7.62]

High strength concrete also exhibits behaviour similar to ordinary concrete, but shows a lower deformation (probably due to a higher modulus of elasticity) and a higher fatigue life under high values of maximum stress.[7.83] The performance of high strength concrete in fatigue can thus be considered to be good, but failure is rather sudden.[7.83]

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The fatigue strength of concrete is increased by rest periods (this does not apply when there are stress reversals), the increase being proportional to their duration between 1 and 5 minutes; beyond the 5-minute limit there is no further increase in strength. With the rest periods at their maximum effective duration, their frequency determines the beneficial effect.[7.47] The increase in strength caused by rest periods is probably due to relaxation of concrete (primary bonds, which remained intact, restoring the internal structure to its original configuration), as evidenced by a decrease in the total strain; this decrease occurs rapidly after the cessation of cycling.

Murdock[7.47] suggested that fatigue failure occurs at a constant strain, independent of the applied stress level or of the number of cycles necessary to produce failure. This behaviour of concrete would add further support to the concept of ultimate strain as failure criterion.

Most fatigue tests are conducted under cyclic loading of constant shape. However, structures

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such as those subjected to wave action undergo variable amplitude loading. Tests involving variable stress levels have shown that the sequence of low-stress and high-stress cycling affects the fatigue life. In particular, if high-stress cycling succeeds the low-stress cycling, the fatigue strength is reduced. It follows that Miner's hypothesis[7.88] of linear accumulation of damage (valid for metals) does not apply to concrete[7.44,7.65,7.89] and may err on the unsafe side. A modification of Miner's hypothesis which takes into account the sequence of variable amplitude loading was developed by Oh;[7.100] its general validity is still to be established.

It should also be noted that, for a given maximum stress in the cycle, as the amplitude of stresses decreases, we are no longer dealing with fatigue but rather with sustained loading which leads to creep failure (see p. 474). The duration of cycling becomes therefore important. Expressions taking this into account were developed by Hsu,[7.90] who considers that separate equations for

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fatigue life are needed for low-cycle loading of the type caused by earthquakes; direct application of test results from laboratory tests at high frequency may be unsafe.[7.97]

While this book is not concerned with the fatigue behaviour of reinforced and prestressed concrete, we should note that fatigue cracks in concrete act as stress-raisers, thus magnifying the vulnerability of the steel to fatigue failure[7.51] (if the stress in it is in excess of its critical fatigue stress value).

Another observation relevant to reinforced concrete is that the fatigue strength of bond of concrete with the reinforcement is the governing factor in reinforced concrete subjected to cyclic loading.[7.86] As bond is improved by the incorporation of silica fume in the mix, this would explain why the presence of silica fume in high strength lightweight aggregate concrete increases the fatigue strength of reinforced concrete members, compared with members made with concrete of the same strength but without silica fume.

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It is possible that the fatigue of bond with reinforcement is best expressed in terms of cumulative deformation (slip) in a static bond test.[7.82]

Impact strength

Impact strength is of importance when concrete is subjected to a repeated falling object, as in pile driving, or a single impact of a large mass at a high velocity. The principal criteria are the ability of a specimen to withstand repeated blows and to absorb energy.

Green[7.52] studied the number of blows of a ballistic pendulum which 100 mm (4 in.) concrete cubes can withstand before reaching the no-rebound condition, this stage indicating a definite state of damage. He found that impact tests on compression specimens, when conducted with a small hammer (25 mm (1 in.) diameter face), lead to a greater scatter of results than tests on static compressive strength of the concrete. This arises from the fact that, in the standard compression test, some relief of a highly stressed weak zone

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is possible due to creep, whereas in the impact test no redistribution of stresses is possible during the very short period of deformation. Hence, local weaknesses have a greater influence on the recorded strength of a specimen.

In general, the impact strength of concrete increases with an increase in compressive strength,[7.92] but the higher the static compressive strength of the concrete the lower the energy absorbed per blow before cracking.[7.52]

Figure 7.22 gives some examples of the relation between the impact strength and the compressive strength.[7.52] It can be seen that the relation is different for each coarse aggregate and storage condition of the concrete. For the same compressive strength, the impact strength is greater for coarse aggregate of greater angularity and surface roughness. This observation was confirmed by Dahms[7.66] and supports the suggestion[7.53] that impact strength is more closely related to the tensile strength of concrete than to its

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compressive strength. Thus, concrete made with a gravel coarse aggregate has a low impact strength, failure taking place due to insufficient bond between mortar and coarse aggregate. On the other hand, when the surface of the aggregate is rough, the concrete is able to develop the full strength of much of the aggregate in the region of failure.

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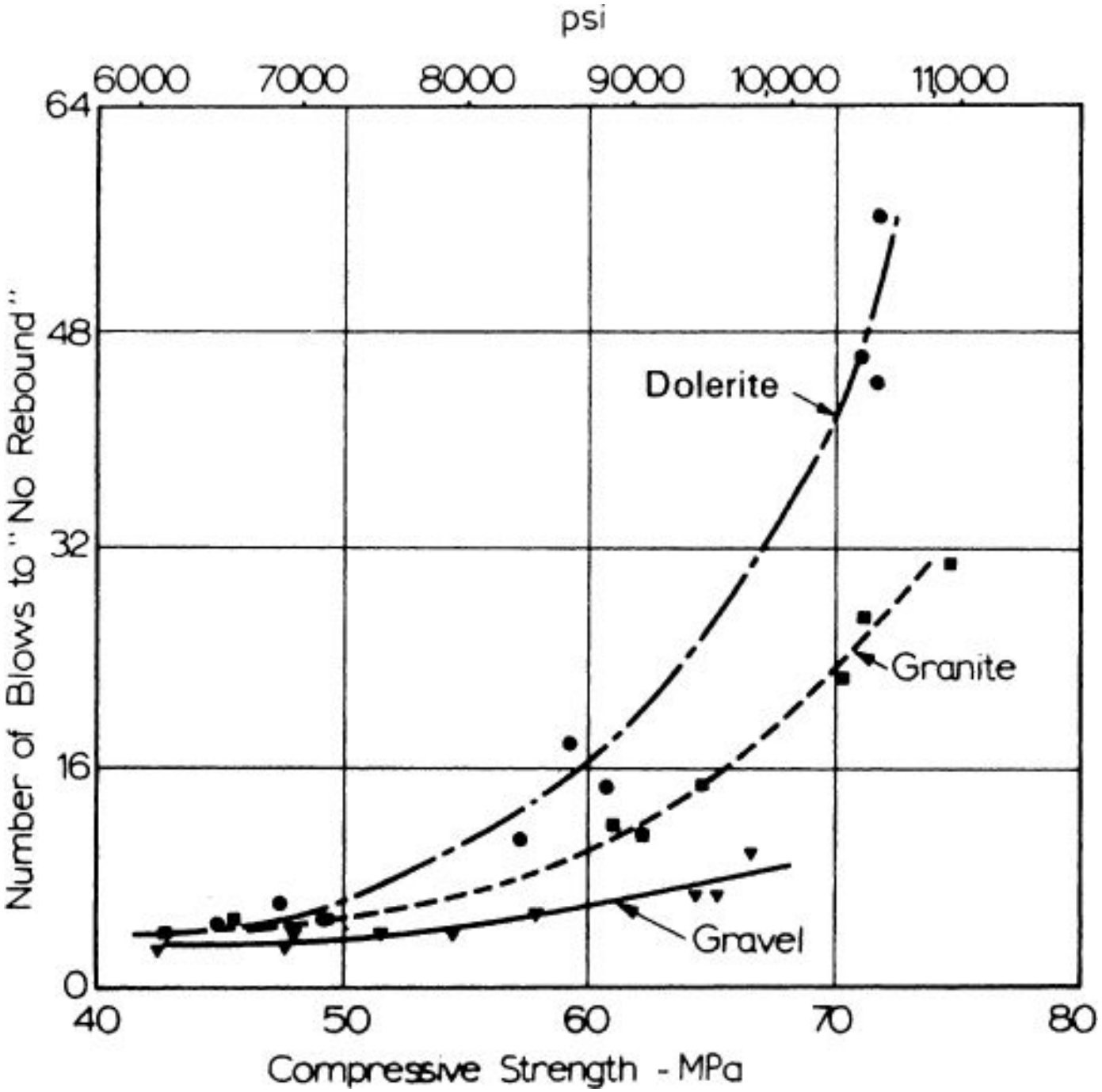


Fig. 7.22. Relation between compressive strength and number of blows to 'no-rebound' for concretes made with different aggregates and Type I cement, stored in water[7.52]

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A smaller maximum size of aggregate significantly improves impact strength both in compression[7.66] and in splitting tension.[7.93] Impact strength in compression is improved by the use of aggregate with a low modulus of elasticity and a low Poisson's ratio.[7.66] Cement content below 400 kg/m^3 (670 lb/yd^3) is advantageous.[7.66] The influence of fine aggregate is not well defined but the use of fine sand usually leads to a slightly lower impact strength. Dahms[7.66] found a high content of sand advantageous. We could try to generalize and say that a mix of materials which have a limited variation in properties is conducive to a good impact strength. Extensive tests on the impact strength of concretes with different properties were made by Hughes and Gregory.[7.54]

Storage conditions influence the impact strength in a manner different from compressive strength. Specifically, the impact strength of water-stored concrete is lower than when the concrete is dry, although the former concrete can

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withstand more blows before cracking. Thus, as already stated, the compressive strength without reference to storage conditions, does not give a satisfactory indication of the impact strength.[7.52]

Repeated impact tests on slabs have also been used,[7.92] the end point being perforation of the slab. Such tests are usually directed towards a direct structural application and often involve fibre-reinforced concrete. Impact tests in splitting tension can also be performed.

There is evidence that, under uniformly applied impact loading (a condition difficult to achieve in practice), the impact strength of concrete is significantly greater than its static compressive strength. This increase in strength would explain the greater ability of concrete to absorb strain energy under uniform impact. Figure 7.23 shows that strength increases greatly when the rate of application of stress exceeds about 500 GPa/s, reaching, at 4.9 TPa/s, more than double the value at normal speeds of loading (about 0.5 MPa/s).[7.67] Impact at a loading rate six orders of

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magnitude greater than in a static test led to a 50 per cent increase above the static compressive strength.[7.91] In splitting tension, the same increase in the loading rate was found to result in an 80 per cent increase above the static strength.[7.93]

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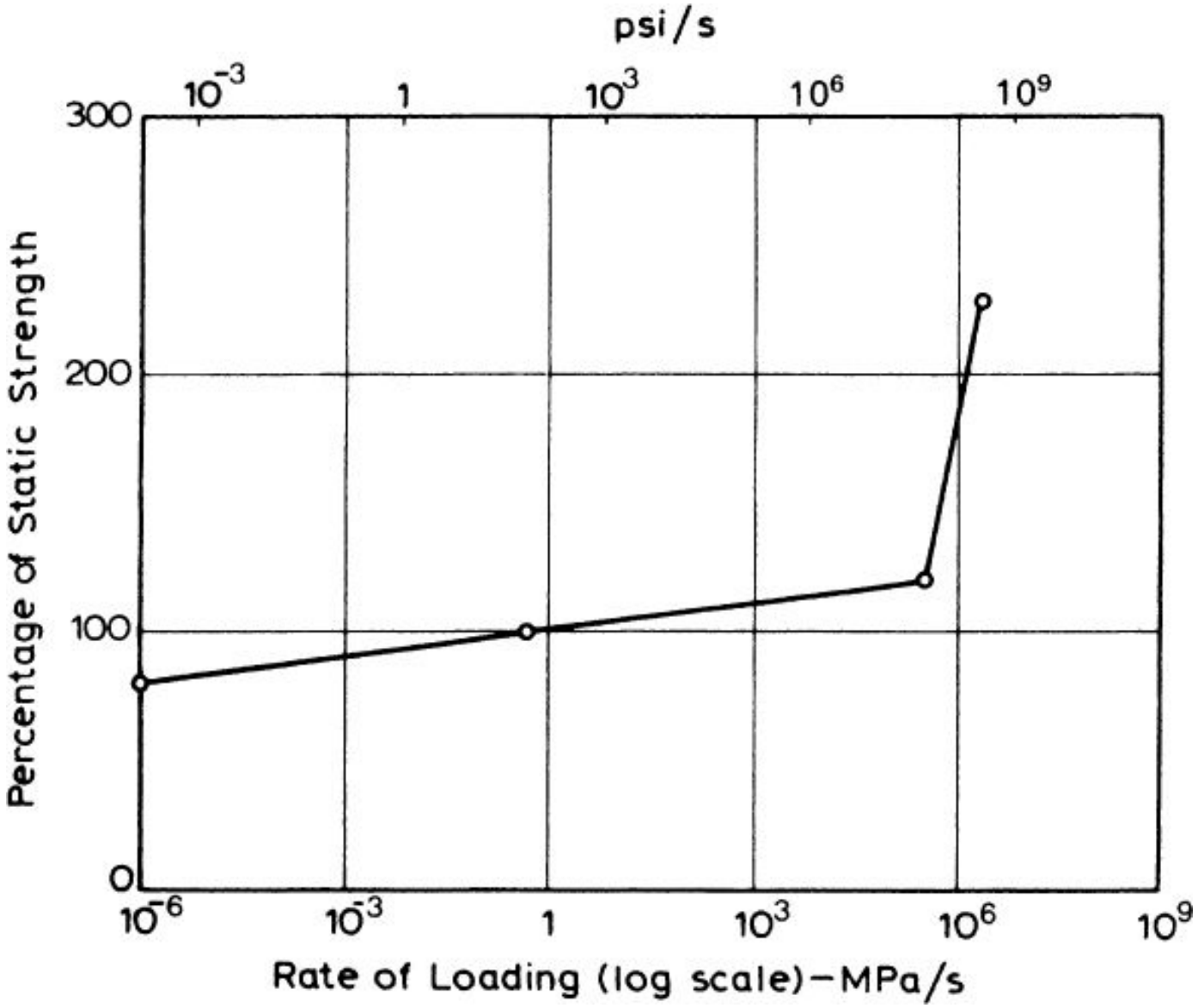


Fig. 7.23. Relation between compressive strength and rate of loading up to impact level[7.67]

The influence of the rate of application of strain on compressive strength is shown in Fig. 7.24. It can be seen that, at very high strain rates, there is a large increase in compressive strength,

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probably due to inertial resistance of concrete to microcracking;[7.80] at low rates, the effect of creep may be dominant. The influence of the strain rate upon the tensile strength of concrete is even larger,[7.81] the free water in the hardened cement paste playing a significant role.[7.79] The subject of the influence of the rate of loading on strength is considered also in Chapter 12 in connection with testing.

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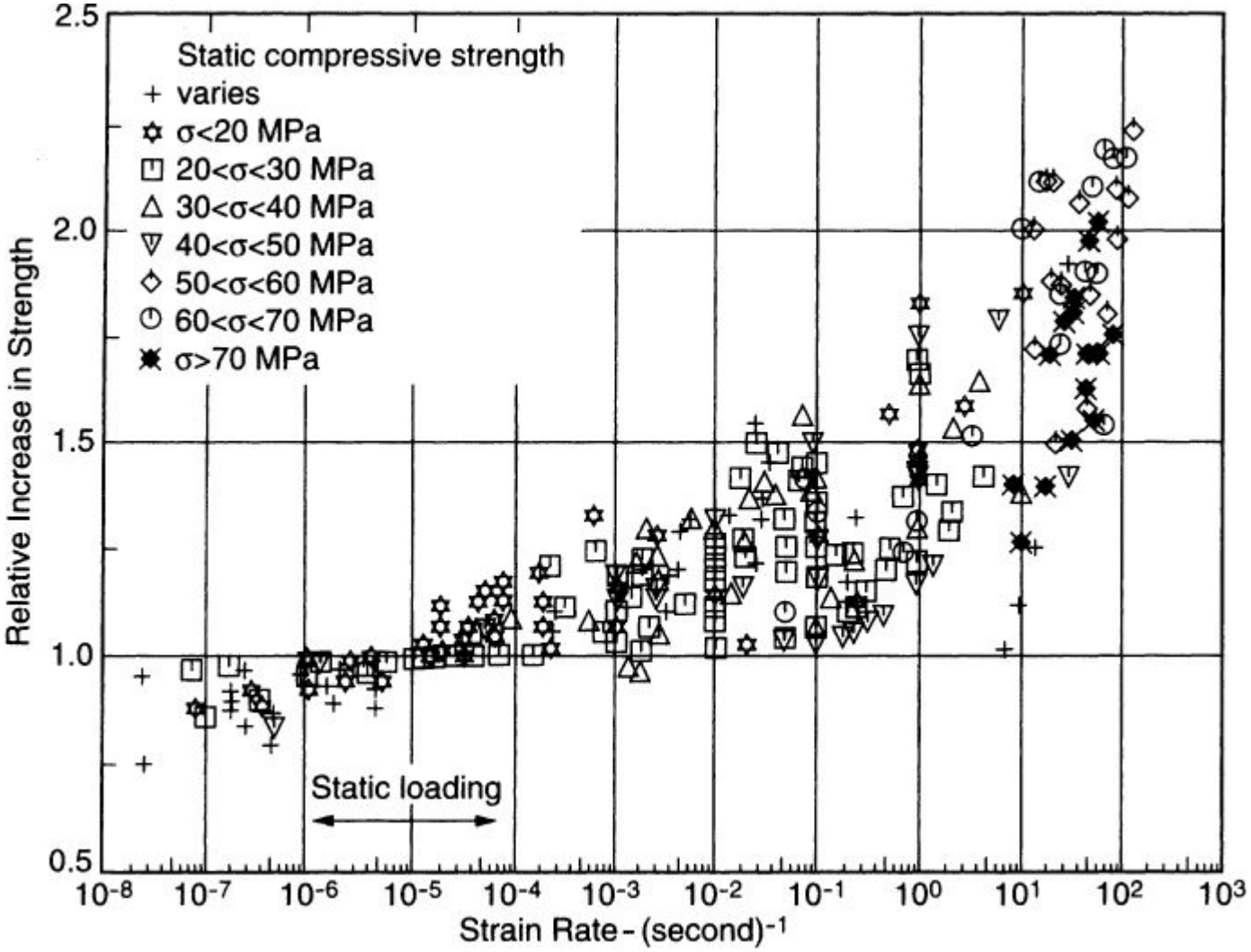


Fig. 7.24. Relation between relative increase in compressive strength (as a proportion of static strength) and the strain rate for concretes of different strengths (based on ref. 7.80)

Electrical properties of concrete

Electrical properties are of concern in some specific applications such as railway ties (sleepers)

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(where inadequate resistivity affects some signalling systems) or in structures in which concrete is used for protection from stray currents. Electrical resistance of concrete also influences progress of corrosion of embedded steel. Electrical properties are also of interest in studies of the properties of both fresh and hardened concrete.

In the vicinity of underground cables, concrete may be subjected to impressed electrical activity but, under the usual operating conditions, concrete offers a high resistance to the passage of electric current to or from embedded steel. This is largely due to the electro-chemical effect which concrete has on steel in contact with it, arising from the alkalinity of the electrolyte within the concrete. Such a protection applies within the potential range of about +0.6 to –1.0 V (with respect to a copper sulfate electrode), the current being primarily controlled by polarization effects and not by the ohmic resistance of concrete.[7.69]

Moist concrete behaves essentially as an electrolyte with *resistivity* of up to about 100 ohm-

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m; this is within the range of semi-conductors. Air-dried concrete has a resistivity of the order of 10^4 ohm-m.[7.19] On the other hand, oven-dry concrete has a resistivity of about 10^9 ohm-m, which means that such concrete is a good insulator.[7.70] The insulating or dielectric properties have been studied by Halabe *et al*.[7.27]

This large increase in resistivity of concrete on removal of water is interpreted to mean that electric current is conducted through moist concrete essentially by electrolytic means, that is by ions in the evaporable water. However, when the capillaries are segmented, passage of the electric current through gel water takes place. The resistivity of normal aggregate is infinitely larger. For concrete of given mix proportions, drying out in the air increases the resistivity of the surface zone. For instance, at a water/cement ratio of 0.50, Tritthart and Geymayer[7.34] reported an eleven-fold increase; the increase was even larger at higher water/cement ratios.

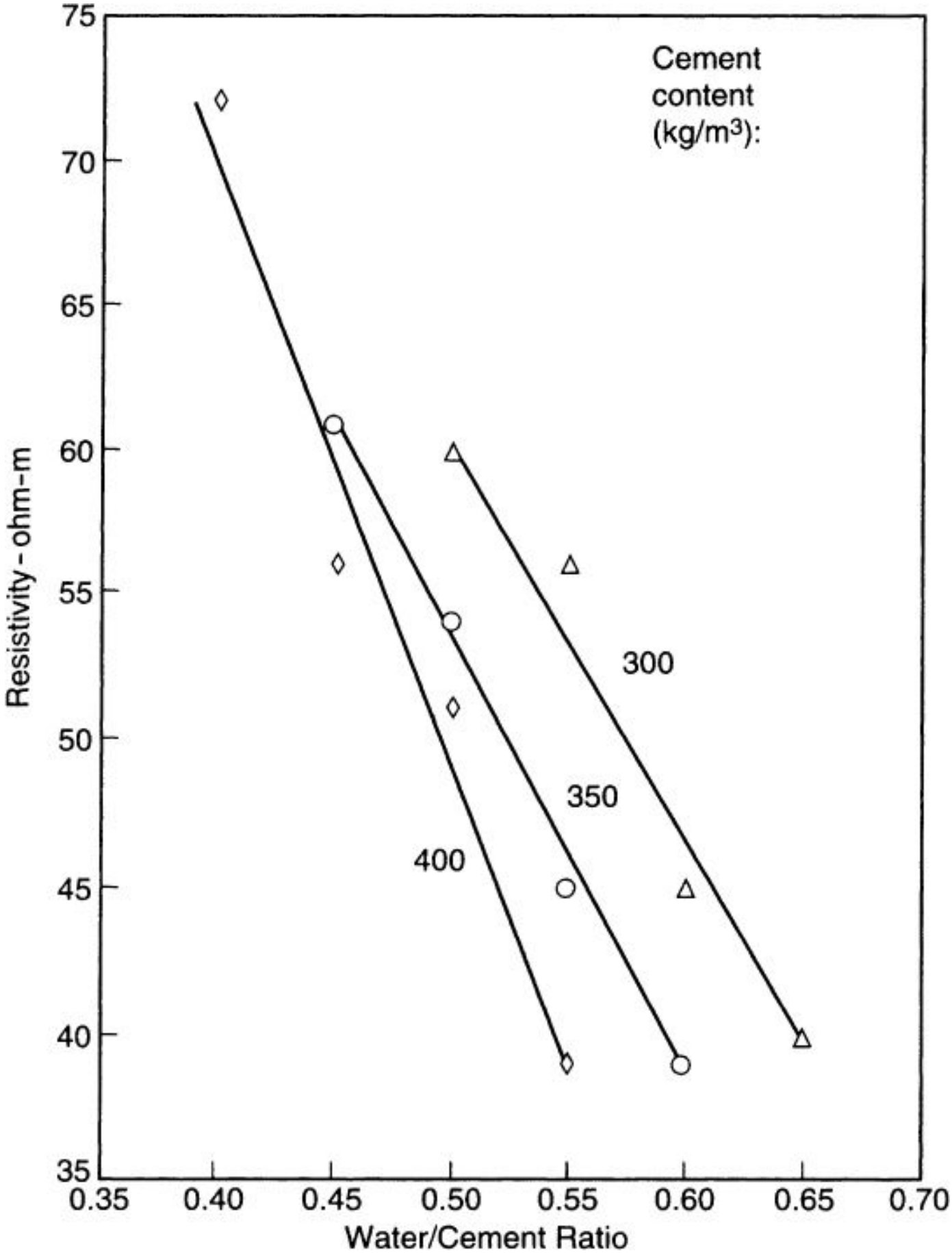
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It can therefore be expected that any increase in the volume of water and in concentration of ions present in the pore water decreases the resistivity of cement paste, and indeed resistivity decreases sharply with an increase in the water/cement ratio. This is shown in Table 7.3 for hydrated cement paste and in Fig. 7.25 for concrete. A decrease in the cement content of the concrete also results in an increased resistivity[7.18] because, at a constant water/cement ratio but a lower cement content, there is less electrolyte available for the current to pass.

Table 7.3. Influence of Water/Cement Ratio and Length of Moist Curing on Resistivity of Cement Paste[7.70]

Cement type	*Equivalent Na_2O content, per cent*	*Water/cement ratio*	*Resistivity (at 1000 Hz, 4V), ohm-m at the age of:*		
			7 days	*28 days*	*90 days*
Ordinary Portland	0.19	0.4	10.3	11.7	15.7
		0.5	7.9	8.8	10.9
		0.6	5.3	7.0	7.6
Ordinary Portland	1.01	0.4	12.3	13.6	16.6
		0.5	8.2	9.5	12.0
		0.6	5.7	7.3	7.9

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Fig. 7.25. Relation between electrical resistivity and water/cement ratio for concrete with a maximum size of aggregate of 40 mm ($1\frac{1}{2}$ in.) made with ordinary (Type I) Portland cement, tested at the age of 28 days (based on ref. 7.18)

The resistivity of concretes of varying composition is given by Hughes *et al*.[7.18] If necessary, the values of resistivity of hydrated cement paste can be converted into resistivity of concrete which includes this paste, approximately in an inverse ratio of the relative volume of hydrated cement paste.[7.19]

The long-term reactions involving ground granulated blastfurnace slag in concrete cause a continuing increase in electrical resistivity. This can be as much as an order of magnitude, compared with concrete containing Portland cement only.[7.30] Silica fume also increases the resistivity. The effects of ground granulated blastfurnace slag and silica fume are of significance when the

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progress of corrosion of steel reinforcement is controlled by the electrical resistance of concrete (see Chapter 11).

Like other ions in the pore water, chlorides greatly reduce the resistivity of concrete and mortar; for the latter, a 15-fold decrease was reported.[7.71] The influence on resistivity of salinity of the mixing water is greatest in concrete with high water/cement ratios and is quite small in high strength concrete.[7.72]

During the first few hours after mixing, the resistivity of concrete increases very slowly, then increases rapidly up to the age of about 1 day, and thereafter increases at a reduced rate or becomes constant[7.18] unless the concrete dries out; drying increases the resistivity.

The resistivity of concrete immersed in sea water can become greatly increased by the formation of a thin surface layer of magnesium hydroxide and calcium carbonate.[7.101] If this layer is re-

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moved, the resistivity is the same as for storage in fresh water.

The relation between resistivity of concrete and the volume fraction occupied by water can be derived from the laws of conductivity of heterogeneous conductors. However, for the range of the usual concrete mixes, the water content varies comparatively little for a given aggregate grading and workability, and the resistivity becomes more dependent on the cement used[7.73] because the chemical composition of the cement controls the quantity of ions present in the evaporable water. Some idea of the influence of cement on resistivity can be obtained from Table 7.4, from which it can be seen that the resistivity of concrete made with high-alumina cement is 10 to 15 times higher than when Portland cement in the same proportions is used[7.73] (see Fig. 7.26).

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Table 7.4. Typical Electrical Properties of Concrete (based on ref. 7.74)

Mix and water/ cement ratio	Type of cement	Period of air drying days	Resistivity (10^3 ohm-m)				Capacitative reactance, 10^3 ohm			Capacitance, microfarad		
			d.c.	50 Hz	500 Hz	25 000 Hz	50 Hz	500 Hz	25 000 Hz	50 Hz	500 Hz	25 000 Hz
	Ordinary	7	10	9	9	9	159	159	32	0.020	0.0020	0.0002
	Portland	42		31	31	30	637	455	64	0.005	0.0007	0.0001
1:2:4*		113	90	82	80	73	1061	398	64	0.003	0.0008	0.0001
0.49	Rapid-hardening Portland	39		28	27	27	796	398	64	0.004	0.0008	0 0001
	High-alumina	5		189	173	139	398	228	106	0.008	0.0014	0.00006
		18		390	351	275	664	398	127	0.005	0.0008	0.00005
		40		652	577	441	910	569	159	0.003	0.0006	0.00004
	Ordinary											
1:2:4†	Portland	126		59	58	58	118	228	127	0.027	0.0014	0.00005
0.49	Rapid-hardening	123		47	47	46	118	212	32	0.027	0.0015	0.00020
	High-alumina	138		1236	1080	840	531	398	106	0.006	0.0008	0.00006
		182		1578	1380	1059	692	424	127	0.005	0.0007	0.00005
Cement	Ordinary											
paste‡	Portland	9	7	6	6	6	9	10	3	0.350	0.0300	0.0020
0.23	Rapid-hardening	9	5	5	5	5	6	6	2	0.500	0.0540	0.0026
	High-alumina	13	240	220	192	128	80	41	21	0.040	0.0077	0.0003

*102 mm cube; external electrodes.
†152 mm cube; embedded electrodes.
‡25 mm thick prism; external electrodes.

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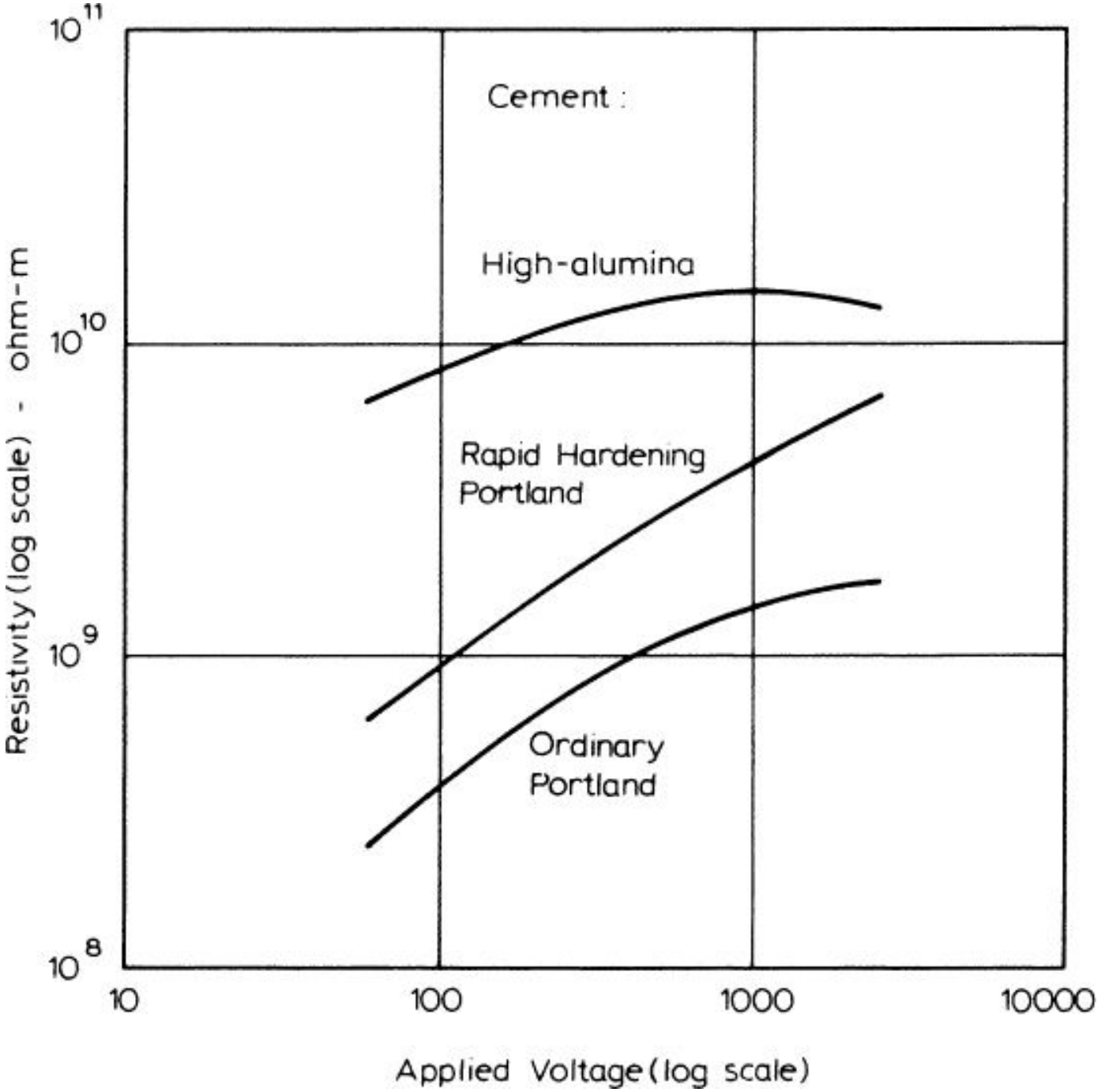


Fig. 7.26. Relation between resistivity and applied voltage for a 1:2:4 concrete with a water/cement ratio of 0.49, oven-dried and cooled in a desiccator[7.74]

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Admixtures generally do not reduce the resistivity of concrete.[7.70] However, special additions can be used to vary the resistivity. For instance, the addition to concrete of finely divided bituminous material, with subsequent heat treatment at 138 °C (280 °F), increases the resistivity, especially under wet conditions.[7.75] Conversely, in cases where static electricity is undesirable and a decrease in the insulation resistance of concrete is required, satisfactory results can be achieved by the addition of acetylene carbon black (2 to 3 per cent by mass of cement).[7.75] Electrically conductive concrete can be obtained by replacing fine aggregate with a granulated conductive aggregate consisting of almost pure crystalline carbon prepared as a proprietary product. Resistivity is between 0.005 and 0.2 ohm-m; the compressive strength and other properties are reported not to be significantly affected.[7.76]

The resistivity of concrete increases with an increase in voltage.[7.74] Figure 7.26 illustrates this

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relation for oven-dried specimens not allowed to absorb moisture during the test. Resistivity of concrete decreases with an increase in temperature.[7.19]

The majority of values quoted in this section are given for alternating current (a.c). The resistivity to direct current (d.c.) may be different because it has a polarizing effect, but at 50 Hz there is no significant difference between resistivity to a.c. and d.c.[7.74] In general, for concrete matured in air, the d.c. resistance is approximately equal to the a.c. impedance.[7.74] Hammond and Robson[7.74] interpreted this to mean that the capacitative reactance of concrete is so much larger than its resistance that it is only the latter that contributes significantly to impedance; as a consequence, the power factor is nearly unity. Typical data for alternating current are given in Table 7.4.

The capacitance of concrete decreases with age and with an increase in frequency.[7.74] Neat

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cement paste with a water/cement ratio of 0.23 has a much higher capacitance than concrete with a water/cement ratio of 0.49 at the same age.[7.74]

Data on the dielectric strength of concrete are given in Table 7.5. It can be seen that the dielectric strength of concrete made with high-alumina cement is slightly greater than when Portland cement is used. The table shows also that, despite the much higher moisture content (and therefore lower resistivity) of air-stored concrete compared with oven-dried concrete, the dielectric strength is approximately the same for the two storage conditions, and seems thus to be unaffected by moisture content.

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Table 7.5. Dielectric Strength of Concrete (1:2:4 Mix with Water/*Cement Ratio of 0.49*)[7.74]

Condition of concrete	*Current*	*Breakdown*	*Dielectric strength 10^6 V/m*		
			Ordinary Portland cement	*Rapid-hardening Portland cement*	*High-alumina cement*
Stored in air	Positive impulses 1/44 μs		1.44	1.46	1.84
		First	1.59	1.33	1.77
	d.c. negative	Second	1.18	1.06	1.24
Dried at 104 °C (220 °F)		Third	1.25	0.79	1.28
air cooled	a.c. (50 Hz)	First	1.43	1.19	1.58
	peak values	Second	1.03	1.00	1.21
		Third	1.00	0.97	0.95

Acoustic properties

In many buildings, acoustic characteristics are of importance and these may be greatly influenced by the material used and by structural details. Here, only the properties of the material will be considered, the influence of the structural form and construction details being a specialized topic.

Basically, two acoustic properties of a building material can be distinguished: sound absorp-

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tion and sound transmission. The former is of interest when the source of sound and the listener are in the same room. Energy of sound waves, when they hit a wall, is partly absorbed and partly reflected, and we can define a *sound absorption coefficient* as a measure of the proportion of the sound energy striking a surface which is absorbed by that surface. The coefficient is usually given for a particular frequency. Sometimes, the term 'noise reduction coefficient' is used to denote the average of sound absorption coefficients at 250, 500, 1000, and 2000 Hz in octave steps. A typical value for normal weight aggregate concrete of medium texture, unpainted, is 0.27. The corresponding value for concrete made with expanded shale aggregate is 0.45. The difference is related to the variation in texture, porosity and structure because, when airflow is possible, there is a large increase in sound absorption through conversion of sound energy into heat, by friction. Thus, cellular concrete, which has discrete air bubbles, exhibits lower sound absorption than concrete made with porous lightweight aggregate.

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Sound transmission is of interest when the listener is in a room adjacent to that in which the source of sound is located. We define the *sound transmission loss* (or airborne-sound insulation) as the difference, measured in decibels (dB), between the incident sound energy and the transmitted sound energy (which radiates into an adjoining room). What constitutes a satisfactory transmission loss depends on the use of the given space: a value of 45 to 55 dB is thought to be adequate between dwellings.[7.22,7.25]

The primary factor in transmission loss is the unit mass of the partition per square metre of area. The loss increases with the frequency of the sound wave and is usually assessed over a range of frequencies. The relation between the transmission loss and the mass of the partition, in general terms, is independent of the type of material used, provided no continuous pores are present, and is sometimes referred to as the ‘mass law’. Figure 7.27 illustrates the relation for the case when the partition edges are ‘firmly fixed’, i.e.

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the flanking walls are of similar material. From Fig. 7.27, it can be seen that a bare concrete wall 150 to 175 mm (6 to 7 in.) thick would provide an adequate transmission loss between dwellings. Information on sound insulation of party walls is given in refs 7.22, 7.23 and 7.24; more general treatment of acoustic properties of concrete is given in ref. 7.26.

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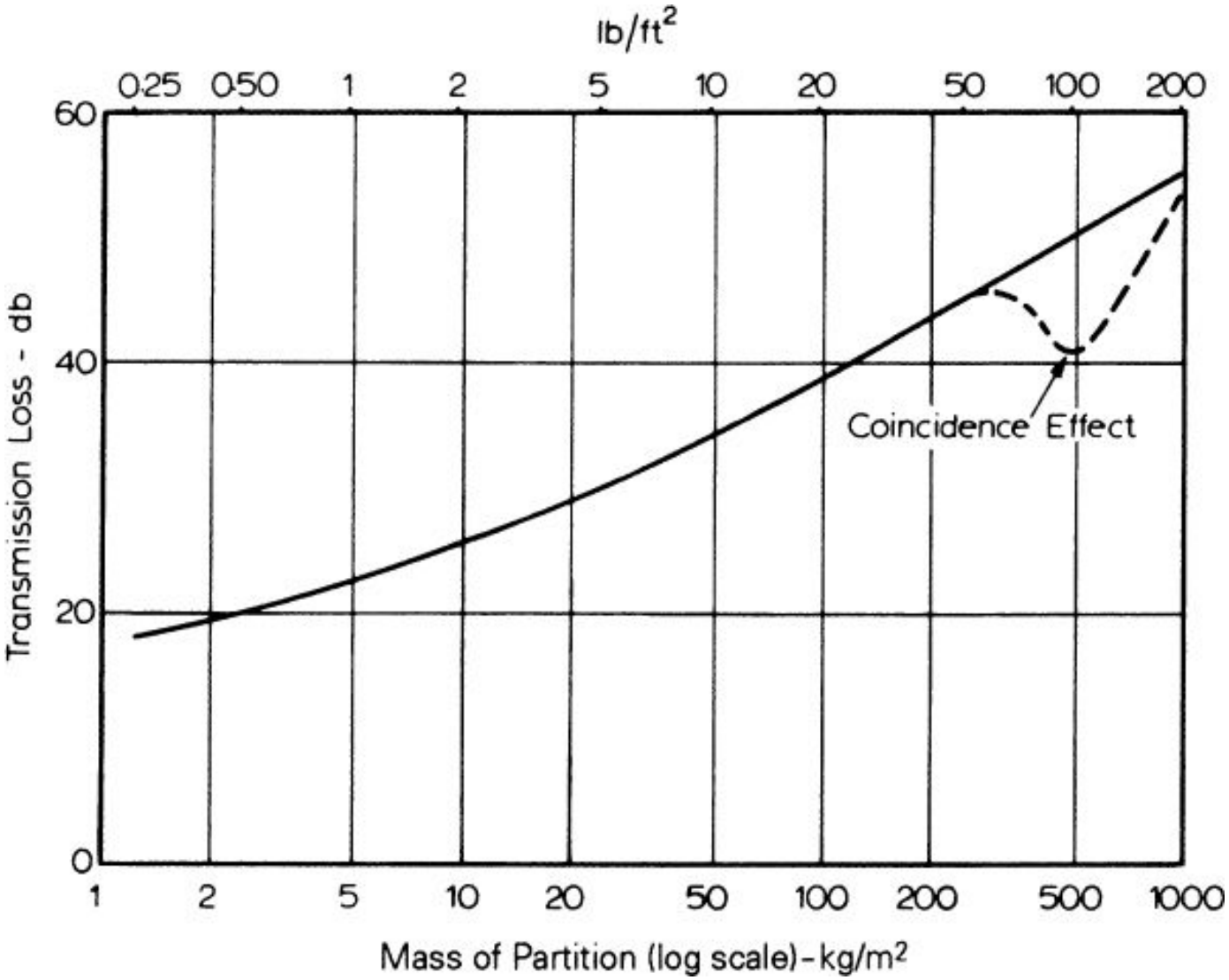


Fig. 7.27. Relation between transmission loss and unit mass of partition [7.68]

The sound transmission around the 'sound obstacle' has, of course, to be considered but, as far as the partition itself is concerned, there are some factors additional to the mass: airtightness, bending stiffness, and the presence of cavities.

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The stiffness of the partition is relevant because, if the wavelength of the forced bending wave imposed on the wall is equal to the wavelength of free bending waves in a wall, a condition of total sound transmission through the wall arises. This coincidence of wavelengths can occur only above a critical value of frequency at which the velocity of free bending waves in the wall is the same as that of air waves parallel to the wall. Above that frequency, a combination of air-wave incidence and frequency is possible at which there can occur the coincidence of air wave at interface and of the structure bending wave. The effect is usually limited to thin walls.[7.68] The critical frequency is given by:

$$q_c = \frac{v^2}{2\pi h}\left[\frac{12\rho(1-\mu^2)}{E}\right]^{1/2}$$

where v = velocity of sound in air

h = thickness of the partition

ρ = density of the concrete

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E = modulus of elasticity of concrete, and

μ = Poisson's ratio of concrete.

The influence of the coincidence effect on the relation between the sound transmission loss and the unit mass of the partition can be seen from the dotted line in Fig. 7.27.

The presence of cavities also affects this relation, a cavity increasing the transmission loss, so that the use of the given total thickness of concrete in the form of two leaves is advantageous. The quantitative behaviour depends on the width of the cavity, on the degree of isolation between the leaves, and also on the presence or absence of a sealed surface facing the cavity if the wall material is porous.

From the foregoing it is apparent that, to a considerable extent, the requirements of a high sound absorption and a high transmission loss are conflicting. For instance, the porous type of lightweight concrete has good sound-absorbing properties but a very high sound transmission. However, if one concrete face is sealed, the trans-

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mission loss is increased and can become equal to that of other materials of the same mass per unit area. It is preferable to seal the side remote from the source of the sound as, otherwise, sound absorption is impaired. However, there is no reason to believe that lightweight concrete provides inherently better insulation with respect to sound transmission.

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Chapter 8. Temperature effects in concrete

Laboratory testing of concrete is usually performed at a controlled temperature, normally constant. As the early testing was done in temperate climates, the standardized temperature chosen was generally in the region of 18 to 21 °C (64 to 70 °F) so that much of the basic information about the properties of both fresh and hardened concrete is based on the behaviour of concrete at these temperatures. In practice, however, concrete is mixed at a wide range of temperatures and also remains in service at different temperatures. Indeed, the actual range of temperatures has widened considerably with much modern construction taking place in countries which have a hot climate. Also, new developments, mainly offshore, take place in very cold regions.

In consequence, knowledge of the temperature effects in concrete is of great importance. These

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effects will be considered in the present chapter. First, the influence of the temperature of fresh concrete upon strength will be discussed; this will be followed by a review of temperature treatment after the placing of concrete, that is curing both by using steam at atmospheric pressure and by high-pressure steam. Next, the effects of the temperature rise in concrete due to the development of the heat of hydration of cement will be discussed, followed by consideration of concreting in hot weather and in cold weather. Finally, thermal properties of hardened concrete will be described and the influence of very high and very low temperatures in service, including the effects of fire, will be discussed.

Influence of early temperature on strength of concrete

We have seen that a rise in the curing temperature speeds up the chemical reactions of hydration and thus affects beneficially the early strength of concrete without any ill-effects on the later strength.

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Higher temperature during and following the initial contact between cement and water reduces the length of the dormant period so that the *overall* structure of the hydrated cement paste becomes established very early.

Although a higher temperature during placing and setting increases the very early strength, it may adversely affect the strength from about 7 days onwards. The explanation is that a rapid initial hydration appears to form products of a poorer physical structure, probably more porous, so that a proportion of the pores will always remain unfilled. It follows from the gel/space ratio rule, that this will lead to a lower strength compared with a less porous, though slowly hydrating, cement paste in which a high gel/space ratio will eventually be reached.

This explanation of the adverse effects of a high early temperature on later strength has been extended by Verbeck and Helmuth[8.77] who suggest that the rapid initial rate of hydration at higher temperatures retards the subsequent hydration

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and produced a non-uniform distribution of the products of hydration within the paste. The reason for this is that, at the high initial rate of hydration, there is insufficient time available for the diffusion of the products of hydration away from the cement particle and for a uniform precipitation in the interstitial space (as is the case at lower temperatures). As a result, a high concentration of the products of hydration is built up in the vicinity of the hydrating particles, and this retards the subsequent hydration and adversely affects the long-term strength. The presence of porous C-S-H in between the cement particles has been confirmed by backscattered electron imaging.[8.74]

In addition, the non-uniform distribution of the products of hydration *per se* adversely affects the strength because the gel/space ratio in the interstices is lower than would be otherwise the case for an equal degree of hydration: the local weaker areas lower the strength of the hydrated cement paste as a whole.

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In connection with the influence of temperature during the early life of concrete on the overall structure of the hydrated cement paste, it is useful to recall that a low early gain of strength has a beneficial effect on strength also when the hydration is slowed down by the use of retarders. Water-reducing and set-retarding admixtures were found to be beneficial in compensating for the reduction in the long-term strength of admixture-free concrete placed at a high temperature.[8.24] It should be realized, however, that their effect arises from water reduction and therefore a lower water/cement ratio.[8.14] Moreover, the rate of loss of slump is higher when these admixtures are used.[8.14]

Figure 8.1 shows Price's[8.11] data on the effect of the temperature during the first two hours after mixing on the development of strength of concrete with a water/cement ratio of 0.53. The range of temperatures investigated was 4 to 46 °C (40 to 115 °F) and, beyond the age of two hours, all specimens were cured at 21 °C (70 °F). The spe-

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cimens were sealed so as to prevent movement of moisture. Tests on cylinders moist-cured during the first 24 hours at 2 °C (36 °F) and at 18 °C (64 °F), and thereafter at 18 °C (64 °F) have shown that, at 28 days, the former are 10 per cent stronger than the latter.[8.80]

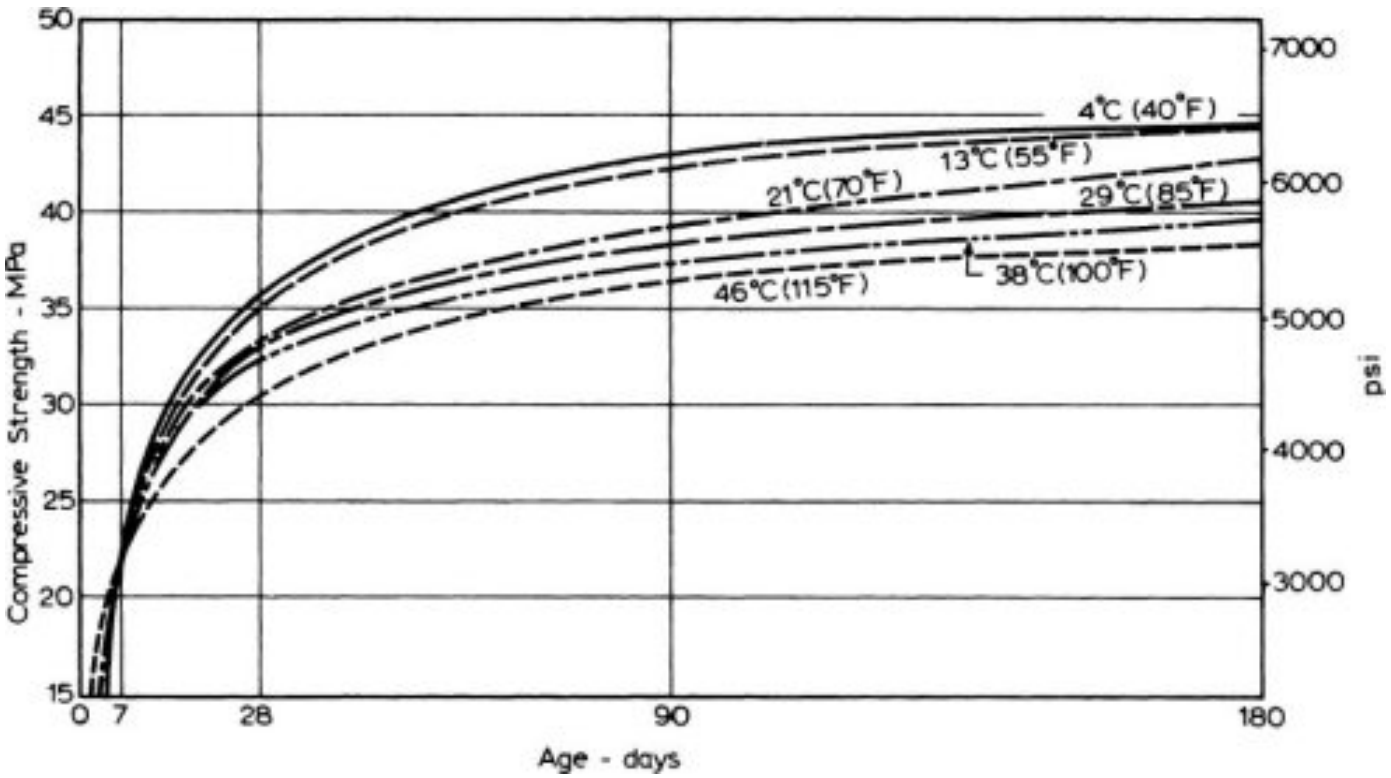


Fig. 8.1. Effect of temperature during the first two hours after casting on the development of strength (all specimens sealed and after the age of 2 hours cured at 21 °C (70 °F))[8.11]

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Some other test data are given below but direct comparisons are difficult because varying combinations of temperature and time were used in the various studies. An increase in the 24-hour strength of concrete, coupled with a decrease in the 28-day strength, in consequence of a higher temperature during the first 4 hours, was observed by Petscharnig[8.26] (see Fig. 8.2). He found the effect to be more pronounced with a more rapid-hardening cement and with a higher cement content.

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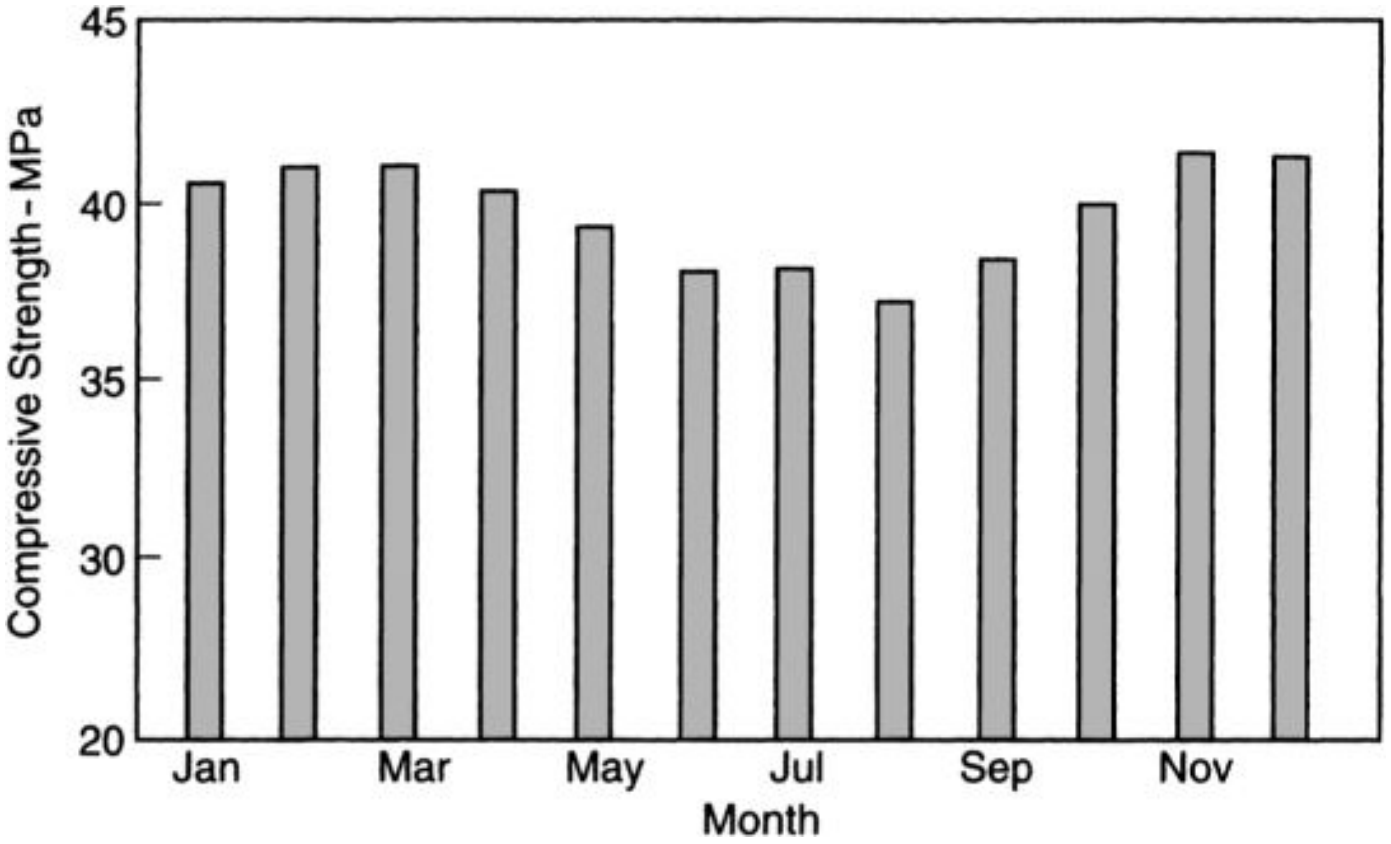


Fig. 8.2. Influence of initial temperature on the average monthly compressive strength of concrete cured at a constant temperature from the age of four hours onwards: the temperature can be inferred from the time of the year when the concrete specimens were made in the open in Austria (based on ref. 8.26)

Temperature of 38 °C (100 °F) during the first 24 hours was reported to result in a loss of strength of concrete at 28 days of 9 to 12 per cent, compared with the same concrete cured at 23 °C

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(70 °F) throughout;[8.25] the concrete had a 28-day standard cylinder strength of 28 MPa (4000 psi).

A review of the effect of a higher temperature during the first few days on the strength of test cylinders,[8.58] as compared with cylinders cured in a standard manner, has shown a significant reduction in the recorded 28-day strength: 1 day at 38 °C (100 °F) results in a reduction of about 10 per cent, and 3 days at 38 °C in a reduction of about 22 per cent.

Some field tests have confirmed the influence of temperature at the time of placing on strength: typically, for an increase of 5 °C (9 °F) there is a decrease in strength of 1.9 MPa (270 psi).[8.85]

The influence of the temperature in the early life of cement paste (from the age of 24 hours onwards) on the structure of the hydrated cement paste was demonstrated by Goto and Roy[8.113] who found that curing at 60 °C (140 °F) results in a much higher volume of pores larger than 150 nm in diameter, compared with curing at 27 °C

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(81 °F). The total porosity varied in the opposite direction, but it is the larger pores that control permeability, which is of great importance with regard to durability.

The influence of curing temperature on strength of concrete (tested after cooling) at 1 and 28 days is shown in Fig. 8.3.[8.77] However, the temperature at the time of testing also appears to be a factor, at least in the case of neat (ordinary Portland) cement paste compacts with a water/cement ratio of 0.14.[8.81] The temperature was kept constant from the initiation of hydration. When tested (at 64 and 128 days) at the curing temperature, the specimens had a lower strength at higher temperatures (Fig. 8.4); but, if cooled to 20 °C (68 °F) over a period of two hours prior to testing, only temperatures above 65 °C (150 °F) had a deleterious effect (Fig. 8.5).

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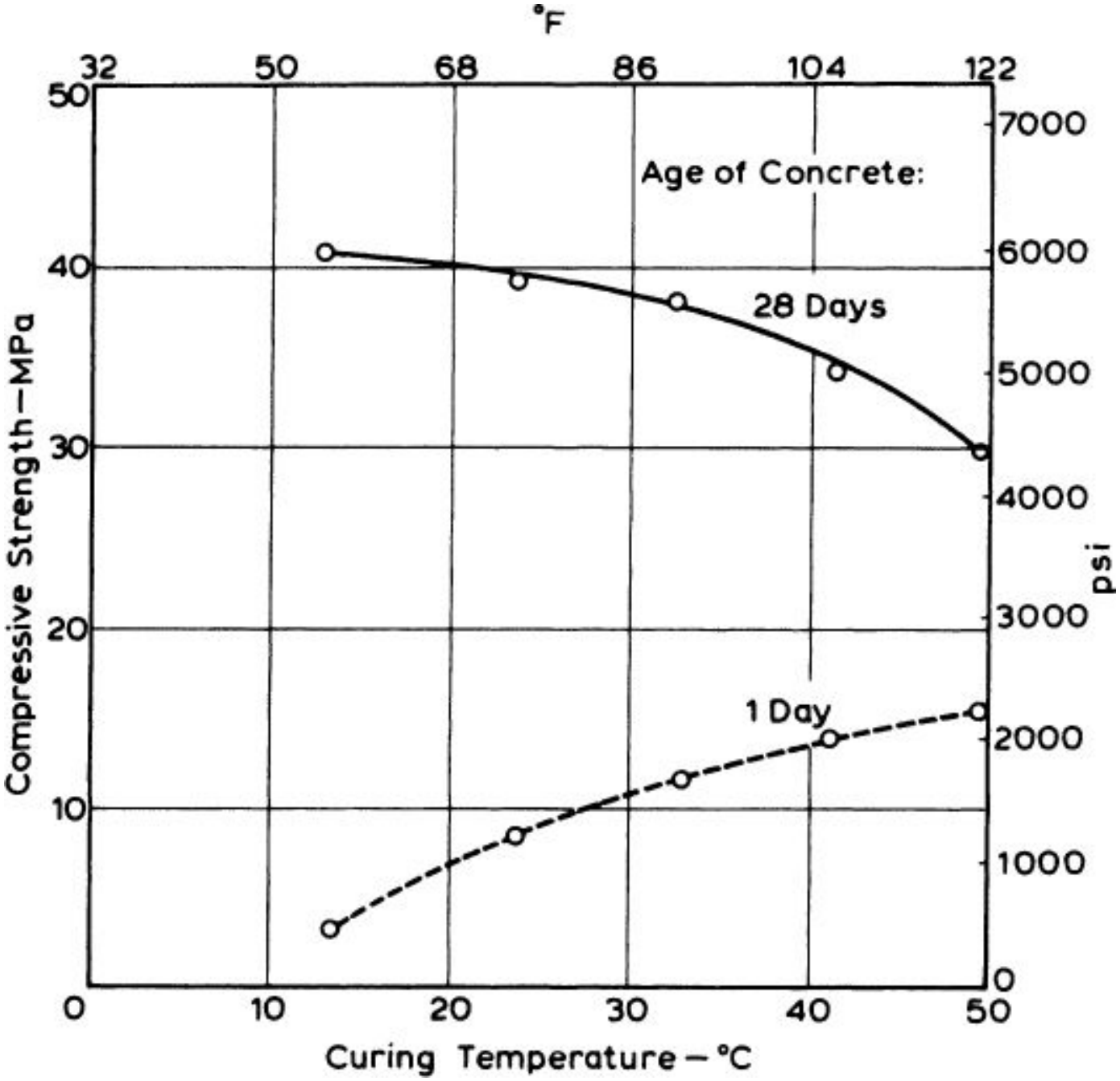


Fig. 8.3. Influence of curing temperature on compressive strength at 1 and 28 days (specimens tested after cooling to 23 °C (73 °F) over a period of two hours)[8.77]

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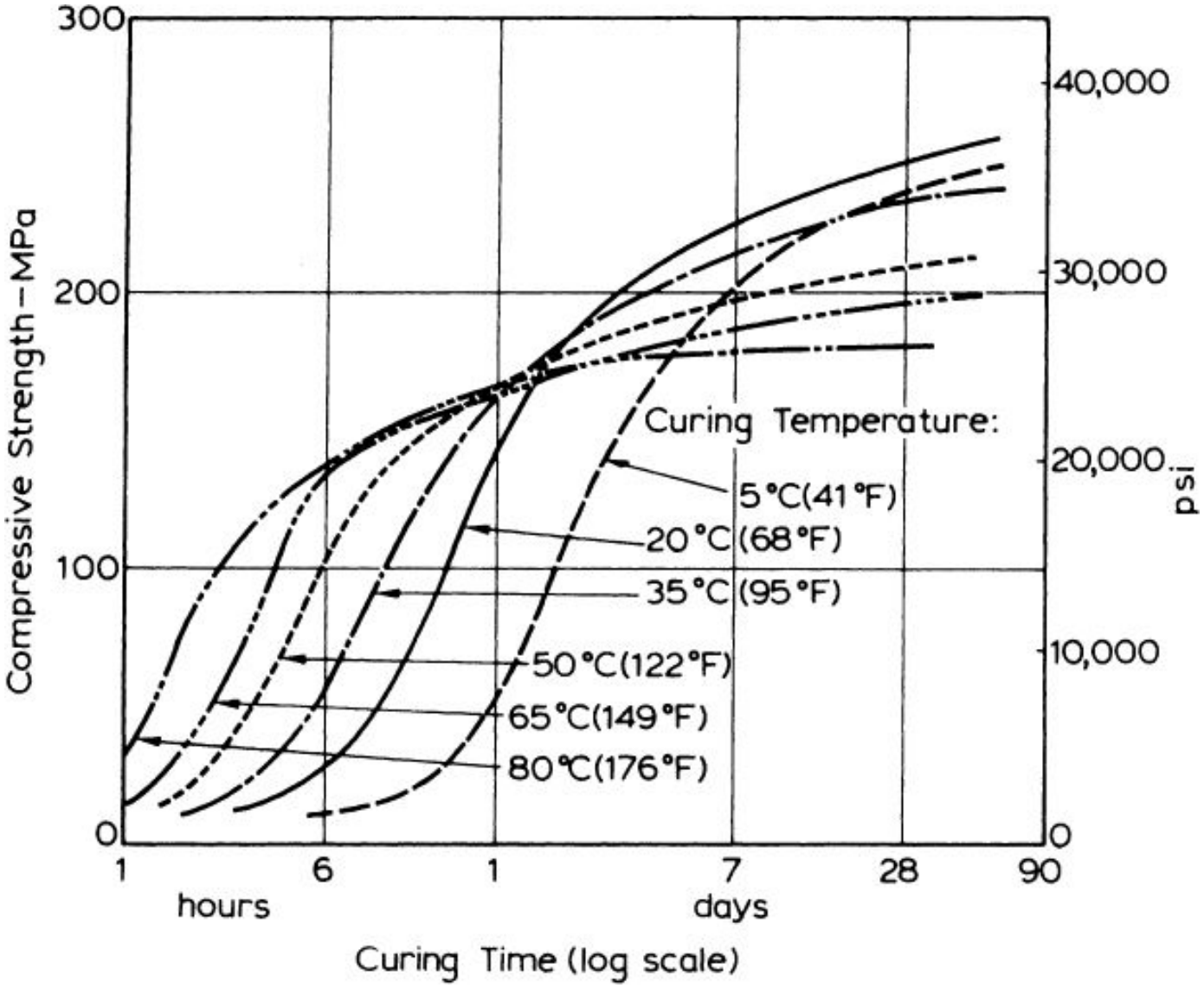
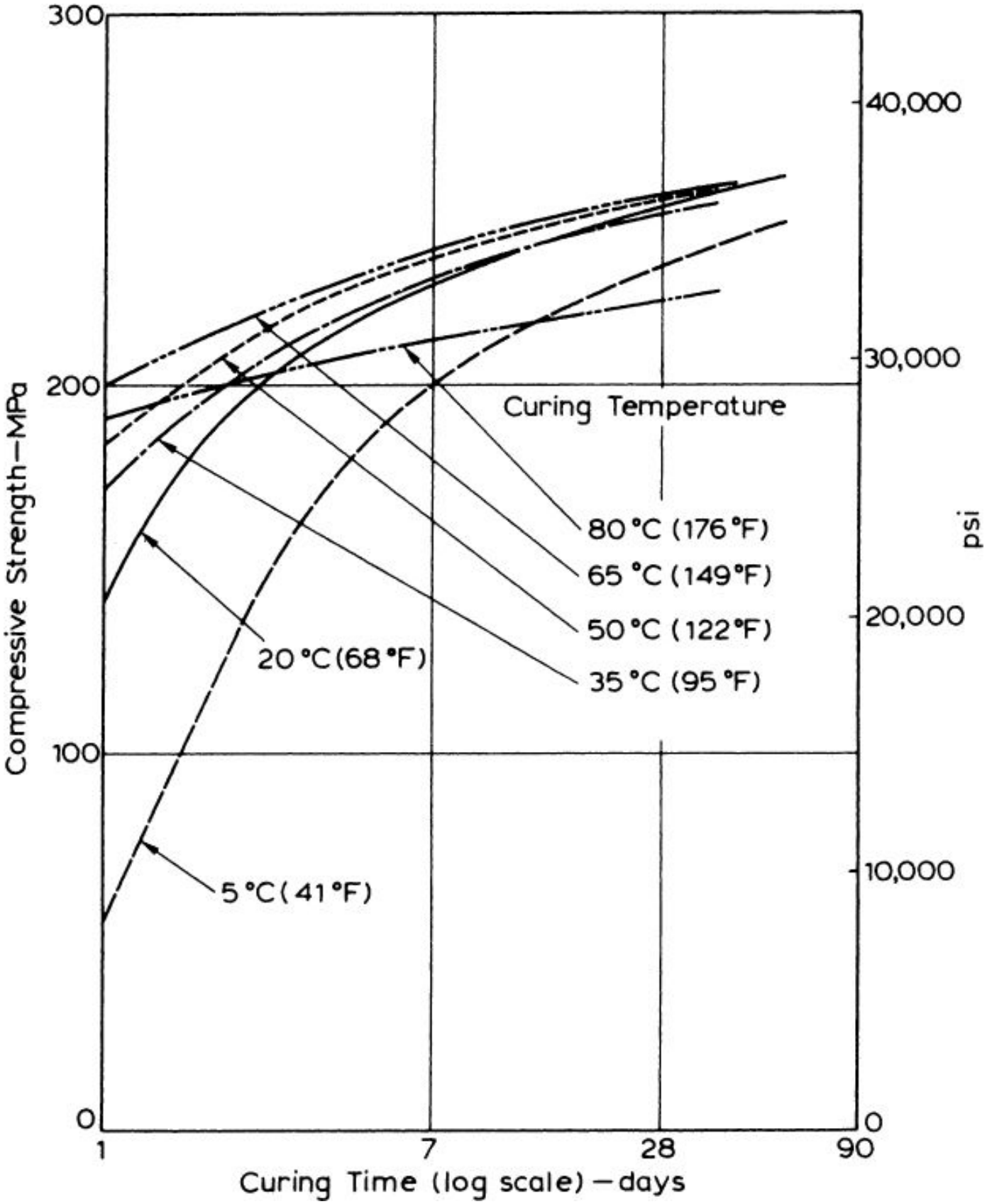


Fig. 8.4. Relation between compressive strength and curing time of neat cement paste compacts at different curing temperatures. The temperature of the specimens was kept constant up to and including the period of testing[8.81]

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Fig. 8.5. Relation between compressive strength and curing time of neat cement paste compacts at different curing temperatures. The temperature of the specimens was moderated to 20 °C at a constant rate over a two-hour period prior to testing (water/cement ratio = 0.14; Type I cement)[8.81]

Tests have also been made on concretes stored in water at different temperatures for a period of 28 days, and thereafter at 23 °C (73 °F).[8.70] As in Price's tests, a higher temperature was found to result in a higher strength during the first few days after casting but, beyond the age of one to four weeks, the situation changed radically. The specimens cured at temperatures between 4 and 23 °C (40 and 73 °F) up to the age of 28 days all showed a higher strength than those cured at 32 to 49 °C (90 to 120 °F). Among the latter, retrogression was greater the higher the temperature but, in the lower range of temperatures, there appeared to be an optimum temperature that yielded the highest strength. It is interesting to note that

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even concrete cast at 4 °C (40 °F) and stored at as low a temperature as –4 °C (25 °F) for four weeks and then at 23 °C (73 °F) is from the age of 3 months onwards stronger than similar concrete stored continuously at 23 °C (73 °F). Figure 8.6 shows typical curves for concrete containing 307 kg of ordinary Portland cement per cubic metre of concrete (517 lb/yd^3) with 4.5 per cent of entrained air. Similar behaviour has been observed when rapid-hardening Portland and modified cement are used.

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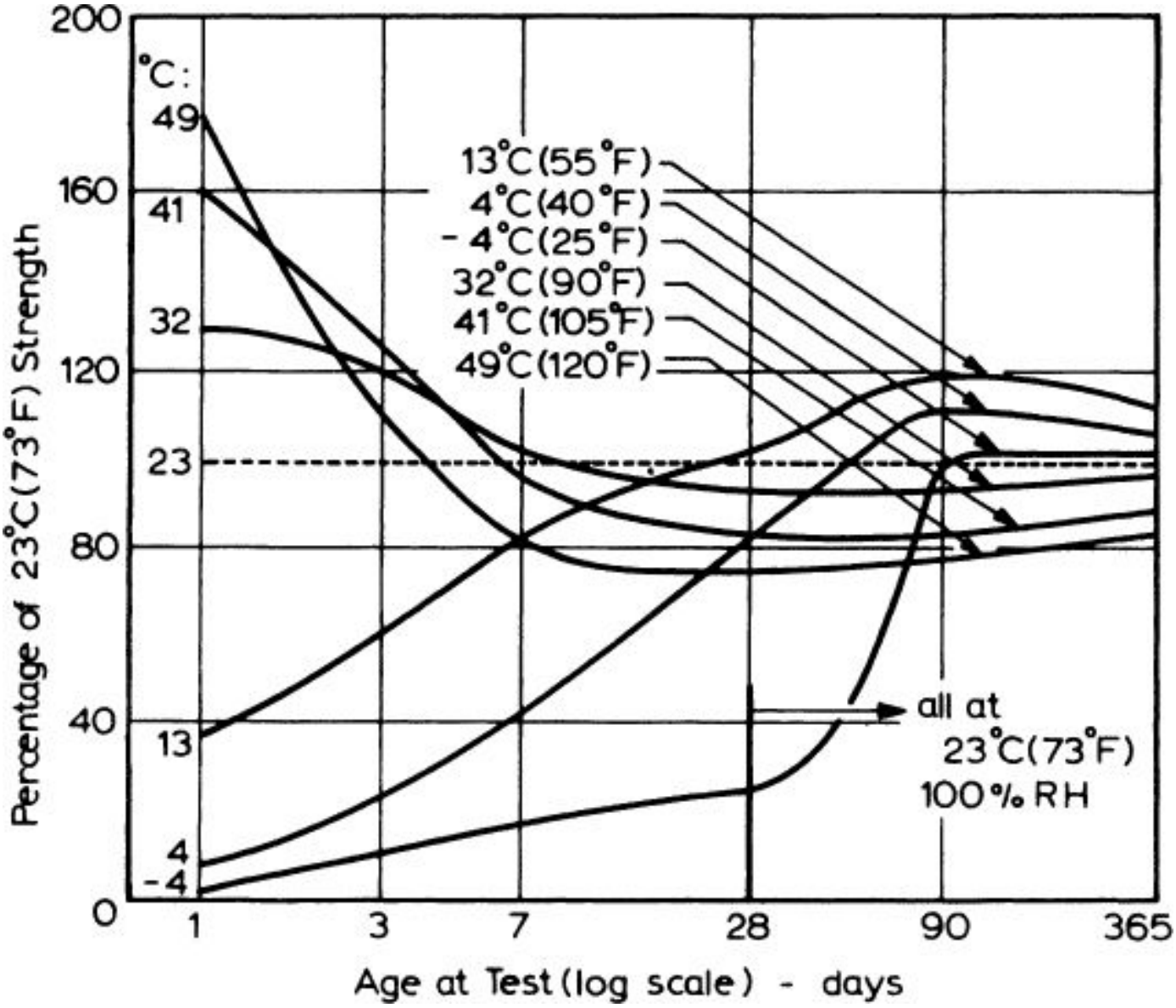


Fig. 8.6. Effect of temperature during the first 28 days on the strength of concrete (water/cement ratio = 0.41; air content = 4.5 per cent; ordinary Portland cement)[8.70]

In concrete members with a high cement content, as is the case with high performance concrete, there is a considerable temperature rise

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even in ordinary structural elements such as beams and columns. The 7-day strength is higher the greater the temperature rise; for example, when the temperature was 20 °C (68 °F) the strength was 96 MPa, but when the maximum temperature was 75 °C (167 °F) the strength was 115 MPa. However, at 28 days, there was a reversal in the strength values with the low temperature leading to a strength of 122 MPa while the high temperature led to a reduced strength of 112 MPa. Maximum temperatures between 45 and 65 °C led to a very slight increase in strength between the ages of 7 and 28 days.[8.57]

With respect to the strength of concrete cured at very low temperatures, Aïtcin *et al.*[8.23] found that, provided concrete with a water/cement ratio between 0.45 and 0.55 was cast and maintained for 9 hours at a temperature not lower than 4 °C (39 °F), subsequent storage in sea water at 0 °C (32 °F) led to an increase in strength. The increase was at first very slow but at the age of 4 days the specimens immersed in sea water

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reached about one-half of the strength of standard-cured specimens. The difference between the strengths for the two storage conditions gradually decreased, after 2 months becoming about 10 MPa (1500 psi); this value persisted for at least a year. Concrete with a lower water/cement ratio performed better than concretes with higher water/cement ratios.[8.18,8.23]

Klieger's tests[8.70] indicate that there is an optimum temperature during the early life of concrete that will lead to the highest strength at a desired age. For laboratory-made concrete, using ordinary or modified Portland cement, the optimum temperature is approximately 13 °C (55 °F); for rapid-hardening Portland cement it is about 4 °C (40 °F). It must not be forgotten, however, that beyond the initial period of setting and hardening the influence of temperature (within limits) accords with the maturity rule: a higher temperature accelerates the development of strength.

The tests described so far were all made in the laboratory or under known conditions, but the be-

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haviour on site in a hot climate may not be the same. Here, there are some additional factors acting: ambient humidity, direct radiation of the sun, wind velocity, and method of curing. It should be remembered also that the quality of concrete depends on *its* temperature and not on that of the surrounding atmosphere, so that the size of the member also enters the picture because it affects the rise in temperature caused by the hydration of cement. Likewise, curing by flooding in windy weather results in a loss of heat due to evaporation so that the temperature of concrete is lower than when a sealing compound is used. These factors are discussed later in the present chapter.

Steam curing at atmospheric pressure

Because an increase in the curing temperature of concrete increases its rate of development of strength, the gain of strength can be speeded up by curing concrete in steam. When steam is at atmospheric pressure, i.e. the temperature is below 100 °C (212 °F), the process can be regarded as a

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special case of moist curing in which the vapour-saturated atmosphere ensures a supply of water. In addition, condensation of the steam releases latent heat. High-pressure steam curing (autoclaving) is an entirely different operation and is considered in the next section.

The primary object of steam curing is to obtain a sufficiently high early strength so that the concrete products may be handled soon after casting: the moulds can be removed, or the prestressing bed vacated, earlier than would be the case with ordinary moist curing, and less curing storage space is required; all these mean an economic advantage. For many applications, the long-term strength of concrete is of lesser importance.

Because of the nature of the operations involved in steam curing, the process is used mainly with precast products. Low-pressure steam curing is normally applied in special chambers or in tunnels through which the concrete members are transported on a conveyor belt. Al-

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ternatively, portable boxes or plastic covers can be placed over precast members, steam being supplied through flexible pipes.

Due to the influence of temperature during the early stages of hardening on the later strength, a compromise between the temperatures giving a high-early and a high-late strength has to be made. Figure 8.7 shows typical values of strength of concrete made with modified (Type II) cement and a water/cement ratio of 0.55; steam curing was applied immediately after casting. Long-term retrogression of strength was observed.

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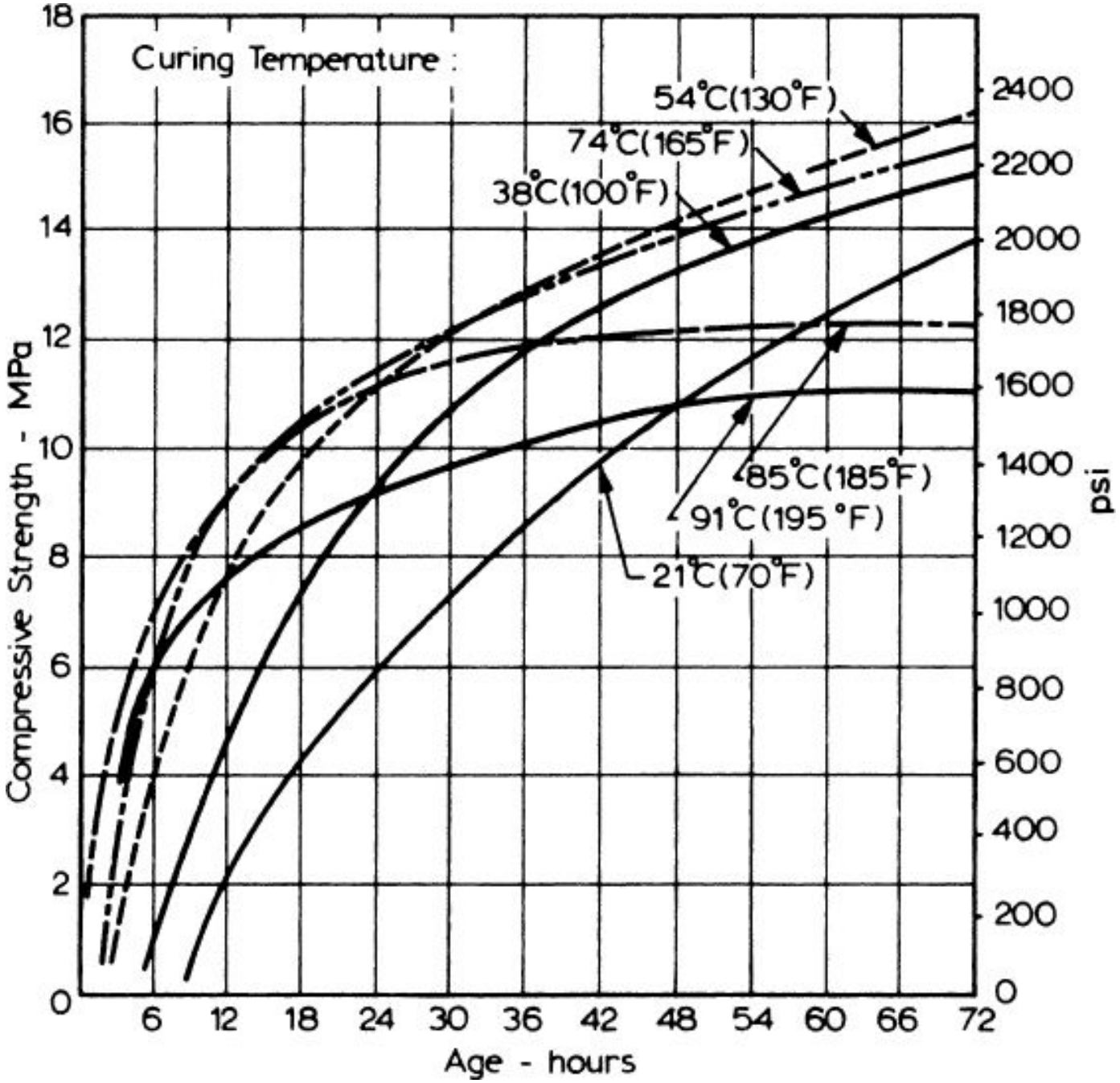


Fig. 8.7. Strength of concrete cured in steam at different temperatures (water/cement ratio = 0.55; steam curing applied immediately after casting)[8.71]

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A probable, possibly partial, explanation of the reduction in the long-term strength of steam-cured concrete lies in the presence of very fine cracks caused by the expansion of air bubbles in the cement paste; the thermal expansion of the air is at least two orders of magnitude greater than that of the surrounding solid material. The expansion of air bubbles is restrained so that the air is put under pressure and, to balance this pressure, tensile stresses are induced in the surrounding cement paste. These stresses may induce very fine cracks. Strictly speaking, therefore, we are dealing not with a long-term loss of strength but with a loss at all ages.[8.82] However, up to the age of 28 days, this loss is masked by the beneficial effect on strength of a higher temperature during curing.

The role of the expanding air bubbles, as well as of water, is indirectly demonstrated by the very high coefficient of thermal expansion of fresh concrete (30×10^{-6}) compared with the coefficient after 4 hours (11.5×10^{-6}) reported by Mamillan.[8.37]

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The disruptive effects of the expansion of air bubbles can be reduced by a prolonged delay period prior to steam curing (during which the tensile strength of concrete increases) or by a lower rate of temperature rise (as the increase in the air pressure is matched by an increase in the strength of the surrounding cement paste). Alternatively, heating in closed formwork or in pressure chambers can be used.[8.82] With short-term curing periods (2 to 5 hours) and moderate temperatures, there is probably little real retrogression of strength, and the apparent low strength at later ages is due to the absence of prolonged wet curing.[8.83]

Because the adverse effect of steam curing on the long-term strength of concrete arises from the changes in porosity and pore size in hydrated cement paste, it can be expected that steam curing affects the durability of concrete; this is discussed on p. 485.

To minimize the long-term retrogression of strength, two aspects of a steam-curing cycle

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should be controlled: the delay in the commencement of heating and the rate of temperature rise.

Because it is the temperature at the time of setting that has the greatest influence on the strength at later ages, a delay in the application of steam curing is advantageous. Some indication of the influence of the delay in heating on strength can be obtained from Fig. 8.8 plotted by Saul[8.72] from the data of Shideler and Chamberlin.[8.73] The concrete used was made with Type II cement, and had a water/cement ratio of 0.6. The solid line shows the gain in strength of moist-cured concrete at room temperature plotted against maturity. The dotted lines refer to different curing temperatures between 38 and 85 °C (100 and 185 °F), and the figure against each point denotes the delay in hours before the higher curing temperature was suddenly applied.

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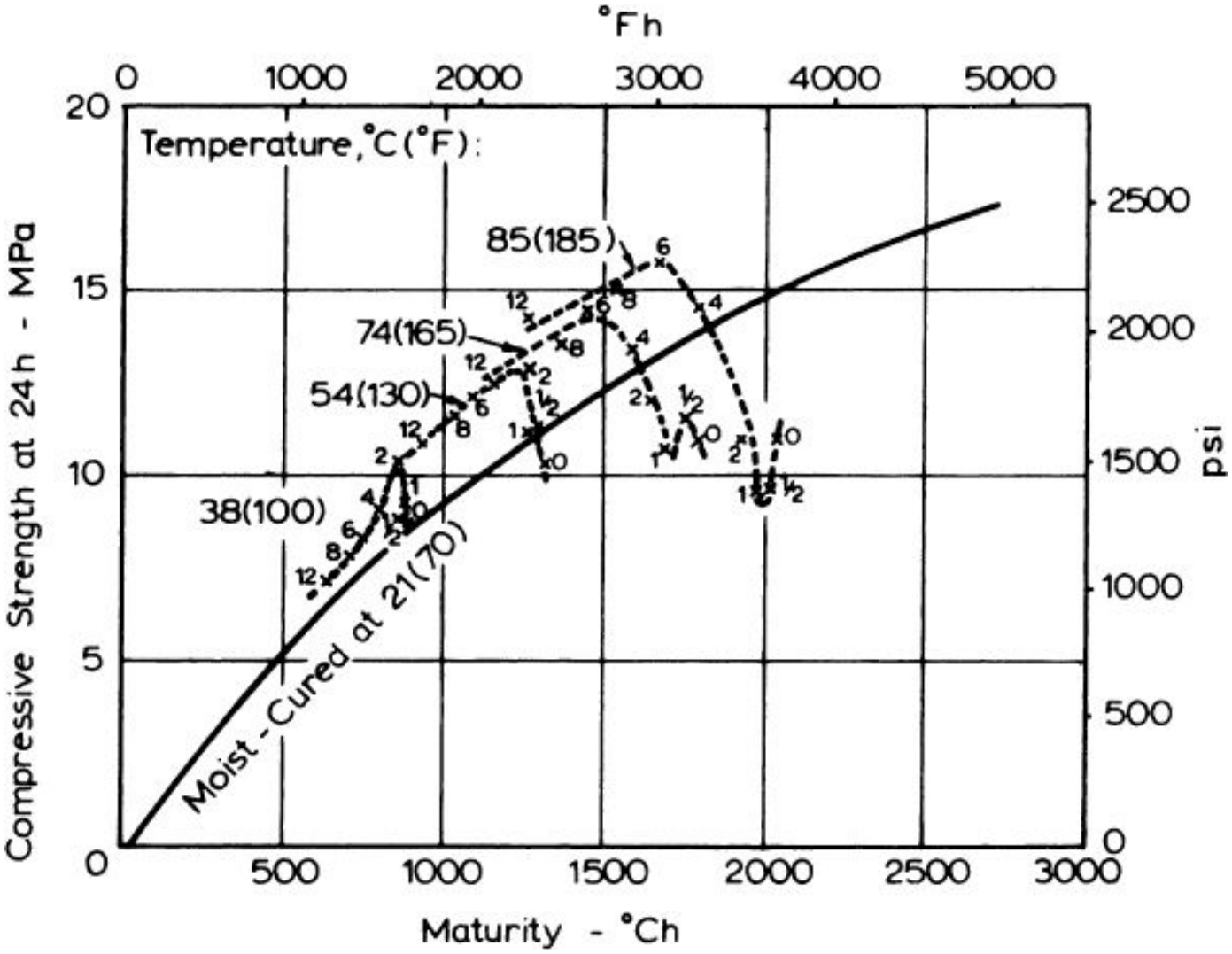


Fig. 8.8. Effect of delay in steam curing on the early gain of strength with maturity.[8.72] Small figures indicate the delay in hours before curing at the temperature indicated

From Fig. 8.8 it can be seen, for each curing temperature, there is a part of the curve showing a normal rate of gain in strength with maturity. In other words, after a sufficient delay, rapid heat-

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ing has no adverse effect. This delay is approximately 2, 3, 5, and 6 hours, respectively, for 38, 54, 74, and 85 °C (100, 130, 165, and 185 °F). If, however, concrete is exposed to the higher temperature with a smaller delay, the strength is adversely affected, as shown by the right-hand portion of each dotted curve; this effect is more serious the higher the curing temperature. Without a delay period, the loss in the 28-day strength of concrete with a water/cement ratio of 0.50, steam cured at 75 °C (167 °F), can be as much as 40 per cent.[8.37]

An additional argument supporting the need for the delay period is that it allows gypsum to react with C_3A. At higher temperatures, the solubility of gypsum is decreased so that some of it might not react with C_3A and do so only later, causing an expansive reaction of the type known as sulfate attack (see p. 508).[8.31] This view has not been confirmed.

Figure 8.8 shows also that, within a few hours of casting, the rate of gain in strength is higher

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than would be expected from the maturity calculations. This confirms the earlier observation that the *age* at which a higher temperature is applied is a factor in the maturity rule.

The desirable length of the delay period (when the ambient temperature should match that of the concrete) depends on the size and shape of the concrete elements being steam cured, on the water content of the concrete, and on the type of cement: when the rate of hardening is slow, the delay should be longer. However, if a large surface area is exposed, fog spray may be required to prevent plastic shrinkage cracking. Guidance on the choice of the delay period is given in ACI 517.2R-87 (Revised 1992).[8.27]

The subsequent rate of rise in temperature also has to be controlled, depending on the nature of the concrete units, so as to prevent the development of steep temperature gradients in the concrete. A trial-and-error approach is necessary. ACI 517.2R-87 (Revised 1992)[8.27] recommends rates ranging between 33 °C (60 °F) per hour for

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small units and 11 °C (20 °F) per hour for large units. The rate of temperature rise has little effect on the long-term strength, but the maximum temperature is a factor: a temperature of 70 to 80 °C (160 to 180 °F) results in a reduction in the 28-day strength of about 5 per cent.[8.27]

This effect has to be balanced, in economic terms, by the fact that a lower maximum temperature requires a longer steam-curing period. It should be noted, however, that the supply of heat need not continue once the temperature of the concrete has stabilized at the maximum value; this time interval is referred to as ‘soaking’.

The period of steam curing at the maximum temperature is followed by cooling. This may be rapid in the case of small units, but in large units rapid cooling may lead to surface cracking. Supplementary wet curing may be beneficial in preventing rapid drying out and improving the subsequent increase in strength.[8.83] Concrete with a lower water/cement ratio responds to steam cur-

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ing much better than a mix with a high water/cement ratio.

In summary, a curing cycle consists of a delay period, known also as *preset period*, a temperature-rise period, a steaming period (which includes soaking) at the maximum temperature, and a cooling period, possibly followed by wet curing. Practical curing cycles are chosen as a compromise between the early and late strength requirements but are governed also by the time available (e.g. length of work shifts). Economic considerations determine whether the curing cycle should be suited to a given concrete mix or, alternatively, whether the mix ought to be chosen so as to fit a convenient cycle of steam curing. Whereas details of an optimum curing cycle depend on the type of concrete product treated, a typical satisfactory cycle would consist of the following:[8.27] a delay period of 2 to 5 hours, heating at the rate of 22 to 44 °C per hour (40 to 80 °F per hour) up to a maximum temperature of 50 to 82 °C (122 to 180 °F), then storage at

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maximum temperature, and finally a cooling period, the total cycle (exclusive of the delay period) occupying preferably not more than 18 hours.

For concrete which is to be exposed to aggressive conditions, CIRIA Report C660 published in 2007 gives guidance on the maximum temperature and on the rate of temperature rise.

Lightweight aggregate concrete can be heated up to between 82 and 88 °C (180 and 190 °F), but the optimum cycle is no different from that for concrete made with normal weight aggregate.[8.79]

Steam curing has been used successfully with different types of Portland cement, as well as with blended cements, but must never be used with high-alumina cement because of the adverse effect of hot, wet conditions on the strength of that cement. Steam curing of concrete made with fly ash accelerates the pozzolanic reaction with $Ca(OH)_2$ but only above a temperature of 88 °C (190 °F). A similar situation obtains with ground granulated blastfurnace slag in the mix above 60 °C (140 °F). An increased fineness of the slag

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(above 600 m^2/kg) is beneficial with respect to the effects of steam curing on strength.[8.28] The slag also leads to a reduction in the average pore size in the steam-cured cement paste.[8.28]

High-pressure steam curing (autoclaving)

This process is quite different from curing in steam at atmospheric pressure, both in the method of execution and in the nature of the resulting concrete.

Because pressures above atmospheric are involved, the curing chamber must be of the pressure-vessel type with a supply of wet steam; superheated steam must not be allowed to come into contact with the concrete because it would cause drying of concrete. Such a vessel is known as an autoclave, and high-pressure steam curing is also referred to as autoclaving.

High-pressure steam curing was first employed in the manufacture of sand–lime brick and

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of lightweight cellular concrete, and is still extensively used for that purpose. In the field of concrete, high-pressure steam curing is usually applied to precast products, generally small, but also to bridge truss members (made both of normal weight and lightweight concrete) when any of the following characteristics are desired:

(a) high early strength: with high-pressure steam curing, the 28-day strength on normal curing can be reached in about 24 hours; strengths of 80 to 100 MPa (12 000 to 15 000 psi) have been reported;[8.29]

(b) high durability: high-pressure steam curing improves the resistance of concrete to sulfates and to other forms of chemical attack, also to freezing and thawing, and reduces efflorescence; and

(c) reduced drying shrinkage and moisture movement.

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The optimum curing temperature has been found experimentally to be about 177 °C (350 °F)[8.75] which corresponds to a steam pressure of 0.8 MPa (120 psi) above atmospheric pressure.

High-pressure steam curing is most effective when finely ground silica is added to the cement, owing to the chemical reactions between the silica and $Ca(OH)_2$ released on hydration of C_3S (see Fig. 8.9). Cements rich in C_3S have a greater capacity for developing high strength when cured at high pressure than those with a high C_2S content, although, for short periods of high-pressure steam curing, cements with a moderately low C_3S/C_2S ratio give good results.[8.76] The high temperature during curing affects also the reactions of hydration of the cement itself. For instance, some of the C_3S may hydrate to C_3SH_x.

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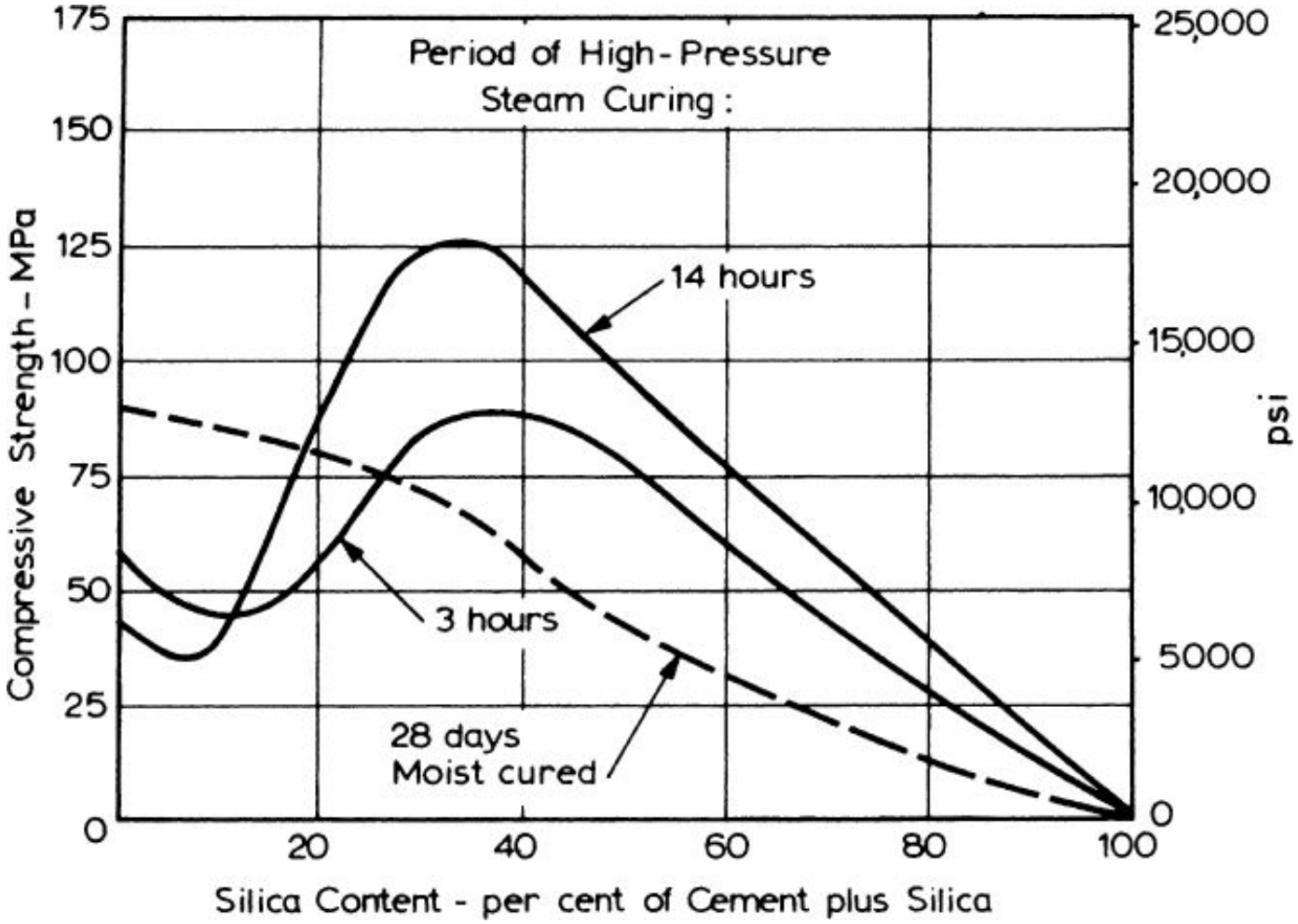


Fig. 8.9. Influence of pulverized silica content on the strength of high-pressure steam-cured concrete (age at commencement of curing, 24 hours; curing temperature, 177 °C (350 °F))[8.75]

The fineness of the silica should be at least equal to that of the cement; a higher fineness, 600 m^2/kg, was found[8.29] to lead to an increase in strength of 7 to 17 per cent compared with

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silica having a fineness of 200 m^2/kg. Cement and silica must be intimately mixed before they are fed into the mixer. The optimum amount of silica depends on the mix proportions but is generally between 0.4 and 0.7 of the mass of cement.

It is essential that the rate of heating during high-pressure steam curing is not too high, as interference with the setting and hardening processes may occur in a manner similar to that discussed in connection with steam curing at atmospheric pressure. A typical steaming cycle consists of a gradual increase to the maximum temperature of 182 °C (360 °F) (which corresponds to a pressure of 1 MPa (145 psi)) over a period of 3 hours. This is followed by 5 to 8 hours at this temperature, and then a release of pressure in about 20 to 30 minutes. A rapid release accelerates the drying of the concrete so that shrinkage in situ will be reduced. At each temperature there is an optimum period of curing (see Fig. 8.10).[8.84]

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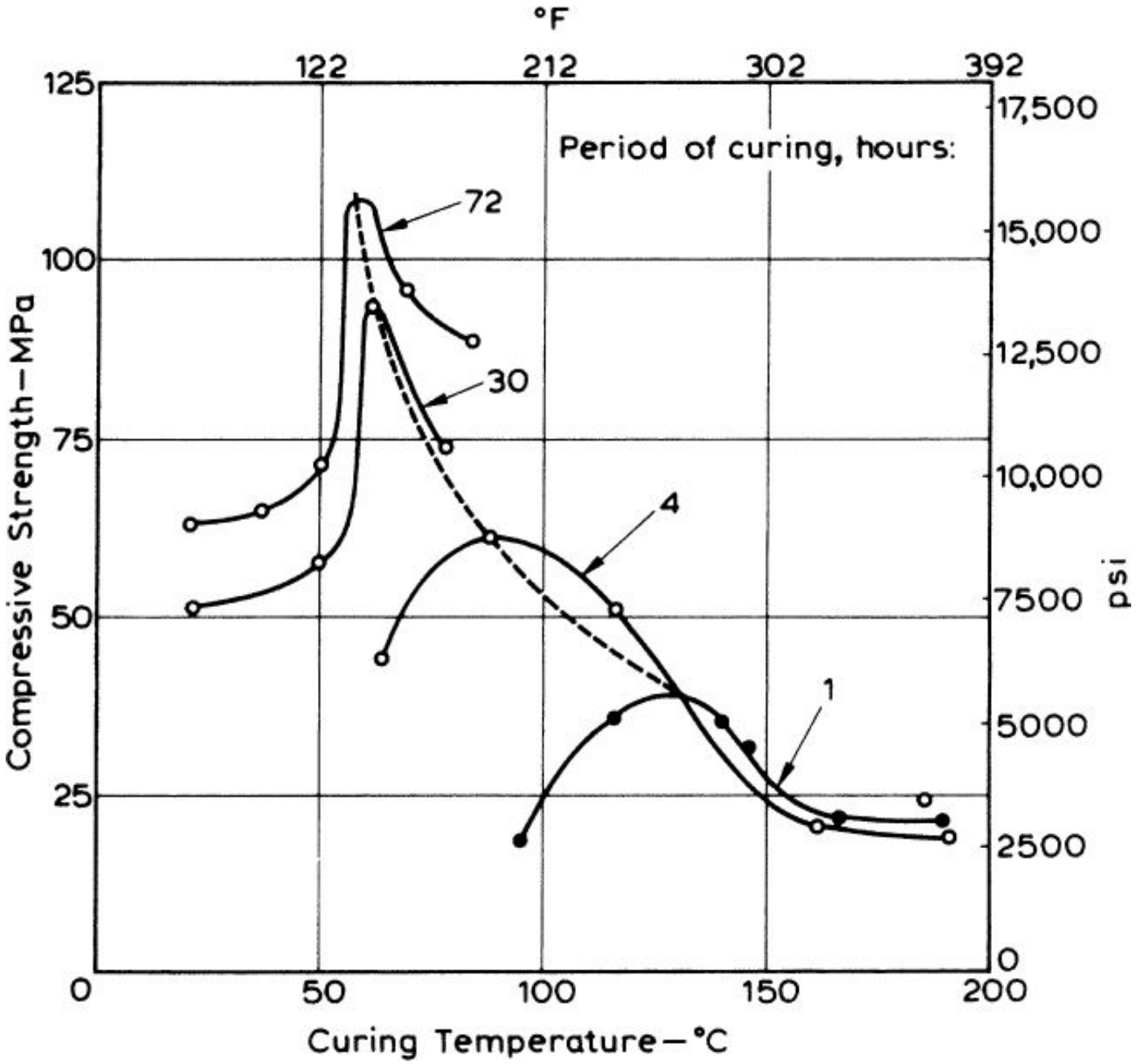


Fig. 8.10. Strength development of concrete at different curing temperatures for various periods of curing[8.84]

It is worth emphasizing that a longer period of curing at a lower temperature leads to a higher optimum strength than when high temperature is

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applied for a shorter time. For any one period of curing, there is a temperature which leads to an optimum strength. Also, for a given set of materials, it is possible to draw a line joining the points of optimum strength at various curing periods and the curing temperature;[8.84] this is shown in Fig. 8.10.

In practice, the details of the steaming cycle depend on the plant used and also on the size of the concrete members being cured. The length of the period of normal curing preceding placing in the autoclave does not affect the quality of the steam-cured concrete, and the choice of a suitable period is governed by the stiffness of the mix, which must be strong enough to withstand handling. In the case of lightweight concretes, the details of the steaming cycle have to be determined experimentally so suit the materials used.

Steam curing should be applied to concretes made with Portland cement only: high-alumina and supersulfated cements would be adversely affected by the high temperature.

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Within the Portland group, the type of cement affects the strength, but not necessarily in the same way as at normal temperatures; no systematic studies have, however, been made. It is known, though, that ground granulated blastfurnace slag may cause trouble if it has a high sulfur content. High-pressure steam curing accelerates the hardening of concrete containing calcium chloride, but the relative increase in stength is less than when no calcium chloride is used.

High-pressure steam curing produces a hydrated cement paste of low specific surface, about 7000 m^2/kg. Because the specific surface of high-pressure steam-cured paste is thus only about $\frac{1}{20}$ of that of cement cured at ordinary temperature, it appears that no more than 5 per cent of the high-pressure cured paste can be classified as gel. This means that the products of hydration are coarse and largely microcrystalline. For this reason, high-pressure steam-cured concrete has a considerably reduced shrinkage, about $\frac{1}{6}$ to $\frac{1}{3}$ of

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that of concrete cured at normal temperatures. When silica is added to the mix, shrinkage is higher, but still only about one-half of the shrinkage of normally cured concrete. By contrast, because low-pressure steam curing does not produce a micro-crystalline hydrated cement paste, no reduction in shrinkage is obtained. Creep is also significantly reduced by high-pressure steam curing.

The products of hydration of cement subjected to high-pressure steam curing, as well as those of the secondary lime–silica reactions, are stable, and there is no retrogression of strength. At the age of one year, the strength of normally cured concrete is approximately the same as that of high-pressure steam-cured concrete of similar mix proportions. The water/cement ratio affects the strength of high-pressure steam-cured concrete in the usual manner, but the actual values of early strength differ, of course, from those for ordinary curing. The coefficient of thermal expansion and the modulus of elasticity of concrete

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seem unaffected by high-pressure steam curing.[8.75]

High-pressure steam curing improves the resistance of concrete to sulfate attack. This is due to several reasons, the main one being the formation of aluminates more stable in the presence of sulfates than those formed at lower temperatures. For this reason, the relative improvement in resistance to sulfate attack is greater in cements with a high C_3A content than in cements which are resistant to sulfates. Another important factor is the reduction in lime in the cement paste as a result of the lime–silica reaction. Further improvement in sulfate resistance is due to the increased strength and lower permeability of the steam-cured concrete, and also to the existence of hydrates in a well-crystallized form.

High-pressure steam curing reduces efflorescence as there is no lime left to be leached out.

High-pressure steam-cured concrete tends to be rather brittle. High-pressure steam curing may reduce the strength in bond with plain reinforce-

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ment but not with deformed bars. Good impact strength of high-pressure steam-cured concrete has been reported.[8.86] On the whole, high-pressure steam curing produces good quality, dense and durable concrete. It is whitish in appearance as distinct from the characteristic colour of normally-cured Portland cement concrete.

Other thermal curing methods

There exist several other methods of applying heat to concrete for the purpose of accelerating the gain of strength. They are all specialized and applicable only in certain cases. For this reason, no more than a brief mention will be given below.

The *hot-mix* method relies upon raising the temperature of fresh concrete to at least 32 °C (90 °F). The long-term strength is consequently reduced by 10 to 20 per cent compared with normally cured concrete, but formwork can be removed at the age of several hours. The rise in temperature is achieved either by heating the aggregate, and also the water, or by injection of

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steam into the mixer. In either case, care is required in controlling the total water content of the mix. Heated or insulated formwork is necessary.

There are several methods of *electrical curing*. In one, electric current passes through the fresh concrete between external electrodes. The current must be alternating as direct current would lead to hydrolysis of the cement paste. In another method, a large current at low voltage is passed through the reinforcement in the concrete member. In a third method, large electric blankets are used to heat the surface of slabs. Yet another method utilizes insulated resistance wires embedded in the concrete member; after curing, they are cut and left in the concrete.

Infrared-radiation curing is used in some countries.

Steel formwork can also be heated electrically or through circulating hot water or oil.

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The various specialized curing methods are discussed in ACI 517.2R-87[8.27] and in some other publications.[8.35,8.36,8.37]

Thermal properties of concrete

The thermal properties of concrete are of interest for a variety of reasons, examples of which are given below. Thermal conductivity and diffusivity are relevant to the development of temperature gradients, thermal strains, warping, and cracking in the very early life of concrete, and are also relevant to thermal insulation provided by concrete in service. Knowledge of thermal expansion of concrete is required in the design of expansion and contraction joints, in the provision of bridge support movement, both horizontally and vertically, and in the design of statically indeterminate structures subject to temperature variation. This knowledge is also required in the assessment of thermal gradients in concrete, and in the design of prestressed concrete members. Behaviour at high temperatures needs to be known

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in special applications and also in consideration of the effects of fire. Thermal effects in mass concrete are of especial interest and will be discussed in a later section.

Thermal conductivity

This measures the ability of the material to conduct heat and is defined as the ratio of the flux of heat to temperature gradient. Thermal conductivity is measured in joules per second per square metre of area of body when the temperature difference is 1 °C per metre of thickness of the body (Btu per hour per sq. ft when temperature difference is 1 °F per ft of thickness).

The conductivity of ordinary concrete depends on its composition and, when the concrete is saturated, the conductivity ranges generally between about 1.4 and 3.6 J/m^{2}s °C/m (0.8 to 2.1 Btu/ft^{2}h °F/ft).[8.10] Density does not appreciably affect the conductivity of ordinary concrete but, due to the low conductivity of air, the thermal conductivity of lightweight concrete varies with

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its density[8.87] (see Fig. 13.16). Typical values of conductivity are listed in Table 8.1. More extensive data have been reported by Scanlon and McDonald,[8.10] and also in ACI 207.1R.[8.53] From Table 8.1, it can be seen that the mineralogical character of the aggregate greatly affects the conductivity of the concrete made with it. In general terms, basalt and trachyte have a low conductivity, dolomite and limestone are in the middle range, and quartz exhibits the highest conductivity, which depends also on the direction of heat flow relative to the orientation of the crystals. In general, crystallinity of rock increases its conductivity.

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Table 8.1. Typical Values of Thermal Conductivity of Concrete (selected from ref. 8.10)

Type of aggregate	*Wet density of concrete*		*Conductivity*	
	kg/m³	*lb/ft³*	*J/m²s °C/m*	*Btu/ft²h °F/ft*
Quartzite	2440	152	3.5	2.0
Dolomite	2500	156	3.3	1.9
Limestone	2450	153	3.2	1.8
Sandstone	2400	150	2.9	1.7
Granite	2420	151	2.6	1.5
Basalt	2520	157	2.0	1.2
Barytes	3040	190	2.0	1.2
Expanded shale	1590	99	0.85	0.5

The degree of saturation of concrete is a major factor because the conductivity of air is lower than that of water. For instance, in the case of lightweight concrete, an increase in moisture content of 10 per cent increases conductivity by about one-half. On the other hand, the conductivity of water is less than half that of the hydrated cement paste, so that the lower the water content of the *mix* the higher the conductivity of the hardened concrete.

A frequent practical difficulty is to know the actual moisture content of the concrete. Loudon

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and Stacey[8.97] assumed to be typical the values of moisture content in per cent by volume shown at the top of Table 8.2 and, on that basis, recommended the use of the values of conductivity given in Table 8.2.

Table 8.2. Values of Thermal Conductivity Recommended by Loudon and Stacey[8.97]

Moisture content by volume per cent		Conductivity, J/m²s °C/m (Btu/ft²h °F/ft)															
		For concrete protected from weather								For concrete exposed to weather							
		5		5		5		2.5		8		8		8		5	
Unit mass kg/m³	Unit mass lb/ft³	Aerated concrete		Lightweight concrete with expanded slag		Lightweight concrete with expanded clay or sintered fly ash		Normal weight aggregate concrete		Aerated concrete		Lightweight concrete with expanded slag		Lightweight concrete with expanded clay or sintered fly ash		Normal weight aggregate concrete	
320	20	0.109	(0.063)	0.087	(0.050)	0.130	(0.075)			0.123	(0.071)	0.100	(0.058)	0.145	(0.084)		
480	30	0.145	(0.084)	0.116	(0.067)	0.173	(0.100)			0.166	(0.096)	0.130	(0.075)	0.187	(0.108)		
640	40	0.203	(0.117)	0.159	(0.092)	0.230	(0.133)			0.223	(0.129)	0.173	(0.100)	0.260	(0.150)		
800	50	0.260	(0.150)	0.203	(0.117)	0.303	(0.175)			0.273	(0.158)	0.230	(0.133)	0.332	(0.192)		
960	60	0.315	(0.182)	0.260	(0.150)	0.376	(0.217)			0.360	(0.208)	0.289	(0.167)	0.433	(0.250)		
1120	70	0.389	(0.225)	0.315	(0.182)	0.462	(0.267)			0.433	(0.250)	0.360	(0.208)	0.519	(0.300)		
1280	80	0.476	(0.275)	0.389	(0.225)	0.562	(0.325)			0.533	(0.308)	0.433	(0.250)	0.635	(0.367)		
1440	90			0.462	(0.267)	0.678	(0.392)										
1600	100			0.549	(0.317)	0.794	(0.459)	0.706	(0.408)							0.808	(0.467)
1760	110			0.649	(0.375)	0.952	(0.550)	0.838	(0.484)							0.952	(0.550)
1920	120							1.056	(0.610)							1.194	(0.690)
2080	130							1.315	(0.760)							1.488	(0.860)
2240	140							1.696	(0.980)							1.904	(1.100)
2400	150							2.267	(1.310)							2.561	(1.480)

Conductivity is little affected by temperature in the region of room temperature. At higher temperatures, the variation in conductivity is complex. It increases slowly with an increase in temperature up to a maximum at about 50 to 60 °C

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(122 to 140 °F). With the loss of water from the concrete as the temperature increases to 120 °C (248 °F), conductivity decreases sharply. At temperatures in excess of 120 to 140 °C (248 to 284 °F), the value of conductivity tends to stabilize:[8.37] at 800 °C (1470 °F) it is about one-half of the value at 20 °C (68 °F).[8.98]

Thermal conductivity is usually calculated from the diffusivity, the latter being easier to measure, but a direct determination of conductivity is of course possible. However, the method of test may affect the value obtained. For instance, the steady-state methods (hot plate and hot box) yield the same thermal conductivity for dry concrete, but give too low a value for moist concrete because the temperature gradient causes migration of moisture. For this reason, it is preferable to determine the conductivity of moist concrete by transient methods; the hot wire test has been found successful.[8.99]

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Thermal diffusivity

Diffusivity represents the rate at which temperature changes within a mass can take place, and is thus an index of the facility with which concrete can undergo temperature changes. Diffusivity, δ, is simply related to the conductivity K by the equation:

$$\delta = \frac{K}{c\rho}$$

where c is the specific heat, and ρ is the density of concrete.

From this expression, it can be seen that conductivity and diffusivity vary in step. Because of this direct relation, diffusivity is affected by the moisture content of the concrete, which depends on the original water content of the mix, degree of hydration of cement, and exposure to drying.

The range of typical values of diffusivity of ordinary concrete is between 0.002 and 0.006 m^2/h (0.02 to 0.06 ft^2/h), depending on the type of

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aggregate used. The following rock types are listed in order of increasing diffusivity: basalt, limestone, and quartzite.[8.10]

The measurement of diffusivity consists essentially of determining the relation between time and the temperature differential between the interior and the surface of a concrete specimen (both initially at the same temperature) when a change in temperature is introduced at the surface. Details of procedure and calculation are given in U.S. Bureau of Reclamation Procedure 4909-92.[8.8] Because of the influence of moisture in the concrete on its thermal properties, diffusivity should be measured on specimens with a moisture content which will exist in the actual structure.

Specific heat

Specific heat, which represents the heat capacity of concrete, is little affected by the mineralogical character of the aggregate, but is considerably increased by an increase in the moisture content of

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the concrete. Specific heat increases with an increase in temperature and with a decrease in the density of the concrete.[8.110] The common range of values for ordinary concrete is between 840 and 1170 J/kg per °C (0.20 and 0.28 Btu/lb per °F). The specific heat of concrete is determined by elementary methods of physics.

Another thermal property of concrete, which is of interest in consideration of fire effects, is *thermal absorptivity*. This is defined as $(K\rho c)^{1/2}$, where K is thermal conductivity, ρ is the density, and c is the specific heat. The thermal absorptivity of normal weight concrete was reported[8.33] as 2190 J/m^2 s$^{1/2}$ per °C (6.44 Btu/ft^2 h$^{1/2}$ per °F). For lightweight concrete with a density of 1450 kg/m^3 (90.5 lb/ft^3), the value is 930 J/m^2 s$^{1/2}$ per °C (2.73 Btu/ft^2 h$^{1/2}$ per °F).

Coefficient of thermal expansion

Like most engineering materials, concrete has a positive coefficient of thermal expansion, but its

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value depends both on the composition of the mix and on its hygral state at the time of the temperature change.

The influence of the mix proportions arises from the fact that the two main constituents of concrete, hydrated cement paste and aggregate, have dissimilar thermal coefficients, and the coefficient of concrete is a resultant of the two values. The linear coefficient of thermal expansion of hydrated cement paste varies between about 11×10^{-6} and 20×10^{-6} per °C (6×10^{-6} and 11×10^{-6} per °F),[8.88] and is higher than the coefficient of aggregate. In general terms, the coefficient of concrete is a function of the aggregate content in the mix (Table 8.3) and of the coefficient of the aggregate itself.[8.89] The influence of the latter factor is apparent from Fig. 8.11, and Table 8.4 gives the values of the coefficient of thermal expansion of 1:6 concretes made with different aggregates.[8.90] The significance of the difference between the coefficients of the aggregate and the hydrated cement paste was discussed

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on p. 148. Here, it can be added that this difference[8.5,8.34] may have a deleterious effect when combined with other actions. Thermal shock which produces a temperature differential of 50 °C (90 °F) between the surface of concrete and its core has been reported to cause cracking.[8.114]

Table 8.3. Influence of Aggregate Content on the Coefficient of Thermal Expansion[8.94]

Cement/sand ratio	*Linear coefficient of thermal expansion at the age of 2 years*	
	10^{-6} per °C	*10^{-6} per °F*
Neat cement	18.5	10.3
1:1	13.5	7.5
1:3	11.2	6.2
1:6	10.1	5.6

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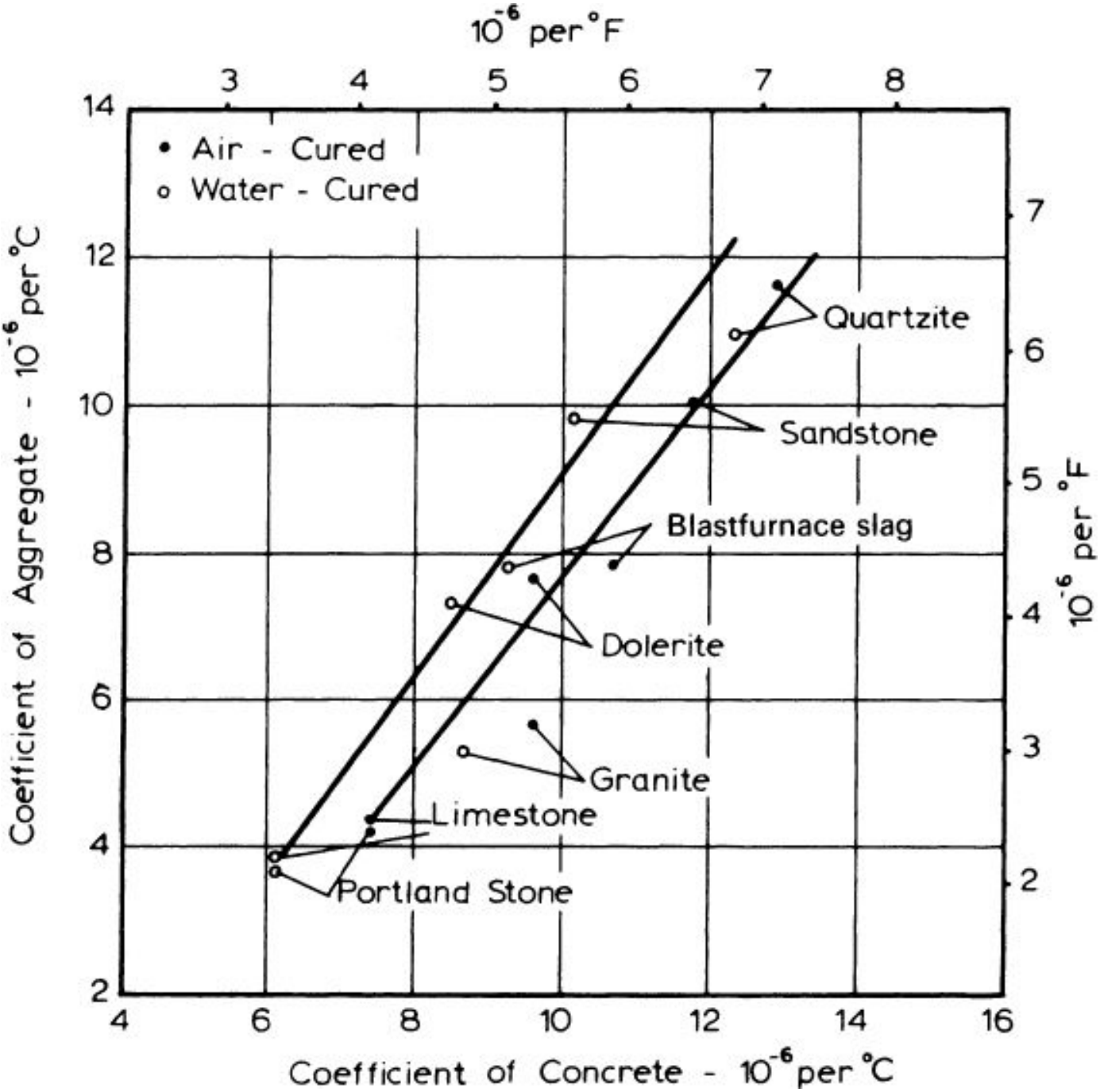


Fig. 8.11. Influence of the linear coefficient of thermal expansion of aggregate on the coefficient of thermal expansion of a 1:6 concrete[8.90] (Crown copyright)

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Table 8.4. Coefficient of Thermal Expansion of 1:6 Concretes made with Different Aggregates[8.90] (Crown copyright)

Type of aggregate	*Linear coefficient of thermal expansion*					
	Air-cured concrete		*Water-cured concrete*		*Air-cured and wetted concrete*	
	10^{-6} per °C	*10^{-6} per °F*	*10^{-6} per °C*	*10^{-6} per °F*	*10^{-6} per °C*	*10^{-6} per °F*
Gravel	13.1	7.3	12.2	6.8	11.7	6.5
Granite	9.5	5.3	8.6	4.8	7.7	4.3
Quartzite	12.8	7.1	12.2	6.8	11.7	6.5
Dolerite	9.5	5.3	8.5	4.7	7.9	4.4
Sandstone	11.7	6.5	10.1	5.6	8.6	4.8
Limestone	7.4	4.1	6.1	3.4	5.9	3.3
Portland stone	7.4	4.1	6.1	3.4	6.5	3.6
Blastfurnace slag	10.6	5.9	9.2	5.1	8.8	4.9
Expanded slag	12.1	6.7	9.2	5.1	8.5	4.7

The influence of the moisture condition applies to the paste component and is due to the fact that the thermal coefficient is made up of two parts: the true kinetic coefficient and swelling pressure. The latter arises from a decrease in the capillary tension of water held by the hydrated cement paste[8.91] and in the adsorbed water in it, with an increase in temperature.[8.40]

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The moisture-dependent part of the coefficient of thermal expansion does not include the movement of free water out of, or into, the concrete which results, respectively, in shrinkage and swelling. Because the moisture-related response to changes in temperature takes time, the resulting part of the coefficient of thermal expansion can be determined only when equilibrium has been reached. No swelling is possible, however, when the cement paste is dry, i.e. the capillaries are unable to supply water to the gel. Likewise, when the hydrated cement paste is saturated, no capillary menisci exist and there is therefore no effect of a change in temperature. It follows that, at these two extremes, the coefficient of thermal expansion is lower than when the paste is partially saturated. When the paste is self-desiccated, the coefficient is higher because there is not enough water for a free exchange of moisture to occur between capillary and gel pores after the temperature change.

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When saturated paste is heated, the moisture diffusion from gel to capillary pores, at a given gel water content, is partially offset by contraction as gel loses water so that the apparent coefficient is smaller.[8.100] Conversely, on cooling, the contraction due to moisture diffusion from capillary to gel pores, at a given gel water content, is partially offset by expansion which occurs when the gel absorbs water.[8.100]

Actual values are shown in Fig. 8.12, and it can be seen that, for young pastes, the coefficient is a maximum at a relative humidity of about 70 per cent. The relative humidity at which the coefficient is a maximum decreases with age, down to about 50 per cent for very old hydrated cement pastes[8.88] (Fig. 8.13). Likewise, the coefficient itself decreases with age owing to a reduction in the potential swelling pressure due to an increase in the amount of 'crystalline' material in the hardened paste. Using saturated concrete, Wittmann and Lukas[8.107] confirmed the decrease in the coefficient with age when the temperature

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is above freezing. No such variation in the coefficient of thermal expansion is found in high-pressure steam-cured cement paste because it contains no gel (Fig. 8.12). Only the values determined on saturated or desiccated specimens can be considered to represent the 'true' coefficient of thermal expansion, but it is the values at intermediate humidities that are applicable to many concretes under practical conditions.

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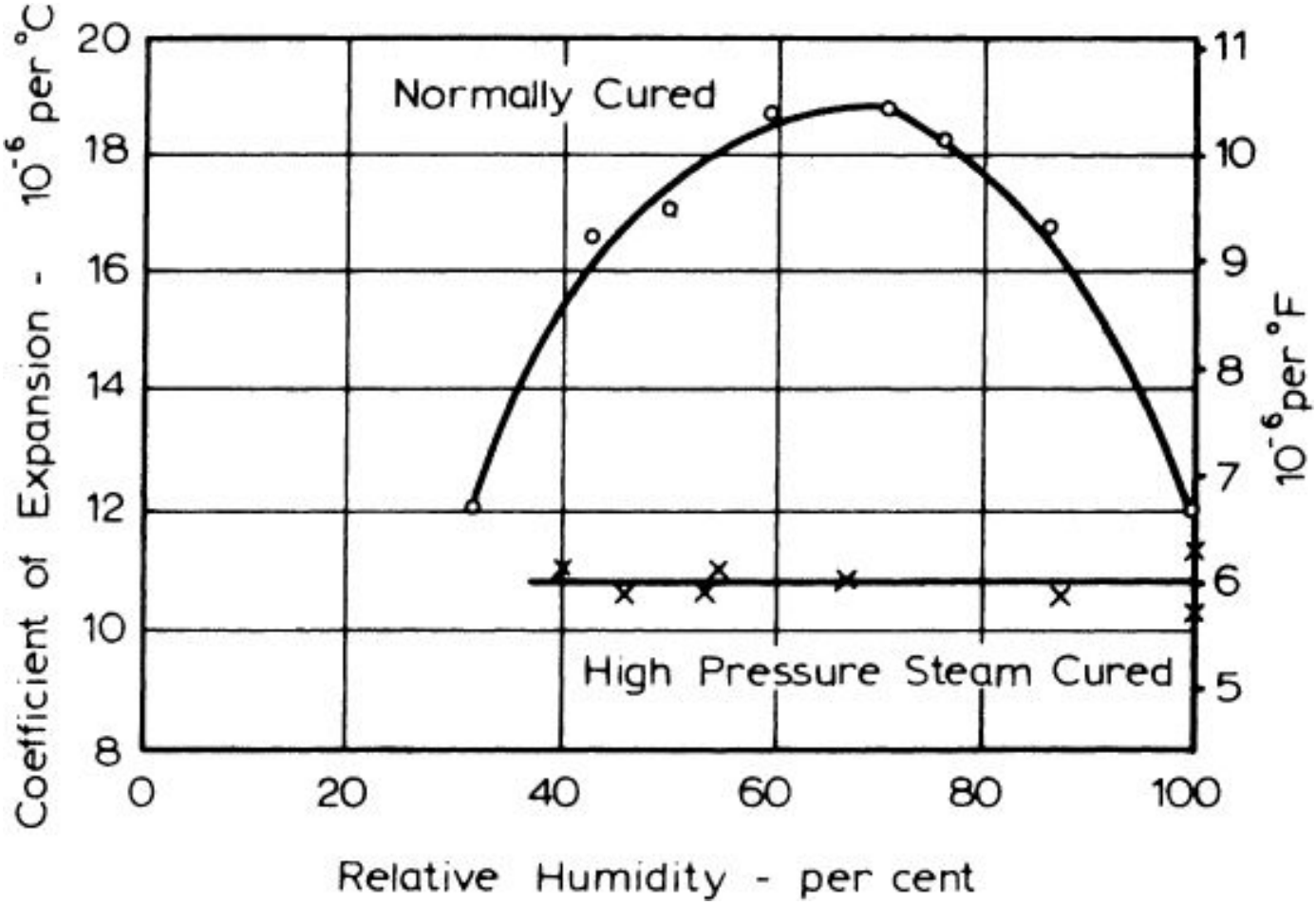


Fig. 8.12. Relation between ambient relative humidity and the linear coefficient of thermal expansion of neat cement paste cured normally and high-pressure steam cured[8.88]

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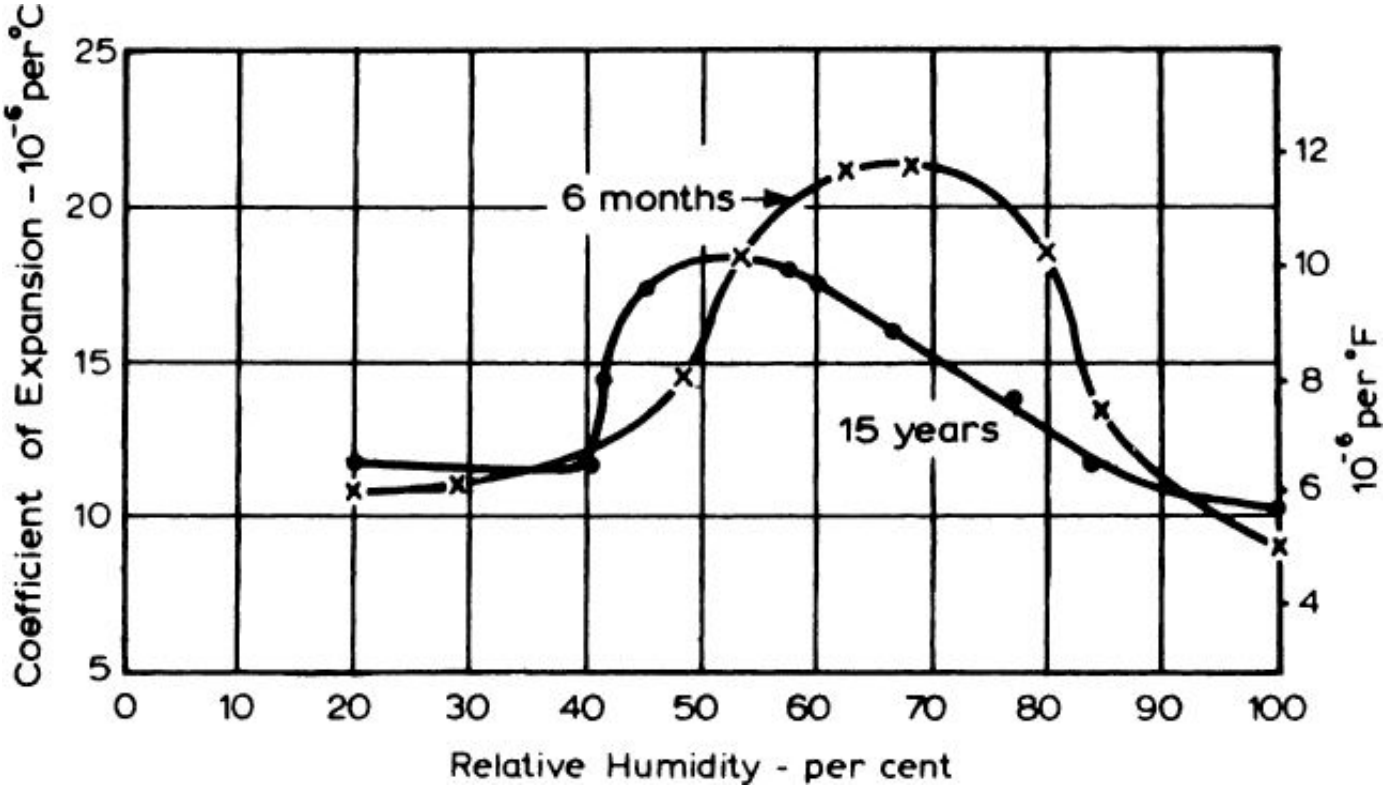


Fig. 8.13. The linear coefficient of thermal expansion of neat cement paste at different ages[8.88]

The chemical composition and fineness of cement affect the thermal expansion only in so far as they influence the properties of gel at early ages. The presence of air voids is not a factor.

Figures 8.12 and 8.13 refer to neat cement pastes but the effects are apparent also in concrete; here, though, the variation in the coefficient is smaller as only the paste component is affected

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by the relative humidity and ageing. Measurements of the coefficient of thermal expansion of concrete in a beam outdoors have confirmed that the coefficient varies with the moisture content of concrete and is higher (by perhaps as much as 10^{-6} per °C) when the concrete is drying.[8.39] For the same concrete, the coefficient of thermal expansion was found to be 11×10^{-6} per °C in winter and 13×10^{-6} per °C in summer.[8.39]

Table 8.4 gives values of the coefficient for 1:6 concretes: cured in air at 64 per cent relative humidity, saturated (water-cured), and wetted after air-curing. A method of determination of the linear coefficient of thermal expansion of oven-dry 'chemical-resistant' mortar is given in ASTM C 531-00 (2005) and of saturated concrete in U.S. Corps of Engineers Standard CRD-C 39-81.[8.30]

The data considered so far apply only at temperatures above freezing and below, say, 65 °C (150 °F). Considerably higher temperatures can, however, be encountered in some industrial ap-

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plications and in airfield pavements used by vertical take-off aircraft where concrete temperature of 350 °C (660 °F) was recorded.[8.38] Before commenting on the effect of high temperatures on the coefficient of thermal expansion of concrete, it is useful to note that the coefficient of neat cement paste decreases above a temperature of about 150 °C (300 °F) and becomes negative above a temperature of 200 to 500 °C; a value of -32.8×10^{-6} per °C was reported.[8.32] The change in the sign of the coefficient occurs at a lower temperature when the increase in temperature occurs slowly.[8.32] The reason for this is the loss of water from the hydrated cement paste and possibly internal collapse. The aggregate, however, has a positive coefficient of thermal expansion at all temperatures and this effect dominates the expansion of concrete, which expands with an increase in temperature up to high values. Values of the coefficient of thermal expansion at high temperatures are listed in Table 8.5.[8.92]

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Table 8.5. Coefficient of Thermal Expansion of Concrete at High Temperatures[8.92]

Curing condition	Water/cement ratio	Cement content kg/m³	Cement content lb/yd³	Aggregate	Linear coefficient of thermal expansion, at the age of: 28 days, Below 260° (500 °F)		28 days, Above 430 °C (800 °F)		90 days, Below 260 °C (500 °F)		90 days, Above 430 °C (800 °F)	
					10^{-6} per °C	10^{-6} per °F	10^{-6} per °C	10^{-6} per °F	10^{-6} per °C	10^{-6} per °F	10^{-6} per °C	10^{-6} per °F
Moist	0.4	435	735	Calcareous gravel	7.6	4.2	20.3	11.3	6.5	3.6	11.2	6.2
	0.6	310	520		12.8	7.1	20.5	11.4	8.4	4.7	22.5	12.5
	0.8	245	415		11.0	6.1	21.1	11.7	16.7	9.3	32.8	18.2
Air, 50 per cent relative humidity	0.4	435	735	Calcareous gravel	7.7	4.3	18.9	10.5	12.2	6.8	20.7	11.5
	0.6	310	520		7.7	4.3	21.1	11.7	8.8	4.9	20.2	11.2
	0.8	245	415		9.6	5.3	20.7	11.5	11.7	6.5	21.6	12.0
Moist air	0.68	355	600	Expanded shale	6.1	3.4	7.5	4.2	—	—	—	—
Air dried	0.68	355	600		4.7	2.6	9.7	5.4	5.0	2.8	8.8	4.9

At the other extreme, temperature near freezing results in a minimum positive value of the coefficient of thermal expansion; at still lower temperatures, the coefficient is higher again, and indeed somewhat higher than at room temperature.[8.107] Figure 8.14 shows the values of the coefficient for saturated hydrated cement paste tested in saturated air. In concrete slightly dried after a period of initial curing and then stored at a relative humidity of 90 per cent and tested at that humidity, the decrease in the coefficient of thermal expansion at low temperatures is absent (Fig. 8.14).

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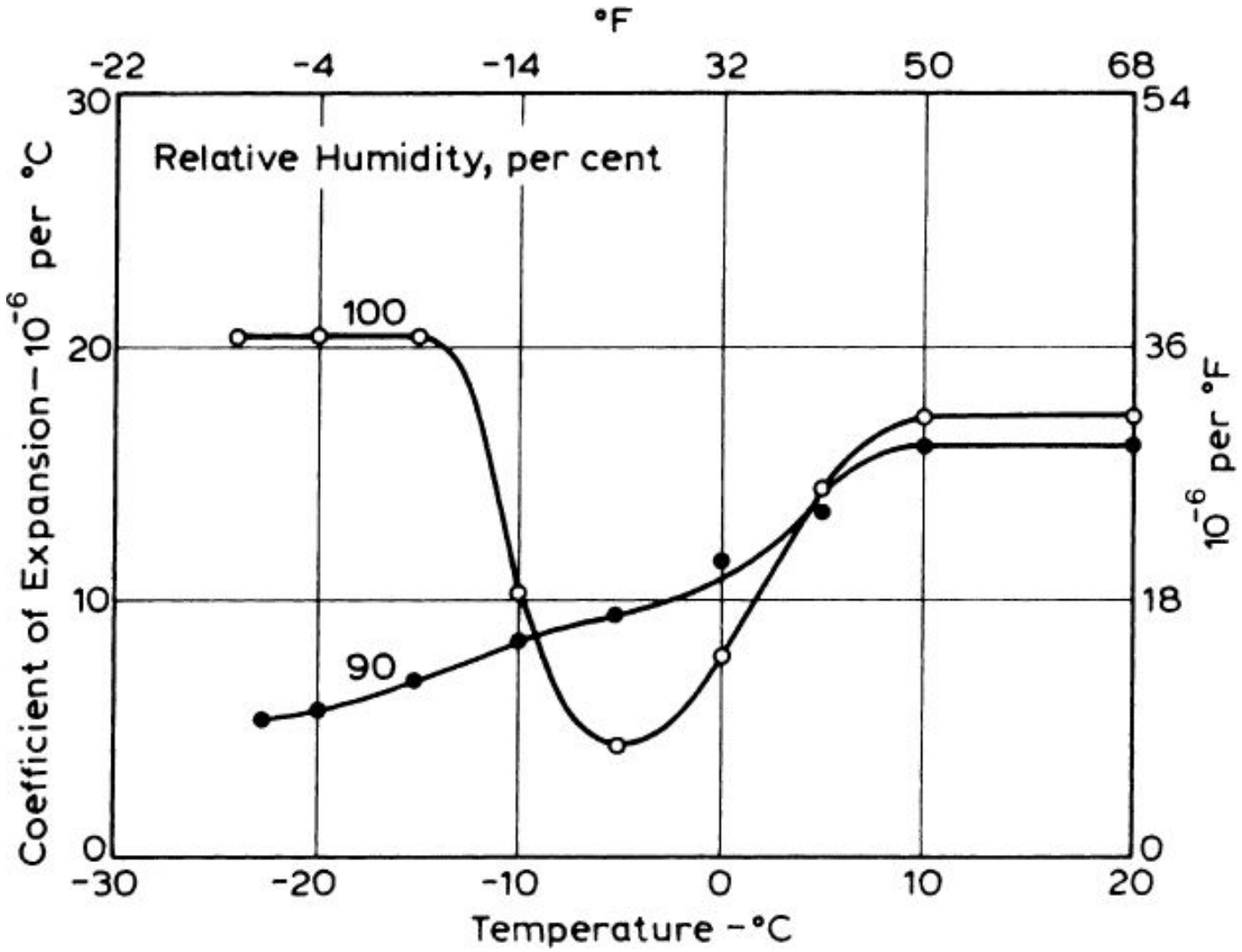


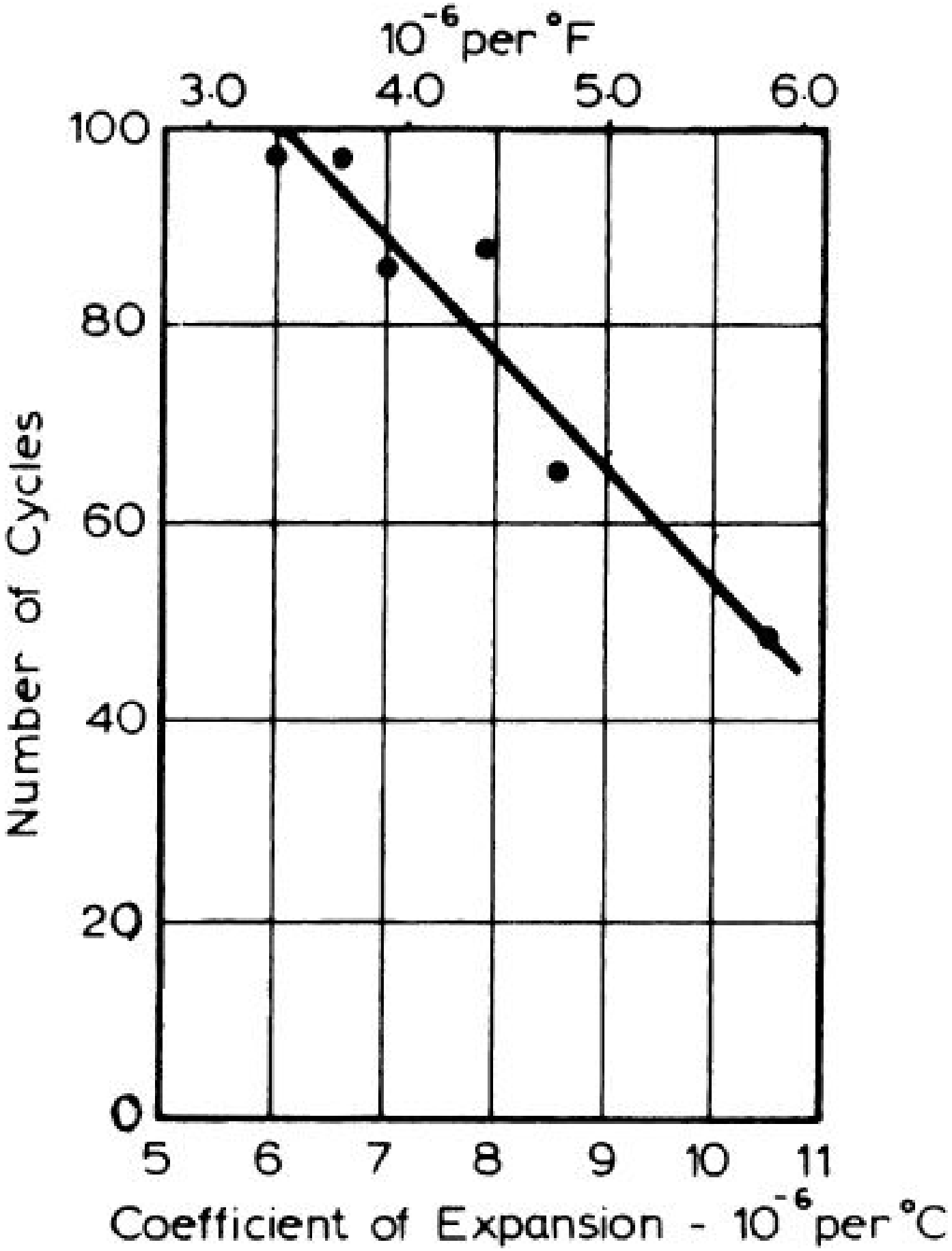
Fig. 8.14. Relation between the linear coefficient of thermal expansion and temperature of hydrated cement paste specimens (with a water/cement ratio of 0.40) stored and tested at the age of 55 days under different conditions of humidity[8.107]

Laboratory tests have shown that concretes with a higher coefficient of thermal expansion are

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less resistant to temperature changes than concretes with a lower coefficient.[8.89] Figure 8.15 shows the results of tests on concrete heated and cooled repeatedly between 4 and 60 °C (40 and 140 °F) at the rate of 2.2 °C (4 °F) per minute. However, the data are not sufficient for the coefficient of thermal expansion to be considered as a quantitative measure of durability of concrete subjected to frequent or rapid changes in temperature (cf. p. 148).

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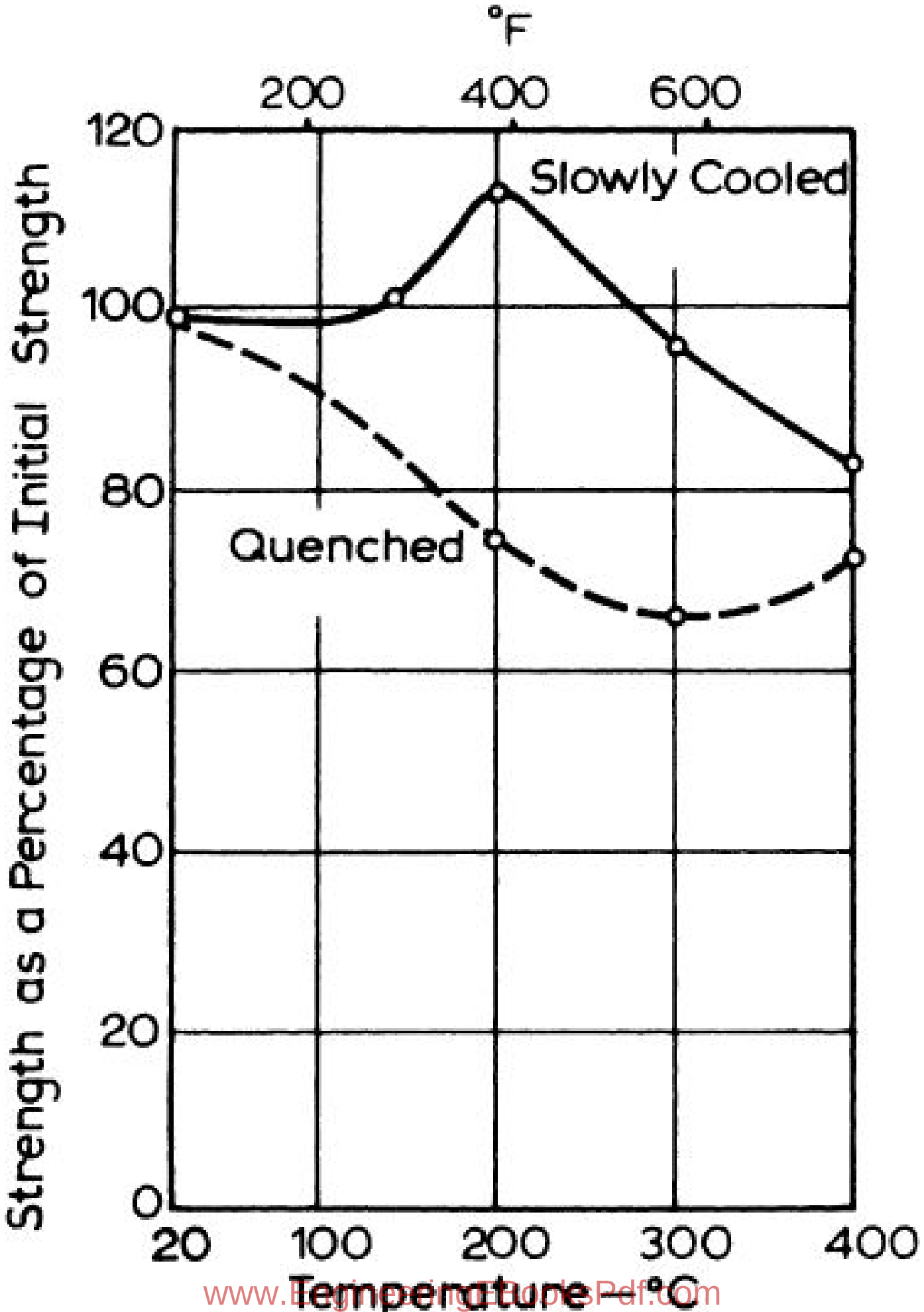


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Fig. 8.15. Relation between the linear coefficient of thermal expansion of concrete and the number of cycles of heating and cooling required to produce a 75 per cent reduction in the modulus of rupture[8.89]

Nevertheless, rapid changes in temperature, generally faster than encountered under normal conditions, may lead to deterioration of concrete: Fig. 8.16 shows the effects of quenching after heating to the indicated temperature.[8.93]

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Fig. 8.16. Effect of the rate of cooling on the strength of concrete made with a sandstone aggregate and previously heated to different temperatures[8.93]

Strength of concrete at high temperatures and resistance to fire

Reports on tests intended to establish the effect of exposure to high temperature, up to about 600 °C (1100 °F), give widely varying results. The reasons for this include: differences in the stress acting upon, and in the moisture condition of, the concrete while being heated; differences in the length of exposure to the high temperature; and the differences in the properties of the aggregate. In consequence, globally valid generalizations are difficult. Moreover, the knowledge of the strength of concrete may be required for different practical conditions of exposure; for instance, in the case of fire, the exposure to the high temperature is only of a few hours' duration but the heat flux is large and so is the mass of con-

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crete subjected to it. Conversely, in cutting concrete by a thermic lance, the exposure to high temperature is only of a few seconds' duration and the heat flux applied is very low. In what follows, test data from several investigations will be referred to, and these have to be interpreted in the light of the foregoing comments. The adverse effects are due to decomposition of the CSH gel and of C_2S.[8.116]

The compressive and splitting tensile strengths of concrete, made with limestone aggregate, exposed to a high temperature for 1 to 8 months are shown in Fig. 8.17.[8.45] The specimens tested were 100 mm by 200 mm (4 in. by 8 in.) cylinders, moist-cured for 28 days, then stored in the laboratory for 16 weeks. They were then heated at the rate of up to 20 °C per hour (36 °F per hour) under conditions such that loss of water from the concrete could take place. From Fig. 8.17, it can be seen that, relative to the strength prior to the exposure to the high temperature, there is a steady loss in strength with an increase

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in temperature. The relative loss in compressive strength is very slightly smaller at the water/cement ratio of 0.60 than at the water/cement ratio of 0.45; this trend does not necessarily continue down to the water/cement ratio of 0.33.[8.42] However, leaner mixes appear to suffer a relatively lower loss of strength than richer ones.[8.95]

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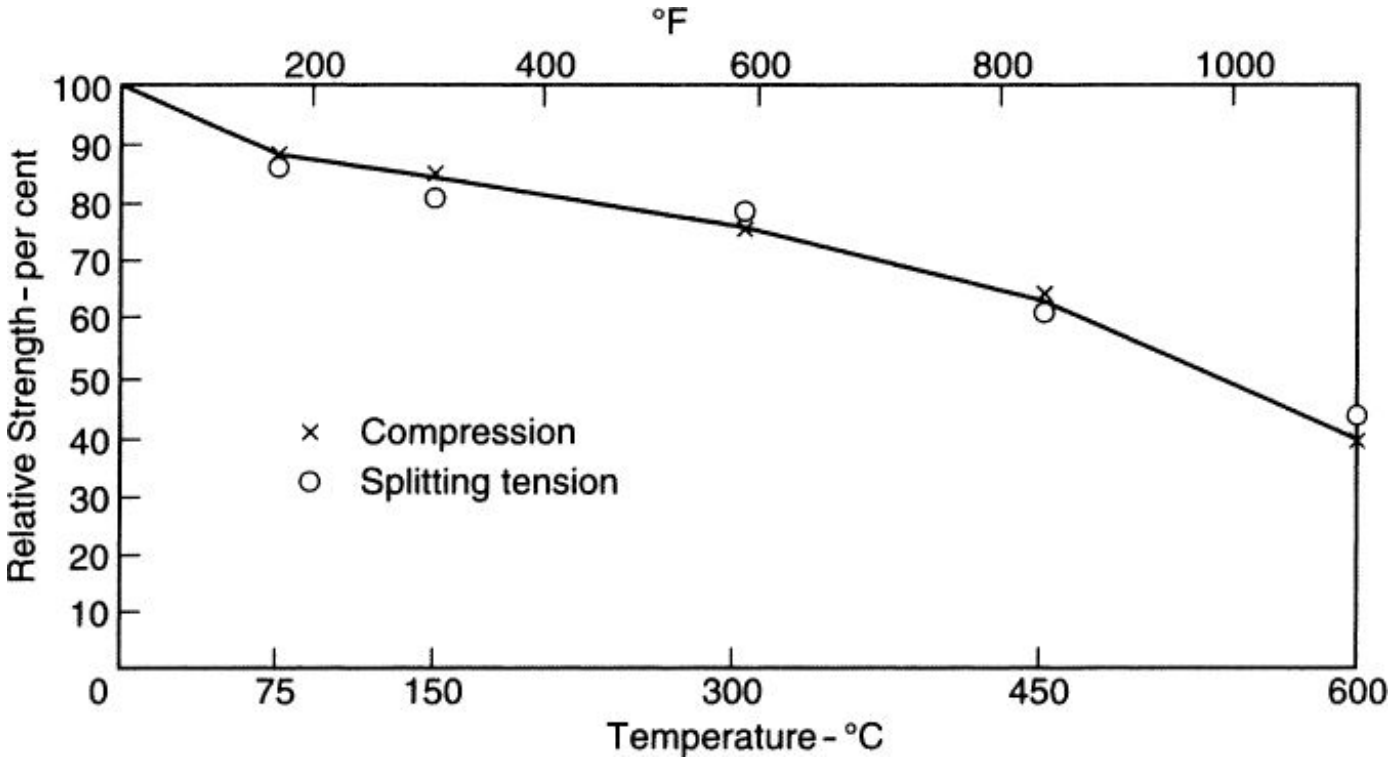


Fig. 8.17. Influence of exposure to a high temperature on the compressive and splitting-tensile strengths of concrete, made with a water/cement ratio of 0.45, expressed as a percentage of strength before exposure (based on ref. 8.45)

The influence of the water/cement ratio on the loss of strength is not noticeable in the splitting tensile strength; the loss in this strength is similar to that in the compressive strength.[8.45] It can be added that no effect of the length of exposure (between 1 and 8 months) was observed.

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Also, there was no difference in the relative loss of strength between concrete made with Portland cement only and concrete containing fly ash or ground granulated blastfurnace slag.[8.45]

Further tests by the same researchers[8.42] have shown that an increase in the length of exposure to a temperature of 150 °C (302 °F) or higher, from 2 to 120 days, increases the loss of compressive strength. However, the major part of the loss occurs early.[8.42] Tests[8.44] on concrete with basalt aggregate showed that the major part of the loss of strength occurs within 2 hours of the rise in temperature. It should be noted, however, that the exposure temperature is not necessarily the same as the temperature within the concrete so that it has to be emphasized once again that the details of the test method influence the measured output of the tests, but these details cannot always be fully appreciated from the published description of the tests. All these factors lead to a broad band of the loss of strength as a function of temperature, as shown in Table 8.6.

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Table 8.6. Compressive Strength as a Percentage of 28-day Strength at Room Temperature (based on ref. 8.44)

Maximum temperature, °C	20	200	400	600	800
Range of residual strength, per cent	100	50–92	45–83	38–69	20–36

Lightweight aggregate concrete exhibits a much lower loss of compressive strength than normal weight concrete: a residual strength of at least 50 per cent after exposure to 600 °C was reported.[8.112]

Tests[8.48] on high strength concrete (89 MPa) suggest a higher relative loss of strength than is the case with normal strength concrete. What is more important with respect to high performance concrete, which contains silica fume, is the occurrence of explosive spalling associated with high temperature. This was observed by Hertz[8.47] in concrete heated to temperatures in excess of about 300 °C (570 °F) even at a relatively slow rate of rise in temperature of 60 °C per hour,

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which is an order of magnitude lower than in a fire. Explosive spalling was confirmed in tests on concrete containing silica fume and having a water/cement ratio of 0.26.[8.43] This might seem surprising as the volume of water involved is small but, on the other hand, the permeability is extremely low.

It can be stated more generally that the risk of explosive spalling is higher the lower the permeability of the concrete and the higher the rate of rise in temperature. An associated observation is that the loss in strength at higher temperatures is greater in saturated than in dry concrete, and it is the moisture content at the time of application of load that is responsible for the difference.[8.101]

The influence of moisture content on strength is apparent also in fire tests on concrete, where excessive moisture at the time of fire is the primary cause of spalling. In general, moisture content of the concrete is the most important factor determining its structural behaviour at higher temperatures.[8.111] In massive concrete

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members, moisture movement is extremely slow so that the effects of a high temperature, while loss of water is prevented, may be more serious than in thin members. Inclusion of polypropylene fibres in the mix is helpful.

One of the changes which occurs as the temperature rises to about 400 °C (or 750 °F) is the decomposition of calcium hydroxide so that lime is left behind in consequence of drying.[8.7] If, however, after cooling, water ingresses into concrete, the re-hydration of lime can be disruptive; thus the damage manifests itself subsequently to the fire. From this standpoint, inclusion of pozzolanas in the mix, which remove calcium hydroxide, is beneficial.

While it is the behaviour of concrete that is of practical interest, the overall behaviour of concrete may mask some of the changes which occur in small specimens of hydrated cement paste. Tests[8.46] on paste specimens having a water/cement ratio of 0.30 and wet-cured for 14 weeks, heated and tested in compression while hot,

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showed a decrease in strength with an increase in temperature up to 120 °C (248 °F). At higher temperatures, the strength was found to be approximately equal to the original value. This strength is maintained up to 300 °C (572 °F). However, at still higher temperatures, there is a severe and progressive decrease in strength. The unimpaired strength at intermediate temperatures is ascribed by Dias *et al.*[8.46] to the disappearance of the disjoining pressure (see p. 37) and densification of the gel. In concrete, such changes would be limited by the difficulty of effective drying.

Modulus of elasticity at high temperatures

The behaviour of structures is often dependent on the modulus of elasticity of the concrete, and this modulus is strongly affected by temperature. The pattern of influence of temperature on the modulus of elasticity is shown in Fig. 8.18. For mass-cured concrete, there is no difference in modulus in the range of 21 to 96 °C (70 to 205 °F),[8.102] but the modulus of elasticity is reduced at temperat-

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ures in excess of 121 °C (250 °F).[8.56] However, when water can be expelled from concrete, there is a progressive decrease in the modulus of elasticity between about 50 and 800 °C (120 and 1470 °F) (see Fig. 8.18);[8.43,8.104] relaxation of bonds may be a factor in this. The extent of the decrease in the modulus depends on the aggregate used, but a generalization on this subject is difficult. In broad terms, the variation of strength and of modulus with temperature is of the same form.

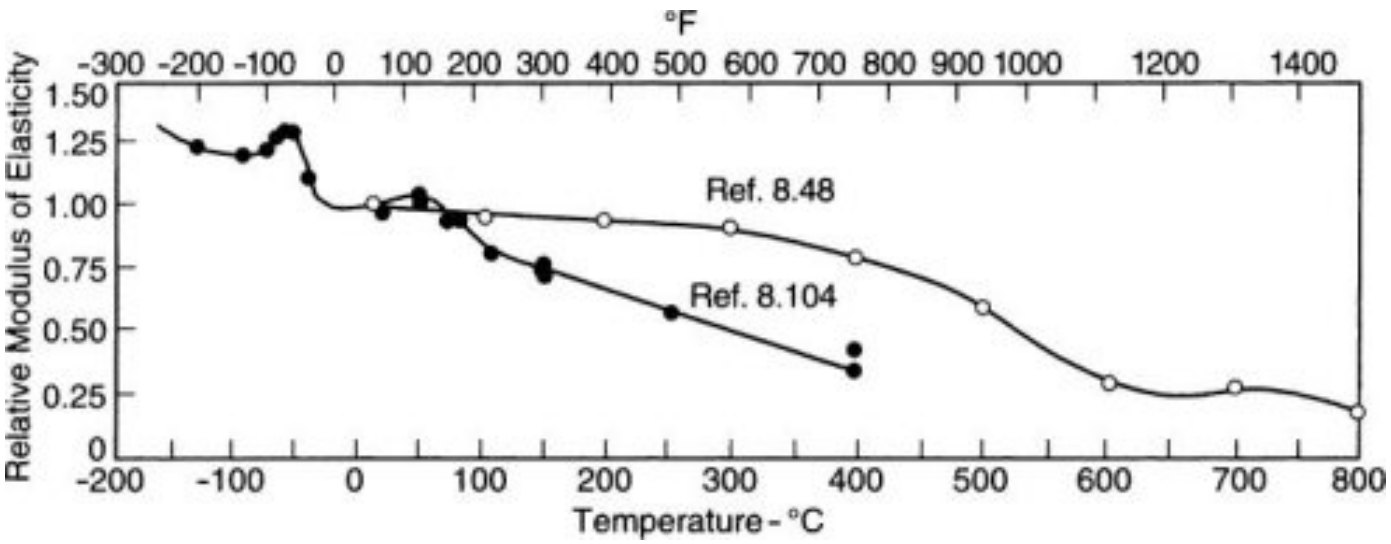


Fig. 8.18. Influence of temperature on modulus of elasticity of concrete (based on refs 8.48 and 8.104)

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Behaviour of concrete in fire

Although reference to fire was made on several occasions, the full treatment of the resistance of concrete to fire is a topic outside the scope of this book because fire endurance applies really to a building element rather than to a building material. We can say, however, that, in general, concrete has good properties with respect to fire resistance; that is, concrete is non-combustible, the period of time under fire during which concrete continues to perform satisfactorily is relatively high, and no toxic fumes are emitted. The relevant criteria of performance are: load-carrying capacity, resistance to flame penetration, and resistance to heat transfer when concrete is used as a protective material for steel. A general review of the resistance of concrete to fire has been written by Smith.[8.6]

In practice, what is required of structural concrete is that it preserves structural action over a desired length of time (known as fire rating). This is distinct from being heat resisting.[8.78] Consid-

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ering the behaviour of concrete as a material, we should note that fire introduces high temperature gradients and, as a result, the hot surface layers tend to separate and spall from the cooler interior of the body. The formation of cracks is encouraged at joints, in poorly compacted parts of the concrete, or in the planes of reinforcing bars; once the reinforcement has become exposed, it conducts heat and accelerates the action of high temperature.

The type of aggregate influences the response of concrete to high temperature. The loss of strength is considerably lower when the aggregate does not contain silica (some forms of which undergo change), e.g. with limestone, basic igneous rocks, and particularly with crushed brick and blastfurnace slag. Concrete with a low thermal conductivity has a better fire resistance so that, for instance, lightweight concrete stands up better to fire than ordinary concrete.

It is interesting to note that dolomitic gravel leads to a very good fire resistance of concrete.

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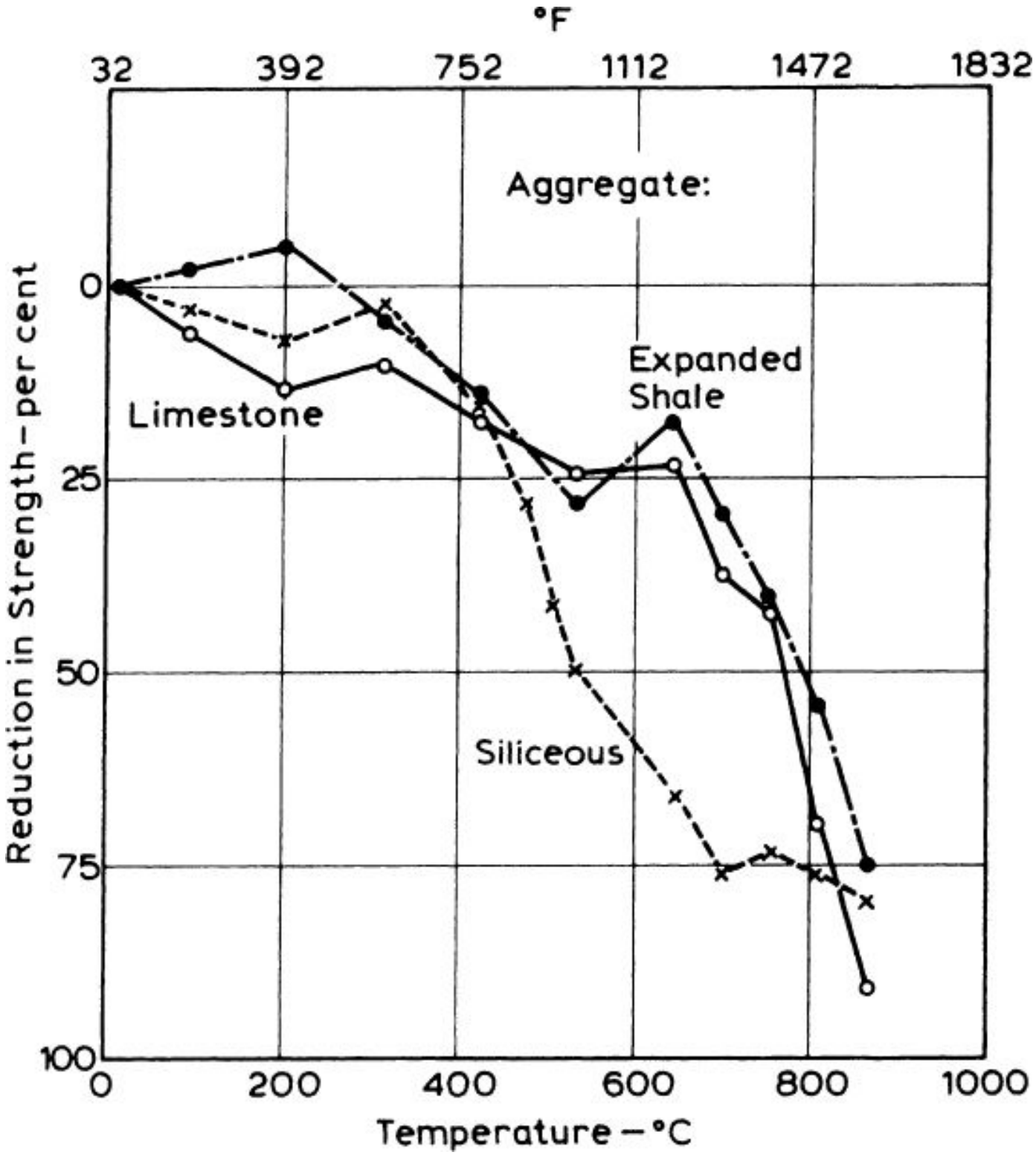
The reason for this is that the calcination of the carbonate aggregate is endothermic;[8.103] in consequence, heat is absorbed and a further temperature rise is delayed. Also, the calcined material has a lower density and therefore provides a measure of surface insulation. This effect is significant in thick members. On the other hand, if pyrites is present in the aggregate, slow oxidation at about 150 °C (330 °F) causes disintegration of the aggregate and consequently rupture of the concrete.[8.42]

Abrams[8.108] confirmed that, at temperatures above about 430 °C (810 °F), siliceous aggregate concrete loses a greater proportion of its strength than concretes made with limestone or lightweight aggregates but, once the temperature has reached some 800 °C (1470 °F), the difference disappears (Fig. 8.19). For practical purposes, about 600 °C (1100 °F) can be considered as the limiting temperature for structural integrity of concrete made with Portland cement; at higher temperatures, refractory concrete has to be used

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(see p. 102). The relevant temperature is that of the concrete itself and not the temperature of the flame or of gases. Sullivan[8.117] showed that explosive spalling occurs especially at a w/c of 0.35 because the concrete has a particularly low surface permeability (see also p. 689.)

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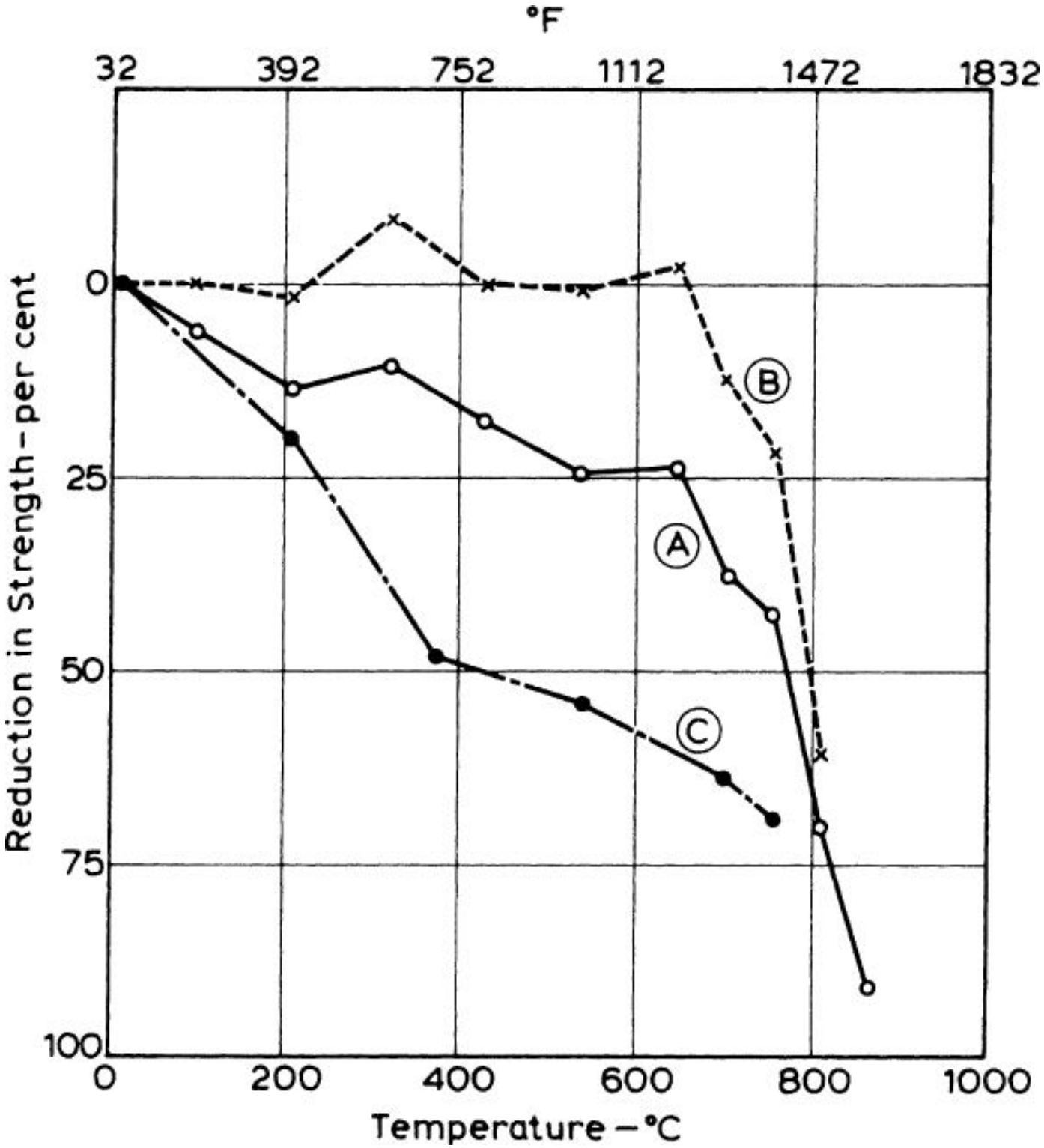


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Fig. 8.19. Reduction in compressive strength of concrete heated without application of load and then tested hot; average initial strength of 28 MPa (4000 psi)[8.108]

With all aggregates, the percentage loss of strength was found to be independent of the original level of strength but the sequence of heating and loading influences the residual strength. Specifically, concrete heated under load retains the highest proportion of its strength, whereas heating unloaded specimens leads to the lowest strength of the subsequently cooled concrete. Application of load while the concrete is still hot leads to intermediate values. Typical results are shown in Fig. 8.20 (Figure 2.9 may be also of interest.)

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Fig. 8.20. Reduction in compressive strength of concrete made with limestone aggregate: (*A*) heated without application of load and then tested hot; (*B*) heated under an initial stress/strength ratio of 0.4 and then tested hot; (*C*) heated without application of load and tested after 7 days of storage at 21 °C (70 °F)[8.108]

Application of water in a fire is tantamount to quenching: this causes a large reduction in strength because severe temperature gradients are set up in the concrete.

Concretes made with siliceous or limestone aggregate show a change in colour with temperature. As this change is dependent on the presence of certain compounds of iron, there is some difference in the response of different concretes. The change in colour is permanent, so that the maximum temperature during a fire can be estimated *a posteriori*. The colour sequence is approximately as follows: pink or red between 300 and 600 °C, then grey up to about 900 °C, and

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buff above 900 °C.[8.93] Thus, the residual strength can be approximately judged: generally, concrete whose colour has changed beyond pink is suspect, and concrete past the grey stage is probably friable and porous.[8.1]

Attempts have been made to determine the maximum temperature which concrete had reached during a fire by measuring the reduction in thermoluminescence. This is a light signal which is a function of temperature. However, the light output is affected by the length of exposure to the high temperature so that the reduction in the strength of concrete exposed to fire for a prolonged period can be significantly underestimated.[8.41]

Deliberate application of very high temperature over a small area is used in flame cleaning of concrete surfaces. This does not damage the concrete beyond the depth removed, which is 1 to 2 mm (0.04 to 0.08 in.) provided the blowpipe is moved at the requisite rate.[8.109] Under such cir-

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cumstances, even though the flame temperature is about 3100 °C, the maximum concrete temperature is not more than 200 °C.

Strength of concrete at very low temperatures

The development of strength of concrete at temperatures higher than –11 °C (12 °F) was considered on p. 305, this being the lowest temperature at which hydration takes place and a gain in strength occurs. There exist, however, practical situations of exposure to cryogenic temperatures of concrete which has hardened at room temperature; this is, for instance, the case in storage tanks for liquefied natural gas whose boiling point is –162 °C (–260 °F). The effect of these very low temperatures will now be considered.

At temperatures ranging from the freezing point of water down to about –200 °C (–330 °F), the strength of concrete is markedly higher than at room temperature. The compressive strength may be as high as two to three times the strength

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at room temperature when the concrete is moist while being chilled, but the compressive strength of air-dry concrete increases very much less.

The difference in the increase in strength between wet and dry concretes is related to the formation of ice in the hydrated cement paste. The freezing point of gel water is lower the smaller the pore size so that all the adsorbed water becomes frozen at a temperature between –80 and –95 °C (–112 and –139 °F). As ice can resist stress, unlike the water which it replaces, frozen concrete has an extremely low effective porosity and, therefore, high strength. The strength of ice and its coefficient of thermal expansion vary with temperature so that the changes occurring in hydrated cement paste are complex.[849]

If the concrete is not exposed to low temperature, empty pores remain empty so that the increase in strength is small.

The pattern of the relation between compressive strength and temperature, both for moist and for air-dry lightweight aggregate concrete, is

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shown in Fig. 8.21. The corresponding data for splitting-tensile strength are shown in Fig. 8.22. From this figure, it can be seen that the increase in the tensile strength occurs mainly between –7 and –87 °C. Also the relative increase in the tensile strength of air-dry concrete is smaller than the relative increase in compressive strength. The data in Figs 8.21 and 8.22 refer to lightweight aggregate concrete which, for cryogenic purposes, has the advantage of good insulating properties. However, in normal weight concrete, the increase in strength at low temperatures is greater than is the case with lightweight aggregate concrete.

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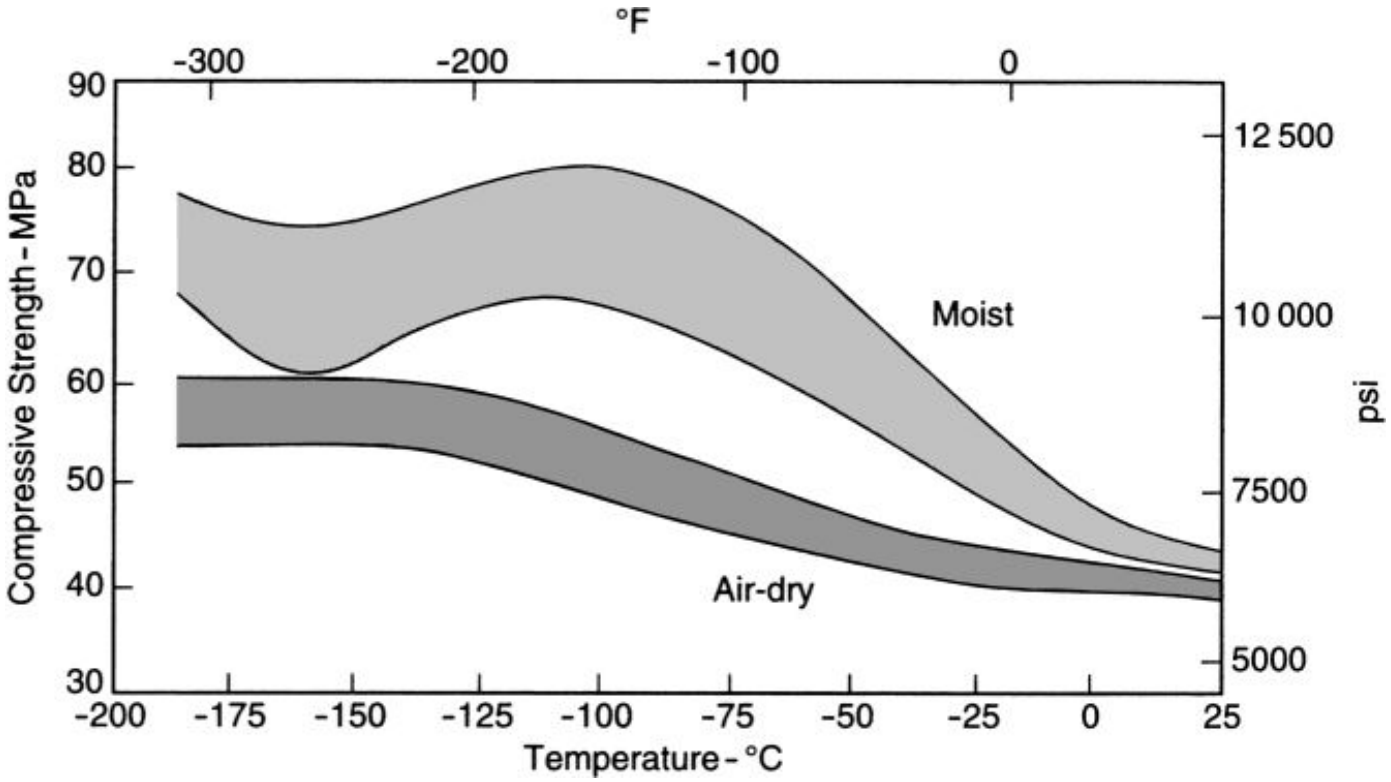


Fig. 8.21. Effect of very low temperatures on compressive strength of concrete (measured on standard cylinders) (based on ref. 8.49)

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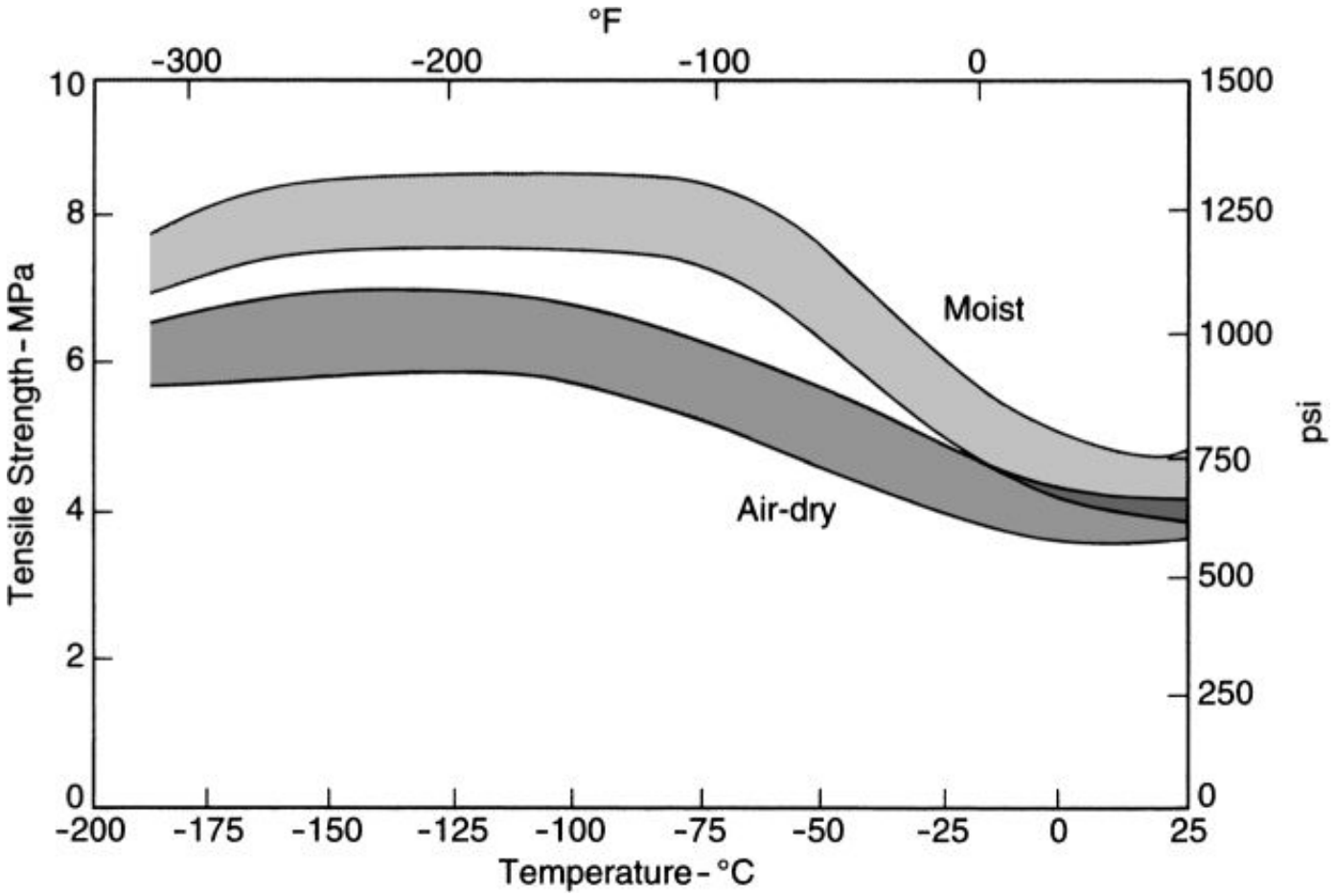


Fig. 8.22. Effect of very low temperatures on splitting-tensile strength of concrete (based on ref. 8.49)

The pattern of the increase in compressive strength with an increase in moisture content is independent of the water/cement ratio; an example of this relation for concrete at –160 °C is shown in Fig. 8.23.[8.50] Similar behaviour applies to concrete with a strength of 80 MPa (12 000 psi) at normal temperature.[8.51]

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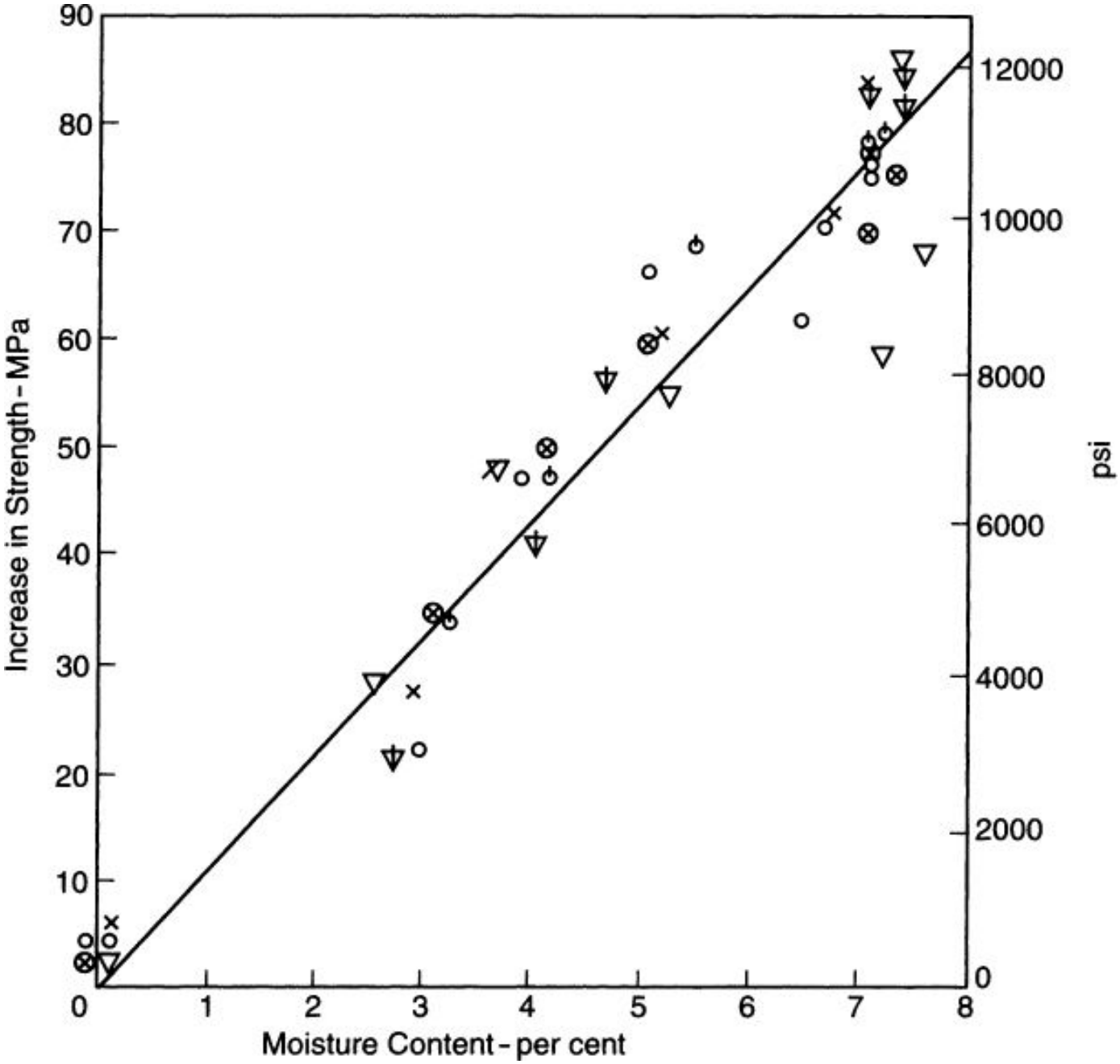


Fig. 8.23. Relation between increase in compressive strength at –160 °C (–256 °F) above the strength at room temperature and moisture content for concretes with water/cement ratios of 0.45 and 0.55 (based on ref. 8.51)

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Figure 8.21 shows that there is little, if any, further increase in compressive strength when the temperature drops below about –120 °C. The reason for this is that, in the region of that temperature, changes occur in the structure of ice. Specifically, at –113 °C, ice changes from hexagonal to orthorhombic structure; this change is accompanied by a decrease in volume of about 20 per cent. The pattern of strain development with a decrease in temperature and the behaviour of concrete under cyclic temperature have been extensively studied by Miura.[8.50] It should be noted that the effects of temperature gradients and of temperature cycling need to be considered in structural design.

The modulus of elasticity of moist concrete increases steadily with the decrease in temperature down to –190 °C. At that temperature, the modulus of elasticity is about 1.75 times the modulus at room temperature; for air-dry concrete the corresponding value is about 1.65.[8.49]

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Mass concrete

In the past, the term ‘mass concrete’ was applied only to concrete of massive dimensions, such as gravity dams, but nowadays the technological aspects of mass concrete are relevant to any concrete member of such dimensions that the thermal behaviour may lead to cracking unless appropriate measures are taken. The crucial feature of mass concrete is, thus, its thermal behaviour, a design objective for such concrete being to avoid or to reduce and control the width and spacing of cracks.

It can be recalled from Chapter 1 that hydration of cement generates heat which causes a rise in the temperature of the concrete. If this rise occurred uniformly throughout a given concrete element without any external restraint, then the element would expand until the maximum temperature has been reached; thereafter, as the concrete cools due to the loss of heat to the ambient atmosphere, uniform contraction would occur. Thus, there would be no thermal stresses

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within the element. In practice, however, restraint exists in all but the smallest of concrete members. There are two categories of restraint: internal and external.

Internal restraint arises from the fact that, when the surface of concrete can lose heat to the atmosphere, there develops a temperature differential between the cool exterior and the hot core of the concrete element, the heat not being dissipated to the outside fast enough in consequence of the low thermal diffusivity of the concrete. As a result, the *free* thermal expansion is unequal in the various parts of the concrete element. Restraint of the free expansion results in stresses, compressive in one part of the element and tensile in the other. If the tensile stress at the surface of the element due to the expansion of the core exceeds the tensile strength of concrete, or if it results in the tensile strain capacity being exceeded (see p. 294), then surface cracking will develop.

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The actual situation is complex because creep, which is high in very young concrete, relieves some of the compressive stress induced in the core so that the *rate of change* of the temperature is also a factor; this behaviour is discussed on p. 474.

Internal restraint can occur also when concrete is placed against a surface at a much lower temperature, such as cold ground or uninsulated formwork in cold weather. In such a situation, different parts of the concrete element set at different temperatures. When, subsequently, the core of the concrete element cools, its thermal contraction is restrained by the already cool external part and cracking in the interior may occur.

Examples of temperature changes are shown in Figs 8.24 and 8.25, which imply that cracking will occur when the temperature difference exceeds 20 °C (36 °F). This limit on the temperature difference was suggested by FitzGibbon.[8.65,8.66] For a temperature difference of 20 °C, taking the coefficient of thermal expansion of concrete as 10

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$\times 10^{-6}$ per °C (5.5×10^{-6} per °F) (see Table 8.4), the differential strain is 200×10^{-6}. This is a realistic estimate of tensile strain at cracking (see p. 292). The following practical experience can be quoted.

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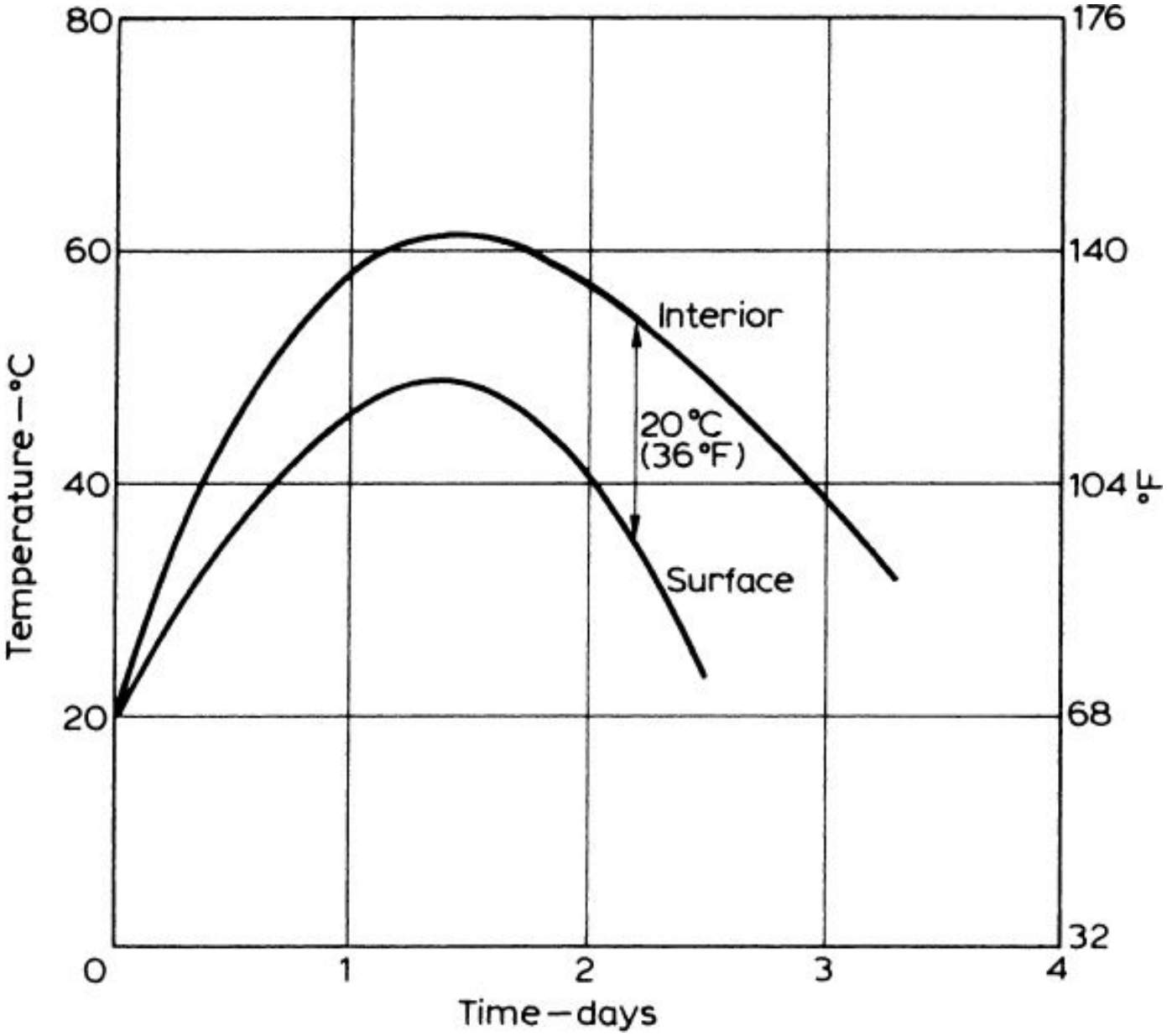


Fig. 8.24. An example of the pattern of temperature change which causes external cracking of a large concrete mass. The critical 20 °C temperature difference occurs during cooling[8.66]

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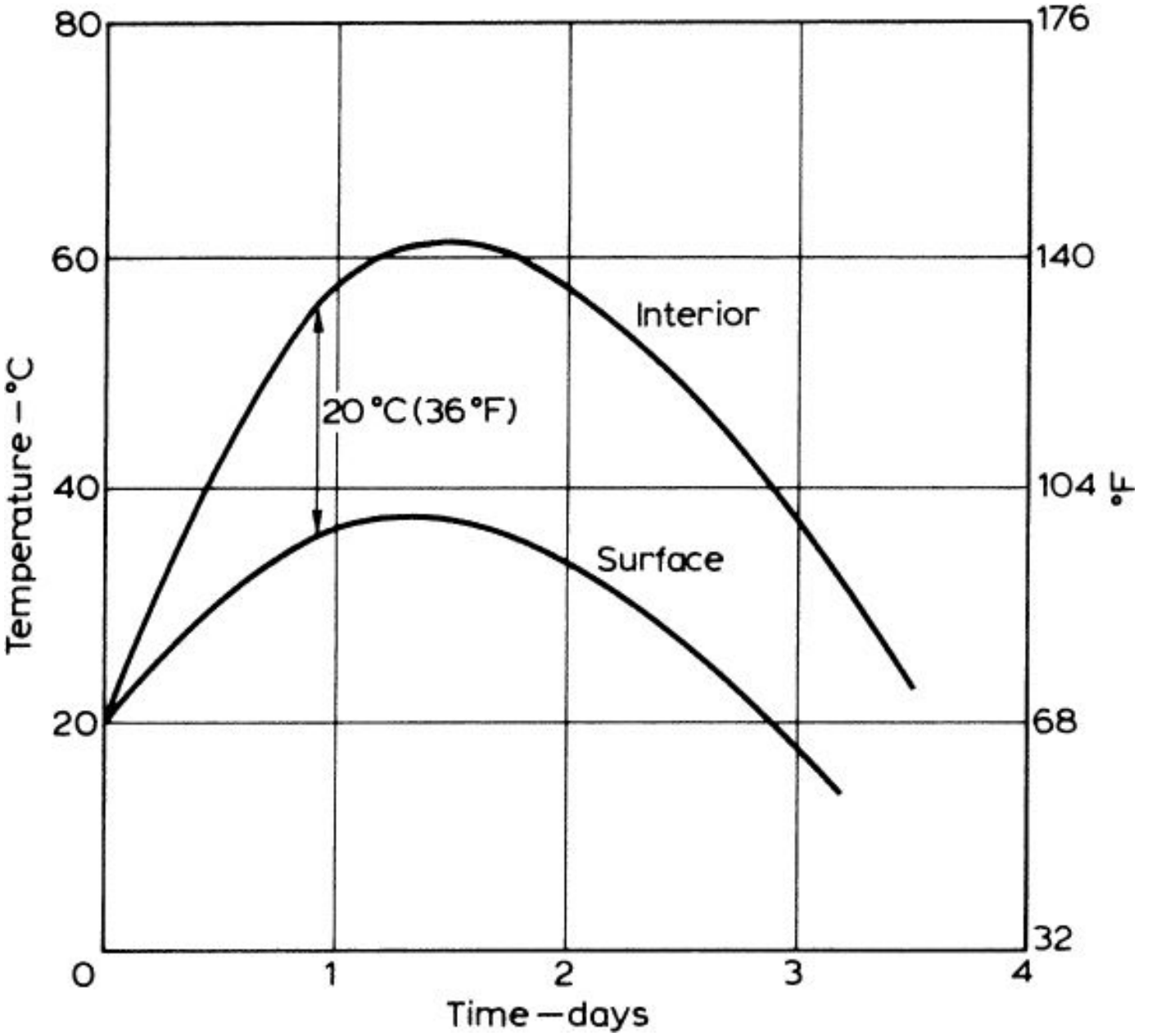


Fig. 8.25. An example of the pattern of temperature change which causes internal cracking of a large concrete mass. The critical 20 °C temperature difference occurs during heating, but the cracks open only when the interior has cooled through a greater temperature range than the exterior[8.66]

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In a 1.1 m (43 in.) square column made of reinforced concrete with a Type I cement content of 500 kg/m^3 (840 lb/yd^3) and a silica fume content of 30 kg/m^3 (50 lb/yd^3), a rise in temperature of 45 °C (81 °F) above the ambient temperature was observed 30 hours after placing.[8.52]

A similar rise in temperature can occur even in sections with a least dimension of 0.5 m (20 in.). The need for not allowing the surface of the concrete to cool too rapidly is obvious so that the insulating properties of the formwork and the time of its removal have to be controlled.

The preceding discussion has shown that the major cause of the temperature differential in a concrete element is the generation of heat by the hydration of cement. This topic was discussed on p. 38 in so far as the heat of hydration of a unit mass of different cements is concerned. It is, therefore, possible to choose a Portland cement with a chemical composition which leads to a low rate of heat development. However, with blended cements, an estimate of the heat hydration

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is more complicated. Moreover, from the standpoint of the development of a temperature differential, it is not only the total heat of hydration but also its rate of development that are relevant. It should be remembered that a higher fineness of cement leads to more rapid hydration, so that it might be desirable to avoid cements with a high specific surface.

The choice of cement, however, offers only a partial solution because it is the cement content per cubic metre of concrete that largely governs the heat generated. The remedy, therefore, lies in using a low cement content as well as in using blended cements because it is the Portland cement that is responsible for early heat generation, pozzolanas reacting chemically more slowly. It follows that, using a low content of blended cement with a high proportion of pozzolanas, the maximum temperature rise can be reduced and its occurrence can be delayed. The benefit of the delay is that the concrete will have a higher tensile strength and be less prone to cracking.

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With any cement, the rate of hydration is higher at higher temperatures so that cooling the fresh concrete below the ambient temperature (see next section) and placing it at a low temperature reduces the rate of generation of heat; in addition, the difference between the maximum temperature of the concrete and the *final* ambient temperature is reduced.

In large *plain* concrete structures, the use of aggregate with a large maximum size, 75 mm (3 in.) or even 150 mm (6 in.), may be desirable because this allows a reduction in the water content of the mix for a given workability. At a fixed water/cement ratio, the cement content can, therefore, be reduced. The water/cement ratio can be high (up to 0.75) because, in structures such as gravity dams, the strength of concrete is of little structural importance, prevention of cracking and durability being critical. In any case, it is the strength at greater ages that is likely to be of relevance. Mixes with a blended cement content of 109 kg/m^3 (184 lb/yd^3), of which 67 per cent

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was pozzolana, have been used; the water content was 48 kg/m^3 (80 lb/yd^3), the slump 40 mm (1$\frac{1}{2}$ in.), and the 28-day cylinder was strength 14 MPa (2000 psi).[8.67] We can note that using a very low cement content is not only economical *per se* but leads also to economy in other measures used to overcome the undesirable effects of the heat of hydration of the cement, such as cooling the concrete in situ by circulating chilled water through embedded pipework.[8.67]

We can add that some recent dams have been built using roller-compacted concrete with a cement content as low as 66 kg/m^3 (112 lb/yd^3) of which 30 per cent was fly ash.[8.54] However, this specialized material and the associated technology are outside the scope of the present book.

Let us now consider *reinforced concrete*: here a much higher strength is required, often at 28 days, and the use of large-size aggregate may be impractical because of reinforcement spacing or because obtaining such aggregate may be uneco-

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nomical. Also, embedding pipework may not be permitted. The essential problem is, nevertheless, the same as in plain concrete, i.e. the interior of the mass will heat up more than the exterior if the loss of heat at the surface is large. If the difference in temperature between the interior and the exterior is large enough, cracking will develop. However, appropriate detailing of the reinforcement can control the width and spacing of the cracks. FitzGibbon[8.65,8.66] estimated that the temperature rise under adiabatic conditions is 12 °C per 100 kg of cement per cubic metre of concrete (13 °F per 100 lb/yd^3), regardless of the type of cement used, for cement contents between 300 and 600 kg/m^3 (500 and 1000 lb/yd^3).

The solution to the problem is not to limit the temperature rise in the interior but rather to prevent the heat loss at the surface. Thus, the entire concrete mass is allowed to heat, more or less to the same degree, and expand *without restraint*; with time, cooling, again more or less uniform throughout, takes place, and the structure reaches

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its final dimensions, again without restraint. To prevent a large heat loss, the formwork and the top surface of the structure must be adequately insulated with polystyrene or urethane; additional insulation is needed at edges and corners where the heat loss occurs in more than one direction and in other sensitive parts of the structure.

In practice, the temperature at various points should be monitored by thermo-couples, and insulation should be adjusted accordingly. The insulation must control loss of heat by evaporation, conduction, and radiation. To achieve the first, a plastic membrane or a curing compound should be used, but not spraying or ponding as these have a cooling effect. Plastic-coated quilts are useful in all respects but softboard can also be used. The insulation must be maintained until the temperature differential has been reduced to 10 °C (18 °F).

Other specialized measures are also required in order to achieve a monolithic structure without cold joints. One measure is a differential use of

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retarders so that the concrete in the lower part remains plastic until completion of placing, possibly in 12 hours; bleeding also needs to be controlled. One of the largest continuous pours to date is that of a reinforced concrete foundation containing 12 000 m^3 (16 000 yd^3) of concrete.[8.53]

It is useful to point out that care is required if concretes with dissimilar thermal properties are placed so as to create a monolithic element. An example of this is a highway slab placed in two layers (so that steel dowels in the contraction joint can be inserted) which contain different blended cements.[8.2]

External restraint of thermal movement can result in cracking of reinforced concrete members, even when thin. This is the case with walls cast onto an existing foundation which restrains the thermal movement due to the rise in the temperature of the wall concrete: vertical cracks through the full thickness of the wall at its base can extend a considerable distance upwards. Pre-

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vention of cracking can be achieved by appropriate structural detailing of the reinforcement, but an understanding of the thermal behaviour of concrete is essential to reduce the severity of the problem.

The preceding extensive discussion of the temperature rise in a concrete mass has shown that the temperature depends on the position in the concrete element, as well as on the age of the concrete, and on the details of the insulation. The properties of concrete at a particular position can be determined by the use of *temperature-matched curing*. This is a technique in which a thermocouple inserted into concrete at a specified position controls the temperature of a bath in which a concrete specimen is placed; the specimen is isolated from the water. The properties of the temperature-matched concrete which are of greatest interest are strength and creep. The knowledge of strength can be used to determine formwork striking times or the transfer of

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prestress. Creep is of relevance in structural design.

The determination of temperature in different locations within a concrete mass can be used to adjust thermal insulation so as to minimize temperature gradients within the mass.

A particular type of continuous pour of concrete is used in certain types of structures such as high-rise buildings and towers. Here, the formwork rises continuously, or nearly so, over a period of days or weeks. This is known as slipforming.[8.115] Horizontal slipforming can be used in the construction of curbs.

Concreting in hot weather

There are some special problems involved in concreting in hot weather, arising both from a higher temperature of the concrete and, in many cases, from an increased rate of evaporation from the fresh mix. These problems concern the mixing, placing and curing of the concrete.

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Hot-weather concreting is not so much an unusual or a specialized process; rather, it requires taking certain recognized measures to minimize or control the effects of high ambient temperature, high temperature of the concrete, low relative humidity, high wind velocity, and high solar radiation. What is required on each construction project where any one or more of the above conditions exist is to develop appropriate techniques and procedures and to follow them rigorously; uniformity is vital and departures from the established norm spell trouble.

A higher temperature speeds up the setting time of concrete, as defined in ASTM C 403-08. Tests on a 1:2 cement–sand mortar[8.3] showed that the initial setting time was approximately halved by a change in the temperature of the concrete from 28 to 46 °C (82 to 115 °F). The effect was similar at water/cement ratios between 0.4 and 0.6, but the actual setting time was shorter the lower the water/cement ratio.[8.3]

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A high ambient temperature causes a higher water demand of the concrete and increases the temperature of the fresh concrete. This results in an increased rate of loss of slump and in a more rapid hydration, which leads to accelerated setting and to a lower long-term strength of concrete (see p. 359). Furthermore, rapid evaporation may cause *plastic shrinkage cracking* and *crazing*, and subsequent cooling of the hardened concrete can introduce tensile stresses. It is generally believed that plastic shrinkage cracking is likely to occur when the rate of evaporation exceeds the rate at which the bleeding water rises to the surface, but it has been observed that cracks also form under a layer of water and merely become apparent on drying.[8.61] Evaporation rate in excess of 1.0 kg/m^2 per hour (0.2 lb/ft^2 per hour) is considered to be critical.[8.14]

Plastic shrinkage cracks can be very deep, ranging in width between 0.1 and 3 mm (0.004 and 0.12 in.), and can be quite short or as long as 1 m (or 3 ft).[8.62] Once developed, they are difficult to

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close permanently.[8.14] A drop in the ambient relative humidity encourages this type of cracking[8.9] so that, in fact, the causes of it appear to be rather complex. According to ACI 305R-91[8.14] the risk of plastic cracking is the same at the following combinations of temperature and relative humidity:

41 °C (105 °F) and 90 per cent
35 °C (95 °F) and 70 per cent
24 °C (75 °F) and 30 per cent.

Wind velocity in excess of 4.5 m/s (10 mph) aggravates the situation;[8.14] wind shields are helpful and so is the provision of sun shades.[8.20]

Another type of cracking on the surface of fresh concrete is caused by differential settlement of fresh concrete due to some obstruction to settlement, such as large particles of aggregate or reinforcing bars. This *plastic settlement cracking* can be avoided by the use of a dry mix, good compaction, and by not allowing too fast a rate of build-up of concrete. Plastic settlement cracking

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can occur also at normal temperatures but, in hot weather, plastic shrinkage cracking and plastic settlement cracking are sometimes confused with one another.

There are some further complications in hot-weather concreting: air-entraining is more difficult, although this can be remedied by using larger quantities of the entraining agent. A related problem is that, if relatively cool concrete is allowed to expand when placed at a higher temperature, the air voids expand and the strength is reduced. This would occur, for instance, with horizontal panels but not with vertical ones in steel moulds where expansion is prevented.[8.64]

Let us now consider the steps which can be taken to avoid or reduce the ill-effects of hot weather. In the past, a maximum air temperature at which concrete could be placed used to be limited. This is not a sensible restriction in countries with very high ambient temperatures. Nevertheless, a limit on the placing temperature of concrete which will be exposed to a humid or ag-

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gressive environment is 30 °C (86 °F). The current British standard dealing with hot weather concreting is BS 8500-1 : 2006. Whenever possible, it is desirable to place the concrete in the coolest part of the day and preferably at a time such that the ambient temperature will rise following the setting of the concrete, that is, after midnight or in the early hours of the morning. It is worth adding that trial batches of the concrete should be made at what is intended to be the placing temperature, and not at some other temperature such as the laboratory temperature of 20 or 25 °C (68 or 77 °F).

There are a number of preventive measures that can be taken. In the first instance, the cement content should be kept as low as possible so that the heat hydration does not unduly aggravate the effects of high ambient temperature. The temperature of the fresh concrete can be lowered by precooling one or more of the ingredients of the mix. A placing temperature of concrete as low as 10

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°C (50 °F) is desirable but may well be impractical.

The temperature T of the freshly mixed concrete can be easily calculated from that of the ingredients, using the expression

$$T = \frac{0.22(T_a W_a + T_c W_c) + T_w W_w}{0.22(W_a + W_c) + W_w}$$

where T denotes temperature in °C or °F, W the mass of ingredient per unit volume of concrete, and the suffixes a, c, w refer to aggregate, cement, and water (both added and in aggregate) respectively. The figure of 0.22 is the approximate ratio of the specific heat of the dry ingredients to that of water, and is applicable to both SI and Imperial (American) systems of units. It may be worth pointing out that, during the night, aggregate and water do not cool as rapidly as the air so that their temperature cannot be assumed to be equal to the air temperature.

The actual temperature of the concrete will be somewhat higher than indicated by the above ex-

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pression due to the mechanical work done in mixing, and will further rise due to the development of the heat of wetting and hydration of cement, as well as due to the heat transfer from the ambient air and formwork. Incidentally, it is important that the formwork should be cooled prior to the placing of concrete. To obtain a better picture, we can say that if the water/cement ratio of a mix is 0.5 and the aggregate/cement ratio is 5.6, then a drop of 1 °C (or 1 °F) in the temperature of fresh concrete can be obtained by lowering the temperature either of the cement by 9 °C (9 °F) or of the water by 3.6 °C (3.6 °F) or of the aggregate by 1.6 °C (1.6 °F). It can be seen that because of its relatively small quantity in the mix the temperature of the cement is not important.

The use of hot cement *per se* is not detrimental to strength but it is preferable not to use cement at temperatures above about 75 °C (170 °F). This statement is of interest because hot cement is sometimes viewed with suspicion and various ill effects have at times been ascribed to its use.

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However, if hot cement is dampened by a small amount of water before it is well dispersed with other solids it may set quickly and form cement balls.

There exist various means of cooling the aggregate and the mix water. Coarse aggregate can be cooled by spraying with chilled water or by inundation. Another method is to use evaporative cooling by blowing air, preferably chilled, through moist aggregate. Fine aggregate can also be cooled by air; freezing by liquid nitrogen has been tried[8.19] but the fine aggregate must be surface dry. Precooling of aggregate in a closed mixer by means of liquefied carbonic acid gas (dry ice), which melts at –78 °C, has also been tried.[8.15]

Mix water can be chilled or can be replaced, usually only partially, by crushed or flaked ice; ice is a highly efficient means of cooling because 1 kg of ice absorbs 334 kJ when melting at 0 °C, which is a quantity of heat four times greater than cooling the water by 20 °C. All the ice must melt

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prior to the end of the mixing operation. Liquid nitrogen, which absorbs 240 kJ/kg when vaporizing at –196 °C, can also be used to chill the water down to 1 °C or can be injected direct into a stationary mixer or a truck mixer immediately prior to discharge. The cost of liquid nitrogen, including the necessary equipment, is high. On the basis of cost per 1 degree drop in temperature of concrete, the use of heat pumps to cool the water is very economical[8.13] but, of course, applicable only at a static mixing plant. A range of cooling techniques is described in ACI 207.4R-93,[8.4] and ACI 305R-91[8.14] contains advice on insulating and painting white the equipment involved in storing the mix ingredients, and also on mixing and transporting concrete.

After placing, concrete should be protected from the sun; otherwise, if a cold night follows, cracking is likely to occur, the extent of cracking being directly related to the temperature difference. In dry weather, wetting concrete and allowing evaporation to take place results in effective

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cooling; there is no cooling by this means when membrane curing is used so that a higher temperature may be reached. Large exposed areas such as roads and airfields are particularly vulnerable.

Proper curing in hot weather may be of shorter duration because an advanced degree of hydration is reached more rapidly than at lower temperatures. The emphasis is on the word 'proper' because, as already mentioned, a higher temperature also promotes more rapid drying of the concrete.[8.60]

The greatest interest in hot-weather concreting is with respect to hot and dry conditions. Generalized information about the behaviour and properties of concrete placed in a hot and continuously humid climate is not available. Data obtained in specific investigations[8.22] show large variations. All we can say is that the absence of drying in the very early life of concrete is tantamount to the provision of moist curing, which is beneficial from the standpoint of gradual gain in strength and reduced drying shrinkage. Nevertheless, the

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initial high temperature has an adverse effect on long-term strength. It is also prudent to assume that plastic shrinkage can occur, depending on the bleeding characteristics of the concrete and on exposure to wind.

Other investigations[8.21,8.59] also indicate that the effects of early high temperature are less detrimental to long-term strength than the absence of moist curing. Great care is required in translating this observation into practice: whereas wet curing is of paramount importance, the harmful consequences of early high temperature are also a reality.

Concreting in cold weather

Before discussing the actual concreting operations, we should consider the action of frost on fresh concrete; the durability of hardened concrete subjected to repeated cycles of freezing and thawing is discussed in Chapter 11.

In Chapter 6, it was stated that hydration of cement occurs even at low temperatures down to

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about –10 °C (14 °F); it is therefore rational to ask: what is then the significance of the temperature at which water freezes? If concrete which has not yet set is allowed to freeze, the action of frost is somewhat similar to that in a saturated soil subject to heaving: the mixing water freezes, with a consequent increase in the overall volume of the concrete. Furthermore, because no water is available for chemical reactions, the setting and hardening of concrete are delayed. It follows from the latter observation that, if concrete freezes immediately after it has been placed, setting will not have taken place, and thus there is no cement paste that can be disrupted by the formation of ice. While the low temperature continues, the process of setting will remain suspended. When, at a later date, thawing takes place, the concrete should be revibrated, and it will then set and harden without loss of strength. However, because of the expansion of the mixing water on freezing, a lack of revibration would allow the concrete to set with a large volume of pores present, and consequently the strength of the con-

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crete would be very low. Revibration on thawing would produce a satisfactory concrete, but such a procedure is not recommended except when unavoidable.

If freezing takes place after the concrete has set but before it has developed an appreciable strength, the expansion associated with the formation of ice causes disruption and an irreparable loss of strength. If, however, the concrete has acquired a sufficient strength, it can resist the freezing temperature without damage, not only by virtue of the higher resistance to the pressure of the ice but also because a large part of the mixing water will have become combined with the cement or located in small pores, and would thus not be able to freeze. It is difficult, however, to establish when this situation has been reached, because setting and hardening of cement depend on the temperature during the period *preceding* the actual advent of freezing. According to ACI 306R-88,[8.55] when concrete has reached a compressive strength of about 3.5 MPa (500 psi), the

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degree of saturation has dropped below the critical value, provided no external water has ingressed into the concrete. At that stage, the concrete is capable of withstanding *one* cycle of freezing and thawing. Higher values of strength are recommended in some other countries, but no reliable data are available on the strength at which concrete can successfully resist temperatures below 0 °C (32 °F).

Generally, the more advanced the hydration of cement and the higher the strength of concrete the less vulnerable it is to frost. This situation can be expressed by means of the minimum age of concrete stored at a given temperature when exposure to frost will not cause damage; typical values (averaged from various sources[8.105,8.106]) are given in Table 8.7. Figure 8.26 shows the influence of the age at which first freezing occurs on the expansion of concrete: the considerable decrease in the magnitude of expansion of concrete allowed to harden for about 24 hours is noticeable, and

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protecting concrete from frost during that period is clearly highly advisable.

Table 8.7. Age of Concrete at Which Exposure to Frost does not Cause Damage

Type of cement	*Water/ cement ratio*	*Age (hours) at exposure when preceding curing temperature was:*			
		5 °C (41 °F)	*10 °C (50 °F)*	*15 °C (59 °F)*	*20 °C (68 °F)*
Ordinary Portland	0.4	35	25	15	12
	0.5	50	35	25	17
	0.6	70	45	35	25
Rapid-hardening Portland	0.4	20	15	10	7
	0.5	30	20	15	10
	0.6	40	30	20	15

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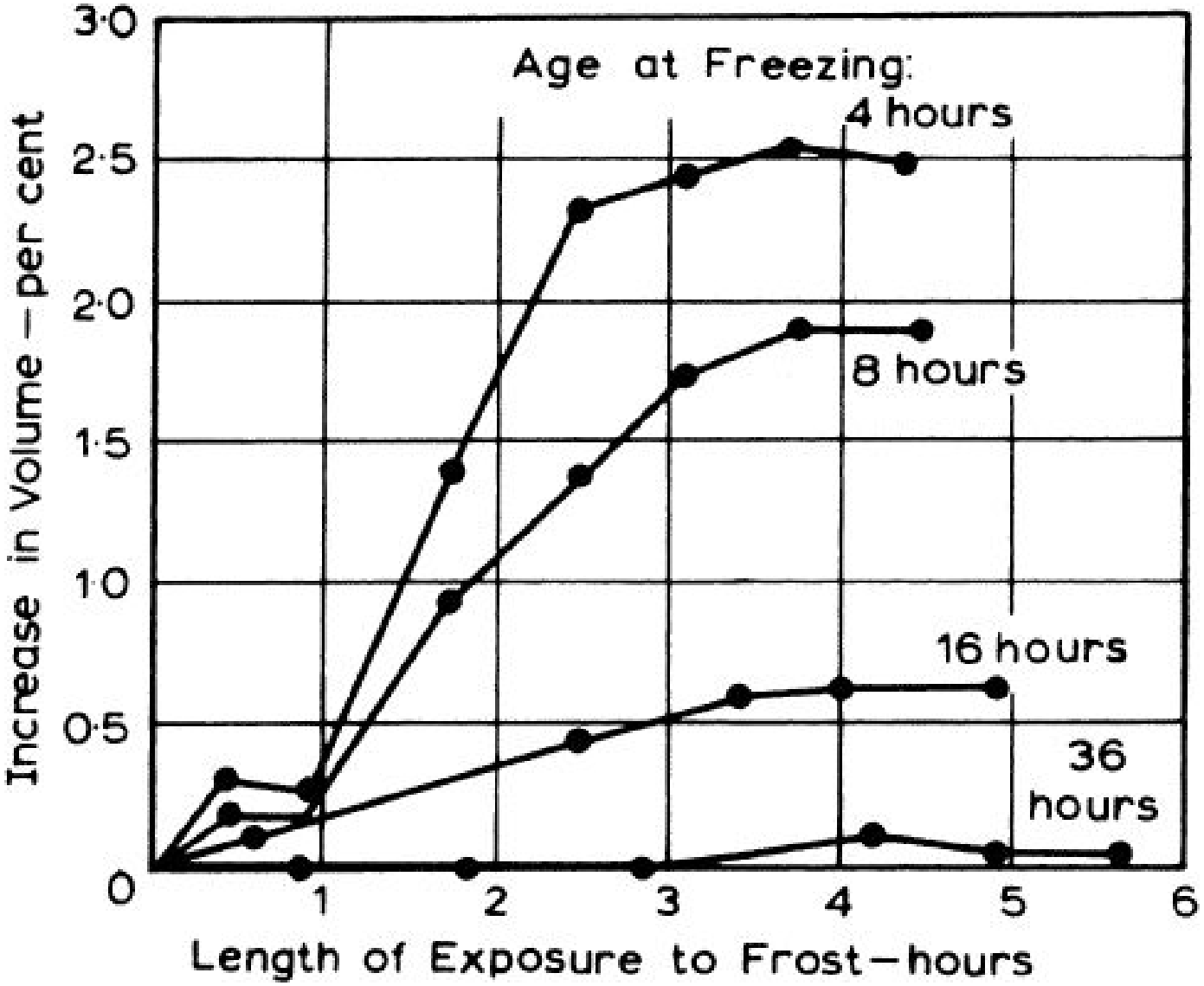


Fig. 8.26. Increase in volume of concrete during prolonged freezing as a function of age when freezing starts[8.68]

The resistance to alternating freezing and thawing also depends on the age of the concrete when the first cycle is applied, but this type of exposure is more severe than prolonged freezing without periods of thaw, and several cycles can

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cause damage even to concrete cured at 20 °C (68 °F) for 24 hours.[8.68] It may be noted that there is no direct relation between the frost resistance of young concrete and the durability of mature concrete subjected to numerous cycles of freezing and thawing,[8.69] a topic which is considered in Chapter 11. Figure 11.2 in that chapter shows a lack of expansion on the first occasion of freezing when this occurs at the age of more than 1 day: this supports the view expressed in ACI 306R-88 to the effect that most "well-proportioned" concretes stored at 10 °C (50 °F) reach the strength of 3.5 MPa (500 psi) during the second day.[8.55]

Concreting operations

When the air temperature is continuously below 0 °C (32 °F), the weather can be unarguably described as cold. The situation is less clear-cut when there is a large diurnal variation in temperature. For convenience, the definition of 'cold weather' used by ACI 306R-88[8.55] can be used. This can be paraphrased by saying that cold

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weather obtains when two conditions exist: when the average of the maximum and minimum air temperatures recorded on 3 consecutive days is less than 5 °C (40 °F) *as well as* when the air temperature during at least 12 hours in any 24-hour period is 10 °C (50 °F) or lower.

Under such circumstances, normal weight concrete should not be placed unless *its* temperature is at least 13 °C (55 °F) for thin sections (300 mm (12 in.)) or at least 5 °C (40 °F) when the minimum dimension of the concrete element is at least 1.8 m (72 in.).[8.55] Lightweight aggregate concrete, which has a lower thermal conductivity, can be somewhat cooler when placed. Such concrete also has a lower specific heat so that a given heat of hydration of cement more effectively keeps the lightweight aggregate concrete from freezing than is the case with normal weight aggregate concrete.

Advantage can also be taken of using rapid-hardening cement and rich mixes with a low water/cement ratio, and of the use of cement with

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a high rate of heat development, i.e. having high C_3S and C_3A contents. Accelerators can be used but chlorides must be avoided if steel is present in the concrete.

To achieve the minimum temperatures cited earlier, when aggregate, water and air are cold, the mix ingredients can be heated. Water can be heated easily, but it is inadvisable to exceed a temperature of 60 to 80 °C (140 to 176 °F) because flash set of the cement may result; the likelihood of this happening depends on the difference between the temperatures of water and cement. It is important to prevent the cement from coming into contact with the hot water, and for this reason the order of feeding the mix ingredients into the mixer must be suitably arranged.

If heating the water does not sufficiently raise the temperature of the concrete, the aggregate may also be heated. This is done preferably by passing steam through coils rather than by the use of live steam because the latter methods leads to a variable moisture content of the aggregate. Heat-

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ing the aggregate above 52 °C (125 °F) is inadvisable.[8.63] At the other extreme, it is important that the aggregate does not contain ice as the heat required to melt it would greatly reduce the temperature of the concrete.

The temperature of the mix ingredients must be controlled, and the temperature of the resulting concrete should be calculated in advance (see p. 401). This calculation should allow for the loss of heat during the transport of the concrete. The objective is to ensure that the temperature of the concrete is high enough to prevent premature freezing but also to make sure that setting does not occur at too high a temperature. This would adversely affect the development of strength of the concrete (see p. 361). In addition, a high temperature of fresh concrete lowers its workability and may lead to high thermal contraction.

It is thus desirable for the concrete to set at, say, 7 to 21 °C (45 to 70 °F). The temperature of 7 °C (45 °F) applies when the air temperature is no lower than –1 °C (30 °F) and the concrete

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element is thick; the value of 21 °C (70 °F) applies when the air is cooler than –18 °C (0 °F) and the concrete section is less than 300 mm (12 in.) thick.

In some countries,[8.12,8.37] the entire concrete mix is heated to between 40 and 60 °C (104 and 140 °F). Such temperatures have an adverse effect on workability and on the long-term strength but these may be balanced by economic considerations: rapid re-use of formwork and no need for post-placing heating. Also, the high initial temperature speeds up the process of hydration, so that 'cost-free' heat is generated.

Placing against frozen ground should not be permitted and the formwork should, if possible, be pre-heated.

Following placement, the concrete must be protected from freezing for at least 24 hours. Drying of the surface of the concrete should be prevented, especially when the concrete is much warmer than the ambient air. However, no active wet curing should be applied so that the concrete

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becomes less than saturated. While this may seem to be counter to the usual recommendations about wet curing, we should note that cold air (below 10 °C (50 °F)) does not cause excessive drying.

Various types of insulation of concrete placed in cold weather are described in ACI 306R-88 (Reapproved 2007).[8.55] What is important is to remove the insulation in a manner such as to avoid a sudden change in temperature at the surface of the concrete and the development of steep temperature gradients within the concrete element. ACI 306R-88 also gives information about protecting and heating concrete in cold weather. It needs to be pointed out that the means of heating should be such that the concrete does not dry out rapidly, that no part of it is heated excessively, and that no high concentration of CO_2 in the atmosphere results. This last point means that combustion heaters, unless vented, should not be used in enclosed spaces.

An alternative to placing concrete under conditions such that normal mix water cannot freeze

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is to depress the freezing point of the mix water well below 0 °C (32 °F). This can be achieved by the use of *anti-freeze admixtures*. Potassium carbonate (potash) was one of the first such admixtures to be used.[8.96] More recent developments include the use of calcium nitrite and sodium nitrite; it may be recalled that these inorganic salts act as accelerators (see p. 247) and that they are non-corrosive with respect to steel. Mixes containing nitrites were found to produce concrete with significant strengths at temperatures down to –10 °C (14 °F).[8.17] As is sometimes the case with admixtures, anti-freeze admixtures of undisclosed composition are claimed[8.16] to result in air-entrained concrete mixes which gain strength at temperatures of –7 °C (20 °F) and even down to –19 °C (–2 °F); in the latter case, however, the solids content of the admixtures is 47 per cent so that the provision of an adequate amount of mix water may not be possible. Practical acceptance of admixtures of this type is yet to come.

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Without using anti-freeze admixtures, it is possible to place air-entrained concrete at 0 °C (32 °F) because, as soon as hydration has begun, the freezing point of the pore water is depressed so that no frost action takes place above about –2 °C (28 °F). Development of strength of concretes with water/cement ratios of 0.35 and 0.45, cast at 0 °C (32 °F) and stored in sea water in the laboratory at 0 °C (32 °F), was determined by Gardner.[8.18] He reported long-term strengths, both compressive and tensile, comparable with those of concrete stored at 16 °C (61 °F). This latter finding is similar to that of Aïtcin.[8.23] Both these investigations indicate that keeping concrete in sea water at 0 °C (32 °F) is not harmful. This may not be the case with storage in air at the same temperature. In any event, under conditions of natural exposure, the absence of a drop in temperature below 0 °C (32 °F) cannot be guaranteed.

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Chapter 9. Elasticity, shrinkage, and creep

Much of the discussion in the preceding chapters referred to the strength of concrete, which is of significant importance in the design of concrete structures. However, with any stress, there is always associated a strain, and vice versa. Strain can arise also from causes other than applied stress. The relation between stress and strain over their full range is of vital interest in structural design. The topic of strain and, more generally, of the different types of deformation of concrete is the subject matter of this chapter.

Like many other structural materials, concrete is, to a certain degree, elastic. A material is said to be perfectly elastic if strain appears and disappears immediately on application and removal of stress. This definition does not imply a linear stress–strain relation: elastic behaviour coupled

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with a non-linear stress–strain relation is exhibited, for instance, by glass and some rocks.

When concrete is subjected to sustained loading, strain increases with time, i.e. concrete exhibits creep. In addition, whether subjected to load or not, concrete contracts on drying, undergoing shrinkage. The magnitudes of shrinkage and creep are of the same order as elastic strain under the usual range of stresses, so that the various types of strain must be, at all times, taken into account.

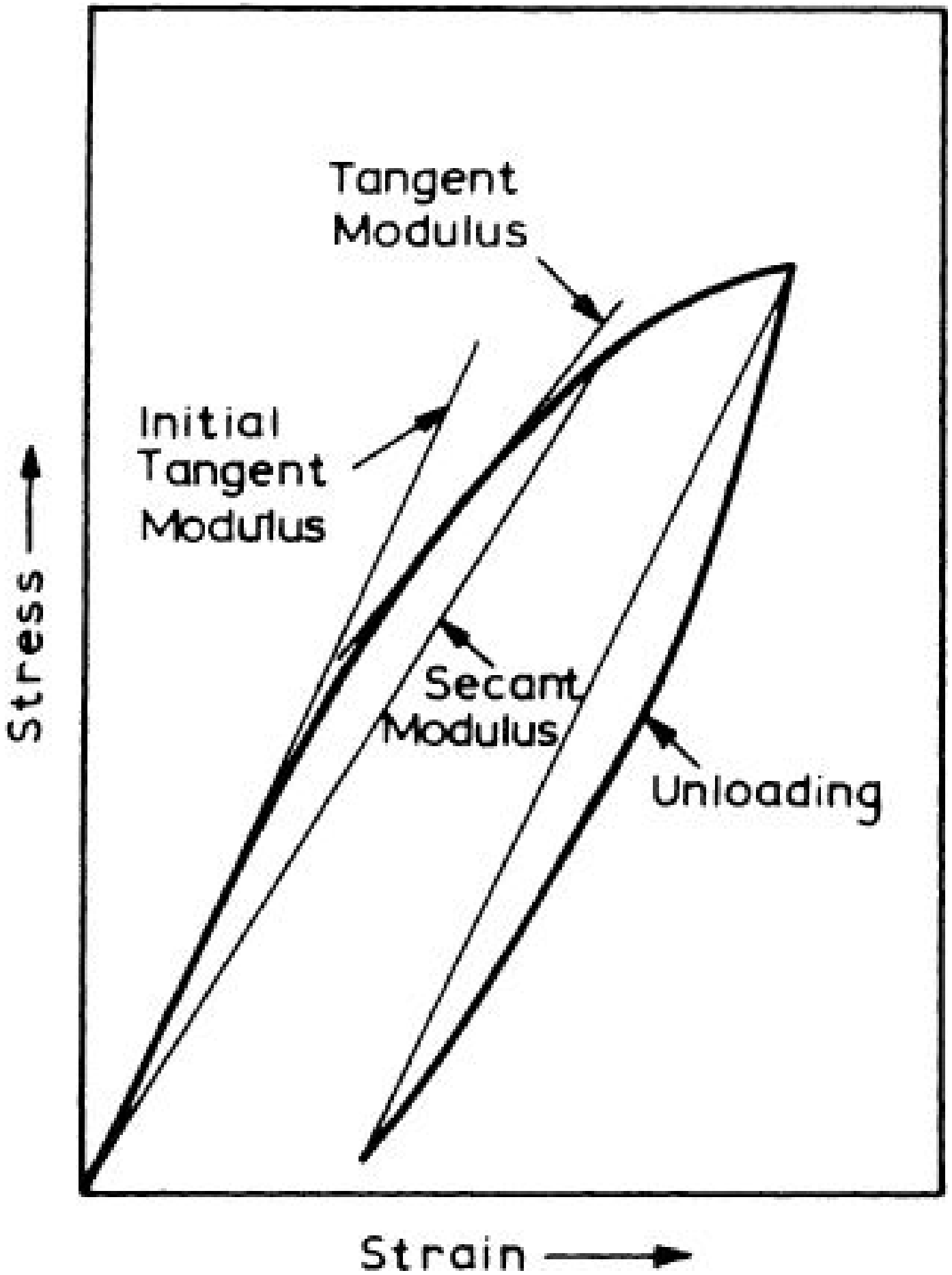
Stress–strain relation and modulus of elasticity

Figure 9.1 shows a diagrammatic representation of the stress–strain relation for a concrete specimen loaded and unloaded in compression or tension up to a stress well below the ultimate strength. In compression tests, a small concave-up part of the curve at the beginning of loading is sometimes encountered; this is due to the closing of pre-existing fine shrinkage cracks. From Fig.

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9.1, it can be seen that the term Young's modulus of elasticity can, strictly speaking, be applied only to the straight part of the stress–strain curve, or, when no straight portion is present, to the tangent to the curve at the origin. This is the *initial tangent modulus*, but it is of limited practical importance. It is possible to find a tangent modulus at any point on the stress–strain curve, but this modulus applies only to very small changes in load above or below the load at which the tangent modulus is considered.

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Fig. 9.1. Diagrammatic representation of the stress–strain relation for concrete

The magnitude of the observed strains and the curvature of the stress–strain relation depend, at least in part, on the rate of application of stress. When the load is applied extremely rapidly, say, in less than 0.01 second, recorded strains are greatly reduced, and the curvature of the stress–strain curve becomes extremely small. An increase in loading time from 5 seconds to about 2 minutes can increase the strain by up to 15 per cent, but within the range of 2 to 10 (or even 20) minutes – a time normally required to test a specimen in an ordinary testing machine – the increase in strain is very small. The relation between the rate of strain and strength, discussed on p. 621, may be of relevance.

The increase in strain while the load, or part of it, is acting is due to creep of concrete, but the dependence of instantaneous strain on the speed of loading makes the demarcation between elastic and creep strains difficult. For practical purposes,

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an arbitrary distinction is made: the deformation occurring during loading is considered elastic, and the subsequent increase in strain is regarded as creep. The modulus of elasticity satisfying this requirement is the *secant modulus* of Fig. 9.1, also known as the *chord modulus*. The secant modulus is a *static modulus* because it is determined from an experimental stress–strain relation on a test cylinder, in contradistinction to the dynamic modulus, considered on p. 421.

Because the secant modulus decreases with an increase in stress, the stress at which the modulus has been determined must always be stated. For comparative purposes, the maximum stress applied is chosen as a fixed proportion of the ultimate strength. This proportion is prescribed as 33 per cent in BS 1881-121 : 1983, and as 40 per cent in ASTM C 469-02. To eliminate creep, and also to achieve seating of the gauges, at least two cycles of pre-loading to the maximum stress are required. The minimum stress must be such that the test cylinder does not move. This minimum

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is specified by BS 1881-121 : 1983 as 0.5 MPa; ASTM C 469-02 specifies a minimum strain of 50×10^{-6}. The stress–strain curve on the third or fourth loading exhibits only a small curvature.

It is interesting to note that the two components of concrete, that is, hydrated cement paste and aggregate, when individually subjected to load, exhibit a sensibly linear stress–strain relation (Fig. 9.2), although some suggestions about the non-linearity of the stress–strain relation of the hydrated cement paste have been made.[9.100] The reason for the curved relation in the composite material – concrete – lies in the presence of interfaces between the cement paste and the aggregate and in the development of bond microcracks at those interfaces.[9.42] The progressive development of microcracking was confirmed by neutron radiography.[9.62]

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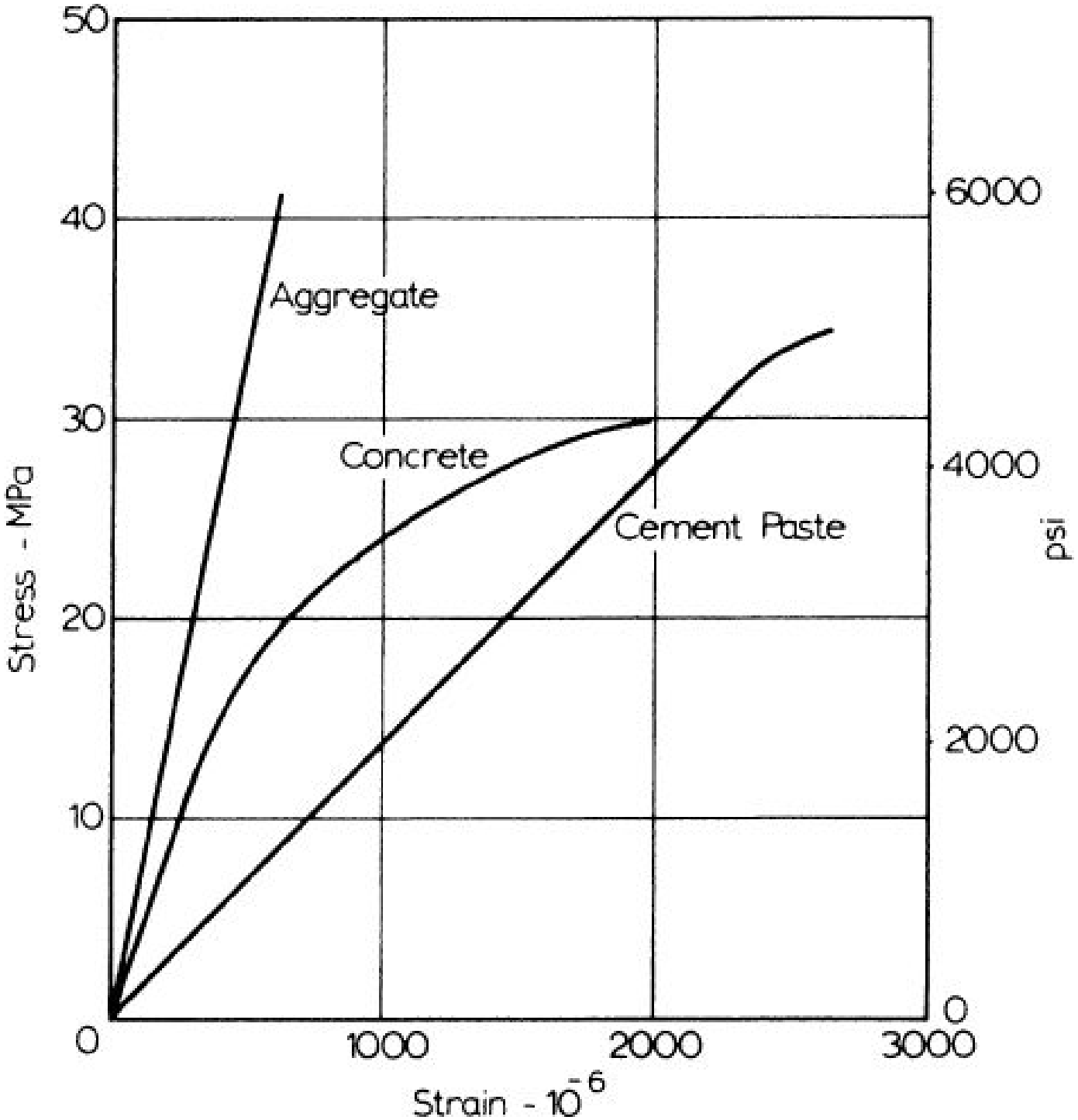


Fig. 9.2. Stress–strain relations for cement paste, aggregate, and concrete

The development of microcracking means that the stored strain energy is transformed into the

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surface energy of the new crack faces. Because the cracks develop progressively at interfaces making varying angles with the applied load, and respond to the local stress, there is a progressive increase in local stress intensity and in the magnitude of strain. In other words, a consequence of the development of the cracks is a reduction in the effective area resisting the applied load, so that the local stress is larger than the nominal stress based on the total cross-section of the specimen. These changes mean that the strain increases at a faster rate than the nominal applied stress, and so the stress–strain curve continues to bend over, with an apparent pseudo-plastic behaviour.[9.43]

When the applied stress increases beyond approximately 70 per cent of the ultimate strength, mortar cracking (connecting the bond cracks) develops (see p. 300) and the stress–strain curve bends over at an increasing rate. The development of a continuous crack system reduces the number of load-carrying paths[9.65] and, eventu-

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ally, the ultimate strength of the specimen is reached. This is the peak of the stress–strain curve.

If the testing machine allows a reduction in the applied load, the strain will continue to increase with a decrease in the nominal applied stress. This is the *post-peak* part of the stress–strain curve which represents strain softening of concrete. However, the observed descending part of the stress–strain curve is not a *material* property[9.65] but is affected by test conditions. The main influencing factors are the stiffness of the testing machine in relation to the stiffness of the test specimen and the rate of strain.[9.67] A typical complete stress–strain curve is shown in Fig. 9.3.[9.36]

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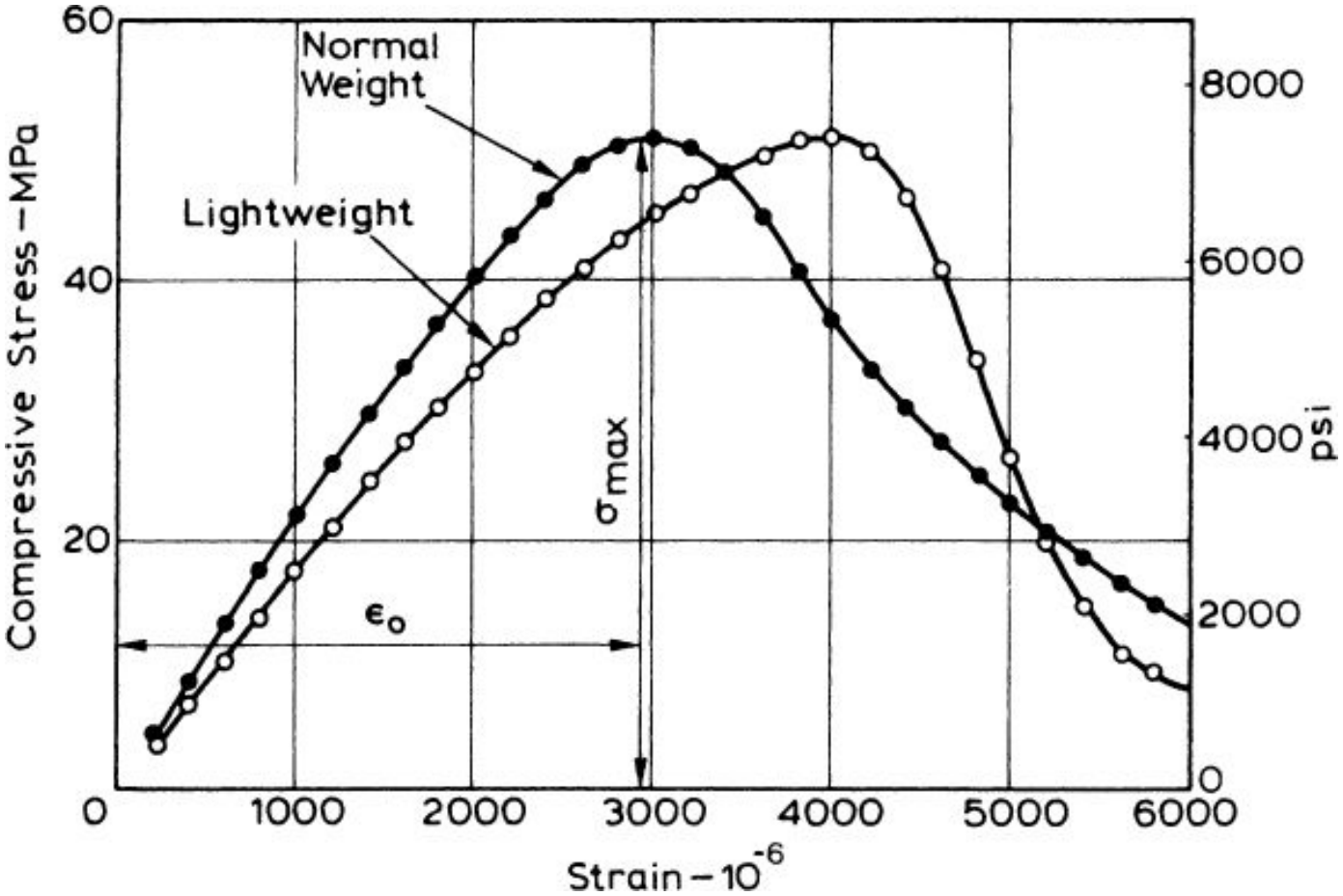


Fig. 9.3. Stress–strain relation of concretes tested in compression at a constant rate of strain[9.36]

It can be noted that, if the stress–strain curve ended abruptly at the peak, the material would be classified as brittle. The less steep the descending part of the stress–strain curve the more ductile the behaviour. If the slope beyond the peak were zero, the material would be said to be perfectly plastic.

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In structural design of reinforced concrete, the entire stress–strain curve, often in idealized form, must be considered. For this reason, the behaviour of concrete which has a very high strength is of especial interest. Such concrete develops a smaller amount of cracking than normal-strength concrete during all stages of loading;[9.66] in consequence, the ascending part of the stress–strain curve is steeper and linear up to a very high proportion of the ultimate strength. The descending part of the curve is also very steep (see Fig. 9.4) so that high strength concrete is more brittle than ordinary concrete, and indeed explosive failure of a local part of specimens of high strength concrete tested in compression has often been encountered. However, the apparent brittleness of high strength concrete is not necessarily reflected in the behaviour of reinforced concrete members made with such concrete.[9.63,9.64]

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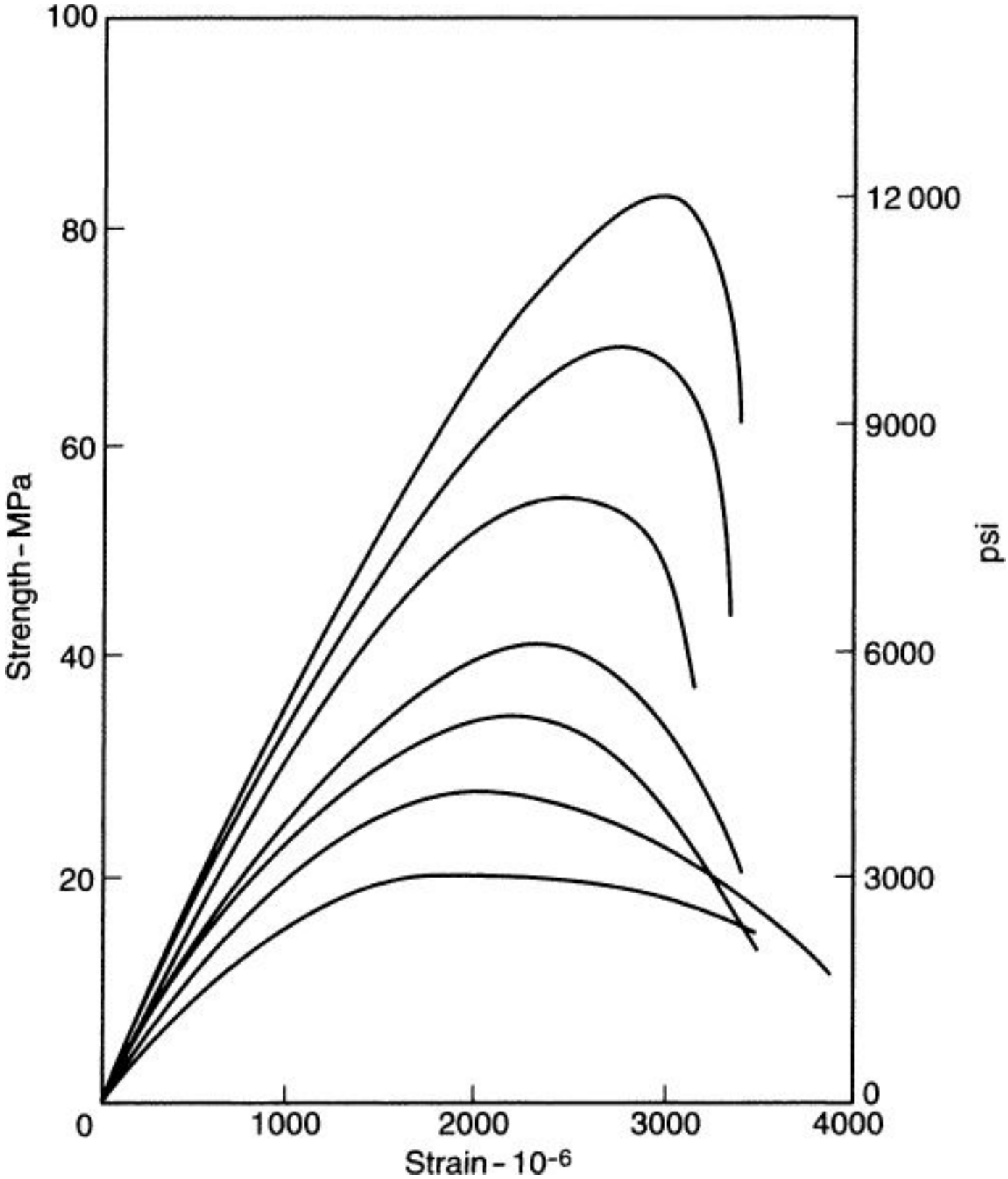


Fig. 9.4. Examples of stress–strain relation in compression for concrete cylinders with compressive strength up to 85 MPa

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The behaviour of high-strength concrete is of interest also with respect to the strain at various stress levels. If the stress considered, for example the stress in service, is expressed as a fraction of the ultimate strength, referred to as the *stress/strength ratio*, then the following observations can be made. At the same stress/strength ratio, the stronger the concrete the larger the strain. At the maximum stress, that is at the stress corresponding to the ultimate strength, in a 100 MPa (15 000 psi) concrete, the strain is typically 3×10^{-3} to 4×10^{-3}; in a 20 MPa (3000 psi) concrete, the strain is about 2×10^{-3}. However, under the same stress, regardless of strength, stronger concrete exhibits a lower strain. It follows that high-strength concrete has a higher modulus of elasticity, as seen in Fig. 9.4.

Parenthetically, we can observe that this behaviour is in contrast to that of different grades of steel, possibly because the strength of hydrated cement paste is governed by the gel/space ratio, which can be expected to affect also the stiffness

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of the cementitious material. On the other hand, the strength of steel is related to the structure and boundaries of crystals but not to voids, so that the stiffness of the material is unaffected by its strength.

Lightweight aggregate concrete exhibits a steeper descending part of the stress–strain curve,[9.36] (see Fig. 9.3) that is, it has a somewhat more brittle behaviour than normal weight concrete.

The stress–strain curve in tension is similar in shape to that in compression (see Fig. 9.5) but a special testing machine is necessary.[9.61] In direct tension, the development of cracks has the effect both of reducing the effective area resisting stress and of increasing the contribution of cracks to the overall strain. This may be the reason why the departure from linearity of the stress–strain relation in tension occurs at a slightly lower stress/strength ratio than in compression.[9.34]

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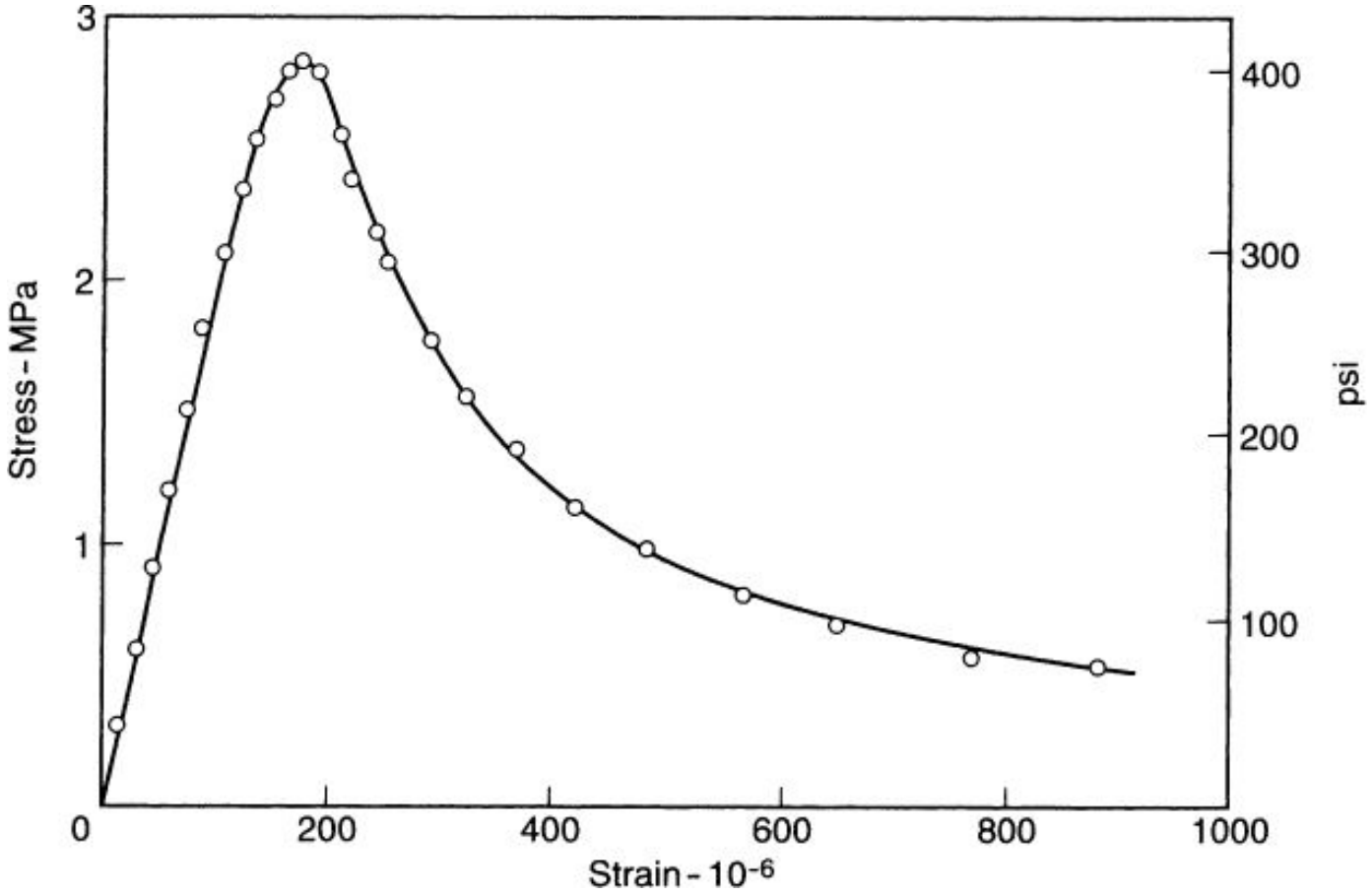


Fig. 9.5. Examples of the stress–strain relation in direct tension (based on ref. 9.61)

Expressions for stress–strain curve

Because the precise shape of the entire stress–strain curve for concrete is not a property of the material *per se* but depends on the test arrangements, there is little of fundamental importance in formulating an equation for the stress–strain relation. This is not to deny the usefulness of such an equation in structural analysis.

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Numerous attempts to develop equations have been made, but probably the most successful equation was suggested by Desayi and Krishnan:[9.44]

$$\sigma = \frac{E\varepsilon}{1 + \left(\frac{\varepsilon}{\varepsilon_0}\right)^2}$$

where ε = strain

σ = stress

ε_0 = strain at maximum stress, and

E = initial tangent modulus, assumed to be twice the secant modulus at maximum stress σ_{max}, i.e.

$$E = \frac{2\sigma_{max}}{\varepsilon_0}.$$

The last assumption is questionable because both σ_{max} and ε_0 are strongly affected by test conditions, and a more general form of the equation

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which is not constrained by this assumption has been developed by Carreira and Chu.[9.67]

Expressions for modulus of elasticity

There is no doubt that the modulus of elasticity increases with an increase in the compressive strength of concrete, but there is no agreement on the precise form of the relationship. This is not surprising, given the fact that the modulus of elasticity of concrete is affected by the modulus of elasticity of the aggregate and by the volumetric proportion of aggregate in the concrete. The former is rarely known so that some expressions, for example that of ACI 318-02[9.98] allow for the modulus of elasticity of aggregate by a coefficient which is a function of the density of the concrete, usually density raised to power 1.5.

All that can be said reliably is that the increase in the modulus of elasticity of concrete is progressively lower than the increase in compressive strength. According to ACI 318-02[9.98] the modulus is proportional to the strength raised to power

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0.5. The expression for the secant modulus of elasticity of concrete, E_c, in pounds per square inch, recommended by ACI 318-02[9.98] for structural calculations, applicable to normal weight concrete, is

$$E_c = 57\,000(f'_c)^{0.5}$$

where f'_c is the compressive strength of standard test cylinders in pounds per square inch. When E_c is expressed in GPa and f'_c in MPa, the expression is

$$E_c = 4.73(f'_c)^{0.5}.$$

Some other expressions use the power index of 0.33, instead of 0.5, and also add a constant term to the right-hand side of the equation.

For concretes with strengths up to 83 MPa (12 000 psi) ACI 363R-92[9.99] quotes

$$E_c = 3.32(f'_c)^{0.5} + 6.9$$

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where E_c is expressed in GPa and f'_c in MPa. In the range of strengths between 80 and 140 MPa (12 000 and 20 000 psi) Kakizaki *et al.*[9.95] found that the modulus of elasticity, E_c, is approximately related to strength f'_c, by the expression

$$E_c = 3.65(f'_c)^{0.5}$$

using the same units of measurement as above. The modulus was found not to be affected by curing but was influenced by the modulus of elasticity of the coarse aggregate in the concrete. This dependence is a consequence of the two-phase nature of concrete.[9.84] The quality of the bond between the two phases is of importance and may affect the value of the modulus of elasticity of concrete when the bond is particularly strong, as is the case in high performance concrete (see p. 678). Furthermore, because such concrete is made with high strength aggregate, which is likely to have a high modulus of elasticity, high performance concrete tends to have a higher

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modulus of elasticity than would be expected from an extrapolation of expressions for ordinary concrete.

When the density of concrete, ρ, is between 145 and 155 lb/ft^3 (taken to be the range for normal weight concrete) and is expressed in pounds per cubic foot, the modulus of elasticity is given by ACI 318-02[9.98] as

$$E_c = 33\rho^{1.5}(f'_c)^{0.5}.$$

In SI units, this expression becomes

$$E_c = 43\rho^{1.5}(f'_c)^{0.5} \times 10^{-6}.$$

The use of the power coefficient of 1.5 applied to the density of concrete may not be correct. According to Lydon and Balendran,[9.70] the modulus of elasticity of aggregate is proportional to the square of its density. Whatever the value of the power index, the argument is that, at a constant aggregate content, the density of concrete

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increases with the increase in the density of aggregate.

The two-phase nature of concrete also means that the volumetric proportions of aggregate and of hydrated cement paste affect the value of the modulus of elasticity at a given strength of concrete. Because normal weight aggregate has a higher modulus of elasticity than hydrated cement paste, a higher content of a given aggregate results in a higher modulus of elasticity of concrete of a given compressive strength.

Lightweight aggregate has a lower density than hydrated cement paste and influences the modulus of elasticity of concrete accordingly. The consideration of density of the concrete in the expression of ACI 318-02[9.98] means that lightweight aggregate concrete can be covered by the same expression. We can note that, because the modulus of elasticity of lightweight aggregate differs little from the modulus of the hardened cement paste, mix proportions do not

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affect the modulus of elasticity of lightweight aggregate concrete.[9.7]

For concrete cast and stored at 0 °C (32 °F) the rate of change in the modulus of elasticity with an increase in the strength of concrete was found to be somewhat steeper than at room temperature[9.59] but the difference does not appear to be of importance.

So far we have considered the modulus of elasticity in compression. Few data are available for the modulus of elasticity of concrete in tension, which can be determined in direct tension or from measurement of deflection of flexure specimens; where necessary, a correction for shear should be applied.[9.5] The best assumption which can be made about the modulus of elasticity in tension is that it is equal to the modulus in compression. This was broadly established by tests,[9.34,9.70] and can also be seen from a comparison of Figs 9.4 and 9.5.

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The modulus of elasticity in shear (modulus of rigidity) is not normally determined by direct measurement.

Curing conditions *per se* are not believed to affect the modulus of elasticity other than through the influence of curing on strength. Some reports to the contrary[9.69] may possibly be explained by the fact that the strength of standard test specimens was considered rather than the strength of the actual concrete. Furthermore, it is necessary to distinguish between the influence of curing on the modulus of elasticity, which also affects strength, on the one hand and, on the other, the influence of the moisture condition during the test. The effects of the latter on the modulus of elasticity and on strength need not be the same; this is discussed on p. 602.

Dynamic modulus of elasticity

The preceding section dealt exclusively with the static modulus of elasticity, which gives the strain response to an applied stress of known intensity.

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There exists another type of modulus, known as the dynamic modulus, which is determined by means of vibration of a concrete specimen, only a negligible stress being applied. The procedure for determining the dynamic modulus of elasticity is described on p. 636.

Because of the absence of a significant applied stress, no microcracking is induced in the concrete and there is no creep. In consequence, the dynamic modulus refers to almost purely elastic effects. For this reason, the dynamic modulus is considered to be approximately equal to the initial tangent modulus determined in the static test and is, therefore, appreciably higher than the secant modulus which is determined by application of load to a concrete specimen. This view has, however, been challenged,[9.68] and it has to be recognized that the heterogeneity of concrete affects the two moduli in different ways.[9.1] It cannot, therefore, be expected that there exists between the two moduli a single relation, based on physical behaviour.

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The ratio of the static modulus of elasticity to the dynamic modulus, which is always smaller than unity, is higher the higher the strength of concrete[9.9] and, probably for this reason, increases with age.[9.1] This variable ratio of the moduli means that there is no simple conversion of the value of the dynamic modulus, E_d, which is easy to determine, into an estimate of the static modulus, E_c, the knowledge of which is required in structural design. Nevertheless, various empirical relations, valid over a limited range, have been developed. The simplest of these, proposed by Lydon and Balendran,[9.70] is

$$E_c = 0.83E_d.$$

An expression which used to be included in the British code for design of concrete structures, BS CP 110:1972, is

$$E_c = 1.25E_d - 19$$

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both moduli being expressed in GPa. This expression does not apply to concretes containing more than 500 kg of cement per cubic metre of concrete (850 lb/yd^3) or to lightweight aggregate concrete. For the latter, the following expression was suggested:[9.39]

$$E_c = 1.04E_d - 4.1.$$

For both lightweight and normal concretes, Popovics[9.57] suggested that the relation between the static and dynamic moduli is a function of density of the concrete, just as is the case with the relation between the static modulus and strength, namely,

$$E_c = kE_d^{1.4}\rho^{-1}$$

where ρ is the density of concrete and k is a constant dependent on the units of measurement.

Whatever the relation between the moduli, it is thought to be unaffected by air entrainment,

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method of curing, condition at test, or the type of cement used.[9.11]

The dynamic modulus of elasticity is of considerable value in studying changes in a single test specimen, for example, in consequence of chemical attack.

Poisson's ratio

When a uniaxial load is applied to a concrete specimen it produces a longitudinal strain in the direction of the applied load and, at the same time, a lateral strain of opposite sign. The ratio of the lateral strain to the longitudinal strain is called Poisson's ratio; the sign of the ratio is ignored. Normally, we are interested in the consequences of an applied compression so that the lateral strain is tensile, but the situation is analogous when a tensile load is applied.

For an isotropic and linear-elastic material, Poisson's ratio is constant but, in concrete, Poisson's ratio may be influenced by specific conditions. However, for stresses for which the relation

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between the applied stress and the longitudinal strain is linear, the value of Poisson's ratio for concrete is approximately constant. Depending on the properties of the aggregate used, Poisson's ratio of concrete lies generally in the range of 0.15 to 0.22 when determined from strain measurements under a compressive load. The value of Poisson's ratio under tensile load appears to be the same as in compression.[9.70]

No systematic data on the influence of various factors on Poisson's ratio are available. Lightweight aggregate concrete has been reported to have Poisson's ratio at the lower end of the range.[9.70] The value of Poisson's ratio has been reported[9.94] not to be affected by the increase in strength with age or by the richness of the mix. The latter observation needs to be confirmed because the elastic properties of coarse aggregate can be expected to influence the elastic behaviour of concrete. No generalizations about Poisson's ratio can, therefore, be made, but this lack of information is not critical in view of the fact that,

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for the majority of concretes, the range of values is small: 0.17 to 0.20.

Tests on saturated mortar have shown that the value of Poisson's ratio is higher at higher rates of strain; for instance, it was found[9.60] to increase from 0.20 at a strain rate of 3×10^{-6} per second to 0.27 at a strain rate of 0.15 per second. This effect may not be generally valid.

Figure 9.6 shows a typical plot of longitudinal strain and lateral strain under a steadily and rapidly increasing axial compressive load applied to a cylinder. In addition, the volumetric strain is plotted. It can be seen that, above a certain stress, Poisson's ratio increases rapidly; this is caused by extensive vertical cracking so that, in fact, we are dealing with an *apparent* Poisson's ratio. Under a further increase in stress, the rate of change in volumetric strain changes sign; further on, Poisson's ratio exceeds the value of 0.5, and the volumetric strain becomes tensile. The concrete is no longer a truly continuous body; this is the stage of collapse (cf. p. 294).

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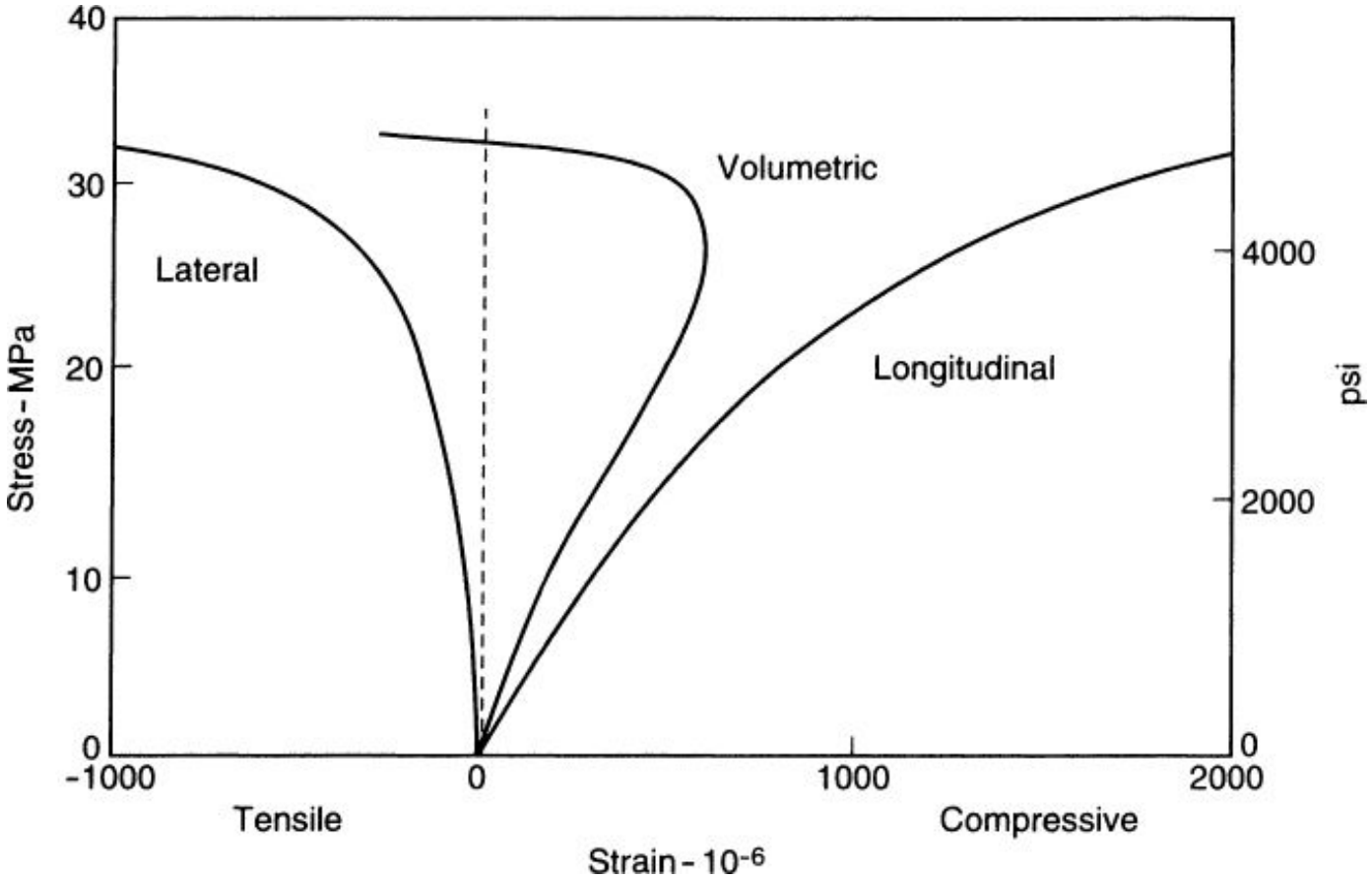


Fig. 9.6. Longitudinal, lateral, and volumetric strains in a concrete cylinder under an increasing stress

It is possible also to determine Poisson's ratio dynamically. The physical situation in such a test is distinct from that under static loading as is the case in the determination of the dynamic modulus of elasticity (see p. 634). For this reason, the value of Poisson's ratio determined dynamically is higher than obtained from static tests; an average value[9.5] is about 0.24.

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The dynamic method of determining Poisson's ratio requires the measurement of pulse velocity, V, and also the fundamental resonant frequency, n, of longitudinal vibration of a beam of length L (see p. 636). Poisson's ratio μ can then be calculated from the expression[9.12]

$$\left(\frac{V}{2nL}\right)^2 = \frac{1-\mu}{(1+\mu)(1-2\mu)}$$

since $E_d/\rho = (2nL)^2$, where ρ = density of concrete.

Poisson's ratio can also be found from the dynamic modulus of elasticity E_d, as determined in longitudinal or transverse mode of vibration (see p. 634), and the modulus of rigidity G, using the expression

$$\mu = \frac{E_d}{2G} - 1.$$

The value of G is normally determined from the resonant frequency of torsional vibration (see p.

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634). Values of μ obtained by this method are intermediate between those from direct static measurements and those from dynamic tests.

Under sustained stress, the ratio of the lateral strain to the longitudinal strain can be called *creep Poisson's ratio*. Data on this ratio are scarce. At low stresses, creep Poisson's ratio is unaffected by the level of stress, indicating that the longitudinal and lateral deformations due to creep are in the same ratio as the corresponding elastic deformations. This means that the volume of concrete decreases with the progress of creep. Above a stress/strength ratio of about 0.5, creep Poisson's ratio increases significantly and progressively with the increase in sustained stress.[9.93] At a sustained stress/strength ratio in excess of 0.8 to 0.9, creep Poisson's ratio exceeds 0.5 and, with time under sustained stress, failure occurs[9.102] (see p. 456).

Under sustained multiaxial compression,[9.45] the creep Poisson's ratio is smaller: 0.09 to 0.17.

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Early volume changes

When water moves out of a porous body which is not fully rigid, contraction takes place. In concrete, from its fresh state to later in life, such movement of water generally occurs. The various stages of water movement and its consequences will now be considered.

When discussing the progress of hydration of cement, the resulting changes in volume were mentioned. The chief of these is the reduction in the volume of the system cement-plus-water: while the cement paste is plastic, it undergoes a volumetric contraction whose magnitude is of the order of one per cent of the absolute volume of dry cement.[9.13] However, the extent of hydration prior to setting is small and, once a certain rigidity of the system of hydrating cement paste has developed, the contraction induced by the loss of water by hydration is greatly restrained.

Water can also be lost by evaporation from the surface of the concrete while it is still in the

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plastic state. A similar loss can occur by suction by the underlying dry concrete or soil.[9.14] This contraction is known as *plastic shrinkage* because the concrete is still in the plastic state. The magnitude of plastic shrinkage is affected by the amount of water lost from the surface of the concrete, which is influenced by temperature, ambient relative humidity, and wind velocity (see Table 9.1). However, the rate of loss of water *per se* does not predict plastic shrinkage;[9.103] much depends on the rigidity of the mix. If the amount of water lost per unit area exceeds the amount of water brought to the surface by bleeding (see p. 207) and is large, surface cracking can occur. This is known as *plastic shrinkage cracking*, referred to on p. 400. Complete prevention of evaporation immediately after casting eliminates cracking.[9.47]

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Table 9.1. Plastic Shrinkage of Neat Cement Paste Stored in Air at a Relative Humidity of 50 per cent and Temperature of 20 °C (68 °F)[9.14]

Wind velocity		*Shrinkage 8 hours after placing* 10^{-6}
m/s	*mph*	
0	0	1 700
0.6	1.35	6 000
1.0	2.25	7 300
7 to 8	16 to 18	14 000

As stated in the section on Concreting in Hot Weather, the effective means of preventing plastic shrinkage cracking is by keeping down the rate of evaporation of water from the surface of concrete: it is recommended that the value of 1 kg/m^2 per hour (0.2 lb/ft^2 per hour) should not be exceeded.[9.97] It should be remembered that evaporation is increased when the temperature of the concrete is much higher than the ambient temperature; under such circumstances, plastic shrink-

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age can occur even if the relative humidity of the air is high. It is, therefore, best to protect the concrete from sun and wind, to place and finish fast, and to start curing very soon thereafter. Placing concrete on a dry subgrade should be avoided.

Cracking develops also over obstructions to uniform settlement, e.g. reinforcement or large aggregate particles; this is *plastic settlement cracking*, which is discussed in the section on Concreting in Hot Weather. Plastic cracking can develop also when a large horizontal area of concrete makes contraction in the horizontal direction more difficult than vertically: deep cracks of an irregular pattern are then formed.[9.15] Such cracks can properly be called *pre-setting cracks*. Typical plastic shrinkage cracks are usually parallel to one another, spaced 0.3 to 1 m (1 to 3 ft) apart, and are of considerable depth. They do not normally extend to free edges of the concrete because unrestrained contraction is possible there.

Plastic shrinkage is greater the greater the cement content of the mix[9.14] (Fig. 9.7) and the

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lower the water/cement ratio.[9.73] The relation between bleeding and plastic shrinkage is not straightforward;[9.15] for example, retardation of setting allows more bleeding and leads to increased plastic shrinkage.[9.73] On the other hand, greater bleeding capacity prevents too rapid a drying out of the surface of the concrete and this reduces plastic shrinkage *cracking*. In practice, it is the cracking that matters.

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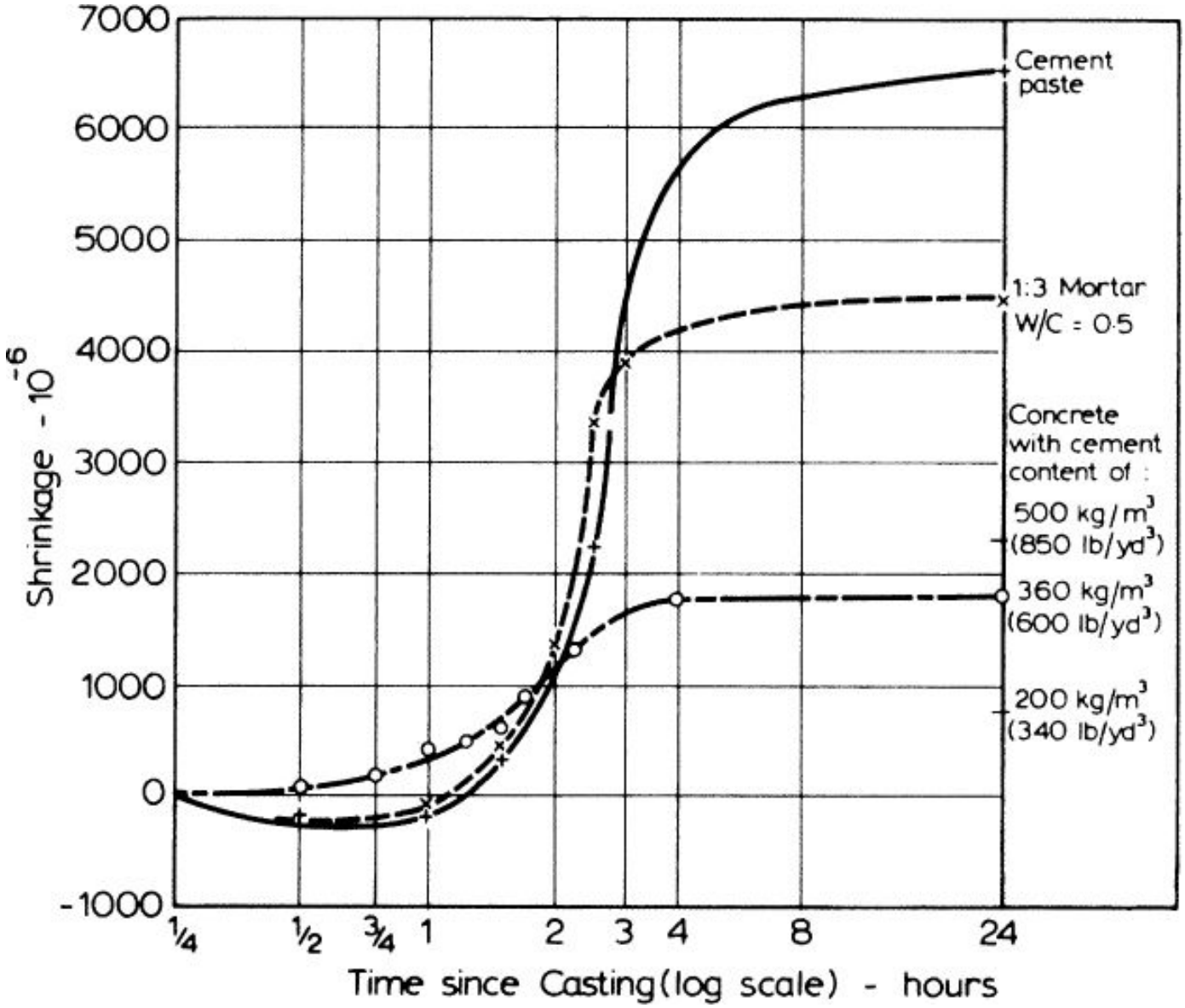


Fig. 9.7. Influence of cement content of the mix on early shrinkage in air at 20 °C (68 °F) and 50 per cent relative humidity with wind velocity of 1.0 m/s (2.25 mph)[9.14]

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Autogenous shrinkage

Volume changes occur also after setting has taken place, and may be in the form of shrinkage or swelling. Continued hydration, when a supply of water is present, leads to expansion (see next section) but, when no moisture movement to or from the cement paste is permitted, shrinkage occurs. This shrinkage is the consequence of withdrawal of water from the capillary pores by the hydration of the hitherto unhydrated cement, a process known as *self-desiccation.*

Shrinkage of such a conservative system is known as autogenous shrinkage or *autogenous volume change*, and it occurs in practice in the interior of a concrete mass. The contraction of the cement paste is restrained by the rigid skeleton of the already hydrated cement paste (mentioned in the preceding section) and also by the aggregate particles. In consequence, autogenous shrinkage of concrete is an order of magnitude smaller than in neat cement paste.[9.74]

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Although autogenous shrinkage is three-dimensional, it is usually expressed as a linear strain so that it can be considered alongside the drying shrinkage. Typical values of autogenous shrinkage are about 40×10^{-6} at the age of one month and 100×10^{-6} after five years.[9.17] Autogenous shrinkage tends to increase at higher temperatures, with a higher cement content, and possibly with finer cements,[9.46] and with cements which have a high C_3A and C_4AF content. At a constant content of blended cement, a higher content of fly ash leads to lower autogenous shrinkage.[9.46] As self-desiccation is greater at lower water/cement ratios, autogenous shrinkage could be expected to increase but this may not occur because of the more rigid structure of the hydrated cement paste at low water/cement ratios. Nevertheless, at very low water/cement ratios, autogenous shrinkage is very high: a value of 700×10^{-6} was reported[9.88] for concrete with a water/cement ratio of 0.17.

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As stated in the preceding paragraph, autogenous shrinkage is relatively small, except at extremely low water/cement ratios autogenous shrinkage. For practical purposes (other than in large mass concrete structures) such as used in high performance concrete need not be distinguished from shrinkage caused by drying out of concrete. The latter is known as drying shrinkage and, in practice, normally includes that contraction which is due to autogenous volume change.

An integrated view of the various types of shrinkage is given in ref. 9.159.

Swelling

Cement paste or concrete cured continuously in water from the time of placing exhibits a net increase in volume and an increase in mass. This swelling is due to the absorption of water by the cement gel: the water molecules act against the cohesive forces and tend to force the gel particles further apart, with a resultant swelling pressure. In addition, the ingress of water decreases the

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surface tension of the gel, and a further small expansion takes place.[9.18]

Linear expansion of neat cement paste (relative to the dimensions 24 hours after casting) has typical values of:[9.14]

1300×10^{-6} after 100 days
2000×10^{-6} after 1000 days, and
2200×10^{-6} after 2000 days.

These values of swelling, like those of shrinkage and creep, are expressed as linear strain in metres per metre or inches per inch.

The swelling of concrete is considerably smaller, approximately 100×10^{-6} to 150×10^{-6} for a mix with a cement content of 300 kg/m^3 (500 lb/yd^3).[9.14] This value is reached 6 to 12 months after casting, and only a very small further swelling takes place.

Swelling is accompanied by an increase in mass of the order of 1 per cent.[9.14] The increase in mass is thus considerably greater than the in-

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crease in volume because water enters to occupy the space created by the decrease in volume on hydration of the system cement-plus-water.

Swelling is larger in sea water and also under high pressure; such conditions exist in deep sea-water structures. At a pressure of 10 MPa (which corresponds to a depth of 100 m), the magnitude of swelling after 3 years can be about eight times higher than at atmospheric pressure.[9.10] Swelling which entails movement of sea water into concrete has implications for the ingress of chlorides into concrete (see p. 569).

Drying shrinkage

Withdrawal of water from concrete stored in unsaturated air causes drying shrinkage. A part of this movement is irreversible and should be distinguished from the reversible moisture movement caused by alternating storage under wet and dry conditions.

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Mechanism of shrinkage

The change in the volume of drying concrete is not equal to the volume of water removed. The loss of free water, which takes place first, causes little or no shrinkage. As drying continues, adsorbed water is removed and the change in the volume of unrestrained hydrated cement paste at that stage is equal approximately to the loss of a water layer one molecule thick from the surface of all gel particles. Since the 'thickness' of a water molecule is about 1 per cent of the gel particle size, a linear change in dimensions of cement paste on complete drying would be expected[9.18] to be of the order of $10\,000 \times 10^{-6}$; values up to 4000×10^{-6} have actually been observed.[9.19]

The influence of the gel particle size on drying is shown by the low shrinkage of the much more coarse-grained natural building stones (even when highly porous) and by the high shrinkage of fine grained shale.[9.18] Also, high-pressure steam-cured cement paste, which is microcrystalline and

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has a low specific surface, shrinks 5 to 10 times,[9.14] and sometimes even 17 times,[9.20] less than a similar paste cured normally.

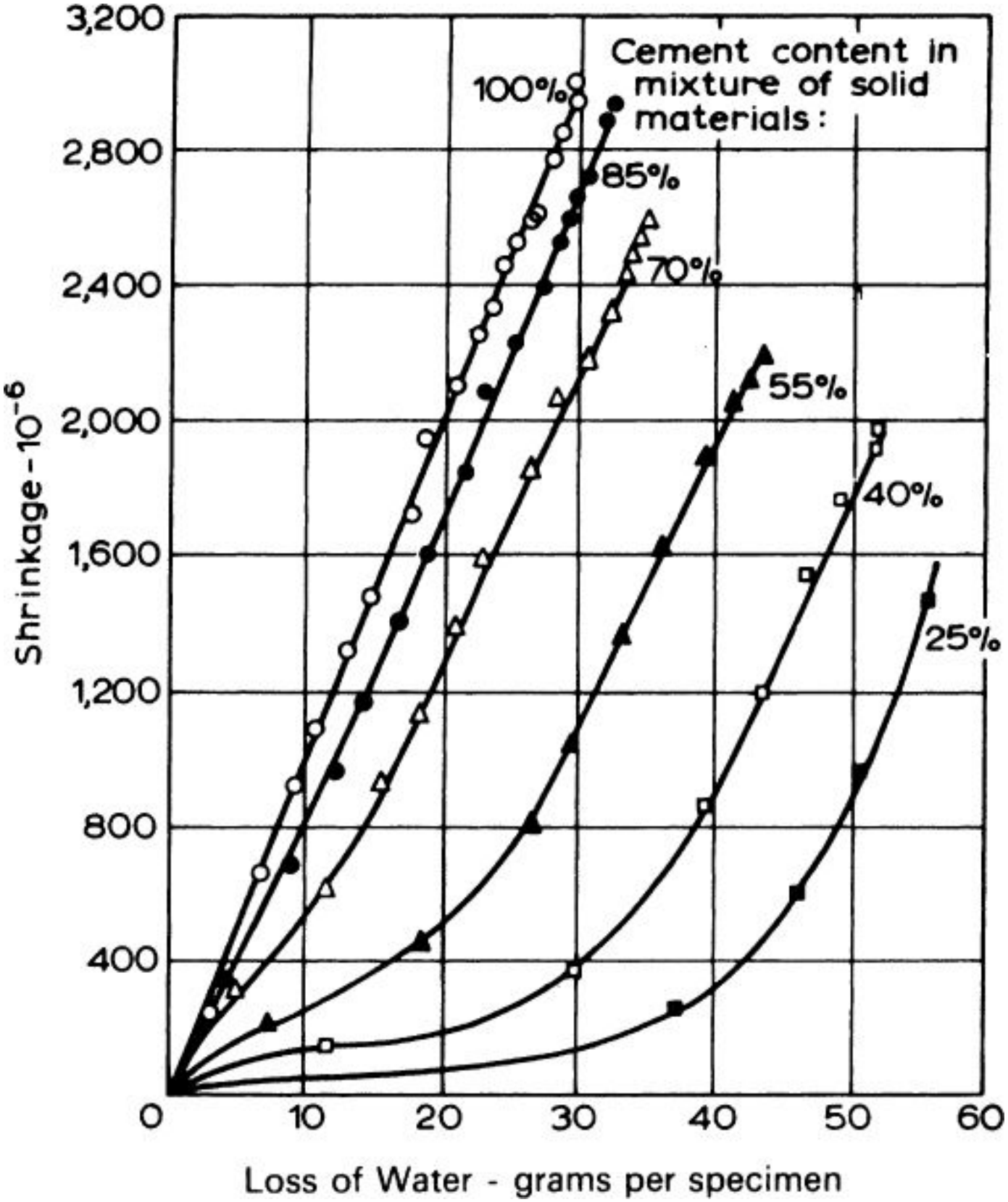
It is possible also that shrinkage, or a part of it, is related to the removal of intracrystalline water. Calcium silicate hydrate has been shown to undergo a change in lattice spacing from 1.4 to 0.9 nm on drying;[9.21] hydrated C_3A and calcium sulfoaluminate show similar behaviour.[9.22] It is thus not certain whether the moisture movement associated with shrinkage is inter- or intracrystalline. But, because pastes made with both Portland and high-alumina cements, and also with pure ground calcium monoaluminate, exhibit essentially similar shrinkage, the fundamental cause of shrinkage must be sought in the physical structure of the gel rather than in its chemical and mineralogical character.[9.22]

The relation between the mass of water lost and shrinkage is shown in Fig. 9.8. For neat cement pastes, the two quantities are proportional to

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one another as no capillary water is present and only adsorbed water is removed. However, mixes to which pulverized silica has been added and which, for workability reasons, require a higher water/cement ratio, contain capillary pores even when completely hydrated. Emptying of the capillaries causes a loss of water without shrinkage but, once the capillary water has been lost, the removal of adsorbed water takes place and causes shrinkage in the same manner as in a neat cement paste. Thus, the final slope of all the curves of Fig. 9.8 is the same. With concretes which contain some water in aggregate pores and in large (accidental) cavities, an even greater variation in the shape of the curves of water loss *vs*. shrinkage is found.

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Fig. 9.8. Relation between shrinkage and loss of water from specimens of cement–pulverized silica pastes cured for 7 days at 21 °C (70 °F) and then dried[9.18]

In concrete specimens, the loss of water with time depends on the size of specimen. A generalized pattern of loss of water with distance from the drying surfaces was developed by Mensi *et al.*[9.75] on the assumption that the rate of diffusion of vapour is proportional to the square root of time elapsed. They suggested that what occurs in a cylinder of diameter D_1 at time t_1 will occur in a geometrically similar cylinder of diameter kD_1 at time k^2t_1. In full-size concrete members, the situation is less simple because of the presence of edges[9.55] (see Fig. 9.9). Data on the time required for concrete, drying from one surface only, to lose 80 per cent of the evaporable water are given in Table 9.2.

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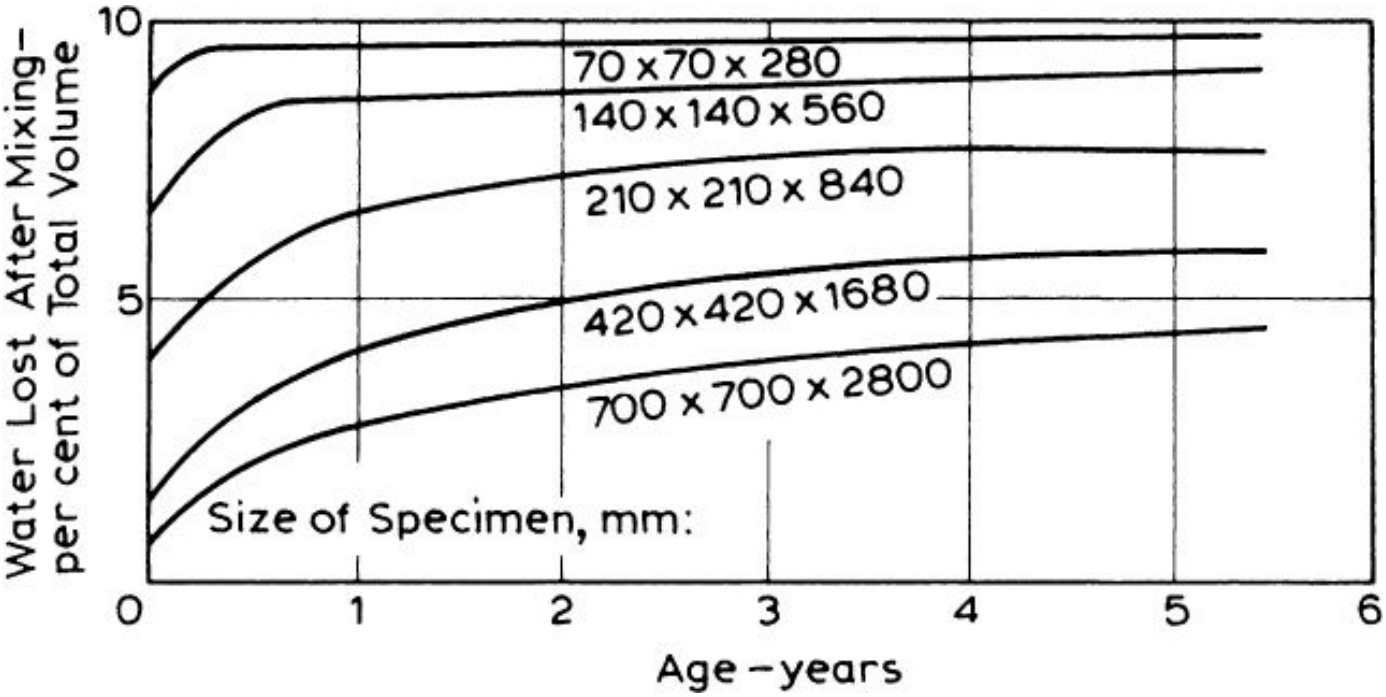


Fig. 9.9. Water loss in prisms of various sizes (relative humidity of air: 55 per cent)[9.55]

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Table 9.2. Indicative Periods of Drying of Concrete[*9.56]

Temperature °C (°F)	*Strength*	*Relative humidity*	*Moisture conductivity*	*Duration of drying for distances from the exposed surface of – mm (in.)*		
				50 (2)	*100 (4)*	*200 (8)*
5 (41)	Low	Low	High	3 m	1 y	4 y
	Medium	Medium	Medium	5 y	20 y	80 y
	High	High	Low	50 y	200 y	800 y
20 (68)	Low	Low	High	1 m	5 m	$1\frac{1}{2}$ y
	Medium	Medium	Medium	$2\frac{1}{2}$ y	10 y	40 y
	High	High	Low	25 y	100 y	400 y
50 (122)	Low	Low	High	10 d	1 m	5 m
	Medium	Medium	Medium	1 y	4 y	15 y
	High	High	Low	10 y	40 y	150 y
100 (212)	Low	Low	High	1 d	4 d	15 d
	Medium	Medium	Medium	1 m	5 m	$1\frac{1}{2}$ y
	High	High	Low	1 y	6 y	25 y

d = days, m = months, y = years.

[*]**For the purpose of this table, drying is defined as loss of 80 per cent of evaporable water.**

In translating the data on the loss of water into shrinkage, there is a further complication in that, whereas in small laboratory specimens surface cracking is minimal and potential shrinkage

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is achieved, in full-size structural members, surface cracking affects the *effective* shrinkage and causes a redistribution of internal stresses. Cracking possibly also increases the rate of loss of water. The topic of the influence of the size of a concrete member upon shrinkage is considered on p. 439.

Factors influencing shrinkage

As far as shrinkage of the hydrated cement paste itself is concerned, shrinkage is larger the higher the water/cement ratio because the latter determines the amount of evaporable water in the cement paste and the rate at which water can move towards the surface of the specimen. Brooks[9.77] demonstrated that shrinkage of hydrated cement paste is directly proportional to the water/cement ratio between the values of about 0.2 and 0.6. At higher water/cement ratios, the additional water is removed upon drying without resulting in shrinkage[9.77] (cf. Fig. 9.8).

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Let us now turn to mortar and concrete. Table 9.3 gives typical values of drying shrinkage of mortar and concrete specimens, 127 mm (5 in.) square in cross-section, stored at a temperature of 21 °C (70 °F) and a relative humidity of 50 per cent for six months. These values are no more than a guide because shrinkage is influenced by many factors.

Table 9.3. Typical Values of Shrinkage of Mortar and Concrete Specimens, 5 in. (127 mm) Square in Cross-section, Stored at a Relative Humidity of 50 per cent and 21 °C (70 °F)[9.19]

Aggregate/cement ratio	*Shrinkage after six months (10^{-6}) for water/cement ratio of:*			
	0.4	*0.5*	*0.6*	*0.7*
3	800	1200	—	—
4	550	850	1050	—
5	400	600	750	850
6	300	400	550	650
7	200	300	400	500

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The most important influence is exerted by aggregate, which restrains the amount of shrinkage that can actually be realized. The ratio of shrinkage of concrete, S_c, to shrinkage of neat cement paste, S_p, depends on the aggregate content in the concrete, a, and is[9.23]

$$S_c = S_p(1 - a)^n.$$

The experimental values of n vary between 1.2 and 1.7,[9.14] some variation arising from the relief of stress in the cement paste by creep.[9.35] Figure 9.10 shows typical results and yields a value of $n = 1.7$.

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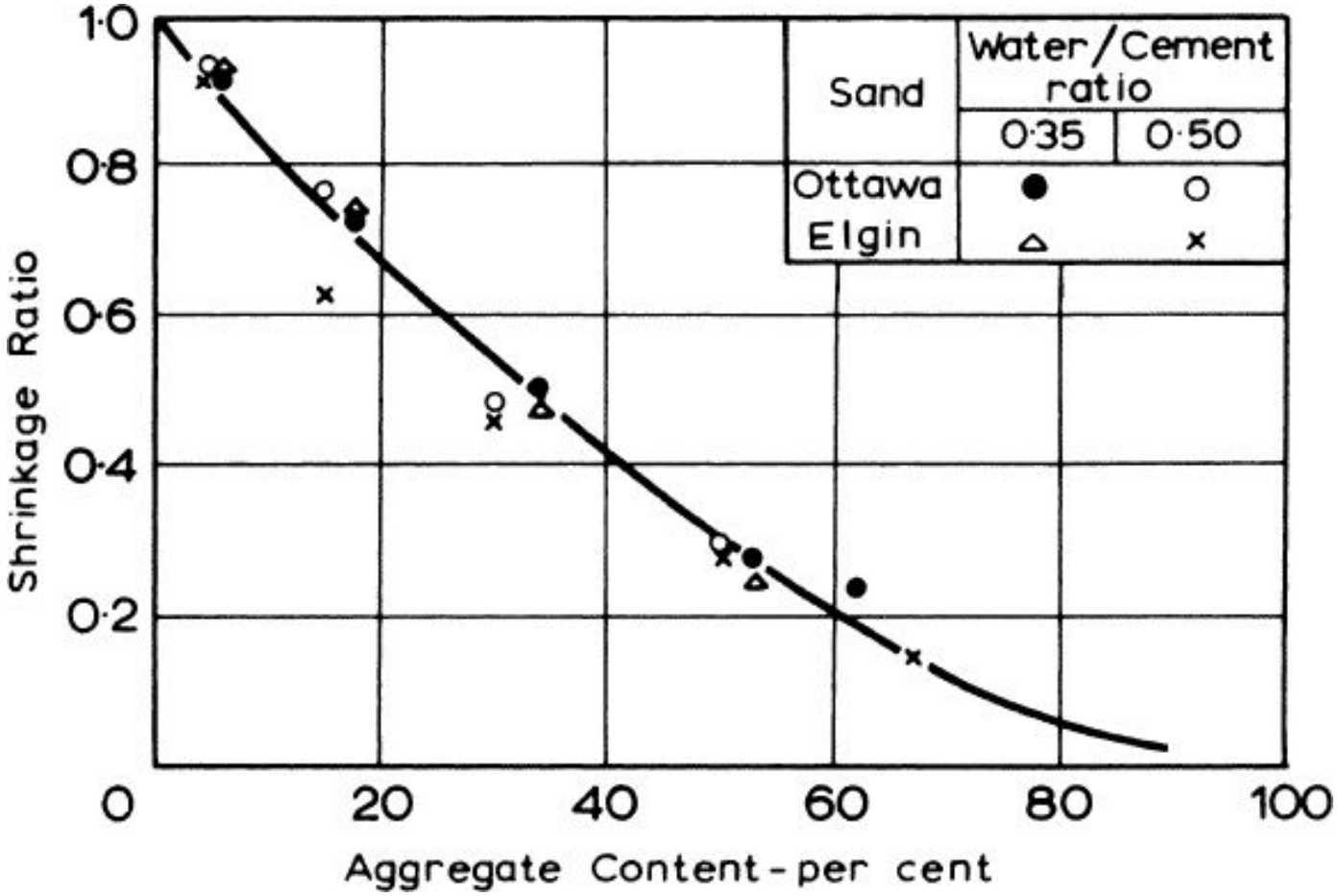


Fig. 9.10. Influence of the aggregate content in concrete (by volume) on the ratio of the shrinkage of concrete to the shrinkage of neat cement paste[9.23]

The validity of estimating the shrinkage of concrete from the shrinkage of neat cement paste, having the same water/cement ratio and the same degree of hydration, by taking into consideration the aggregate content and the modulus of elasti-

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city of the aggregate, was confirmed by Hansen and Almudaiheem.[9.72]

The size and grading of aggregate *per se* do not influence the magnitude of shrinkage, but a larger aggregate permits the use of a leaner mix and, hence, results in a lower shrinkage. If changing the maximum aggregate size from 6.3 to 152 mm ($\frac{1}{4}$ in. to 6 in.) means that the aggregate content can rise from 60 to 80 per cent of the total volume of concrete, then, as shown in Fig. 9.10, a threefold decrease in shrinkage will result.

Similarly, for a given strength, concrete of low workability contains more aggregate than a mix of high workability made with aggregate of the same size and, as a consequence, the former mix exhibits lower shrinkage.[9.18] For instance, increasing the aggregate content of concrete from 71 to 74 per cent (at the same water/cement ratio) will reduce shrinkage by about 20 per cent (Fig. 9.10).

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The twin influences of water/cement ratio and aggregate content (Table 9.3 and Fig. 9.10) can be combined in one graph; this is done in Fig. 9.11, but it must be remembered that the shrinkage values given are no more than typical for drying in a temperate climate. In practical terms, at a constant water/cement ratio, shrinkage increases with an increase in the cement content because this results in a larger volume of hydrated cement paste which is liable to shrinkage. However, at a given workability, which approximately means a constant water content, shrinkage is unaffected by an increase in the cement content, or may even decrease, because the water/cement ratio is reduced and the concrete is, therefore, better able to resist shrinkage. The overall pattern of these influences on shrinkage[9.76] is shown in Fig. 9.12.

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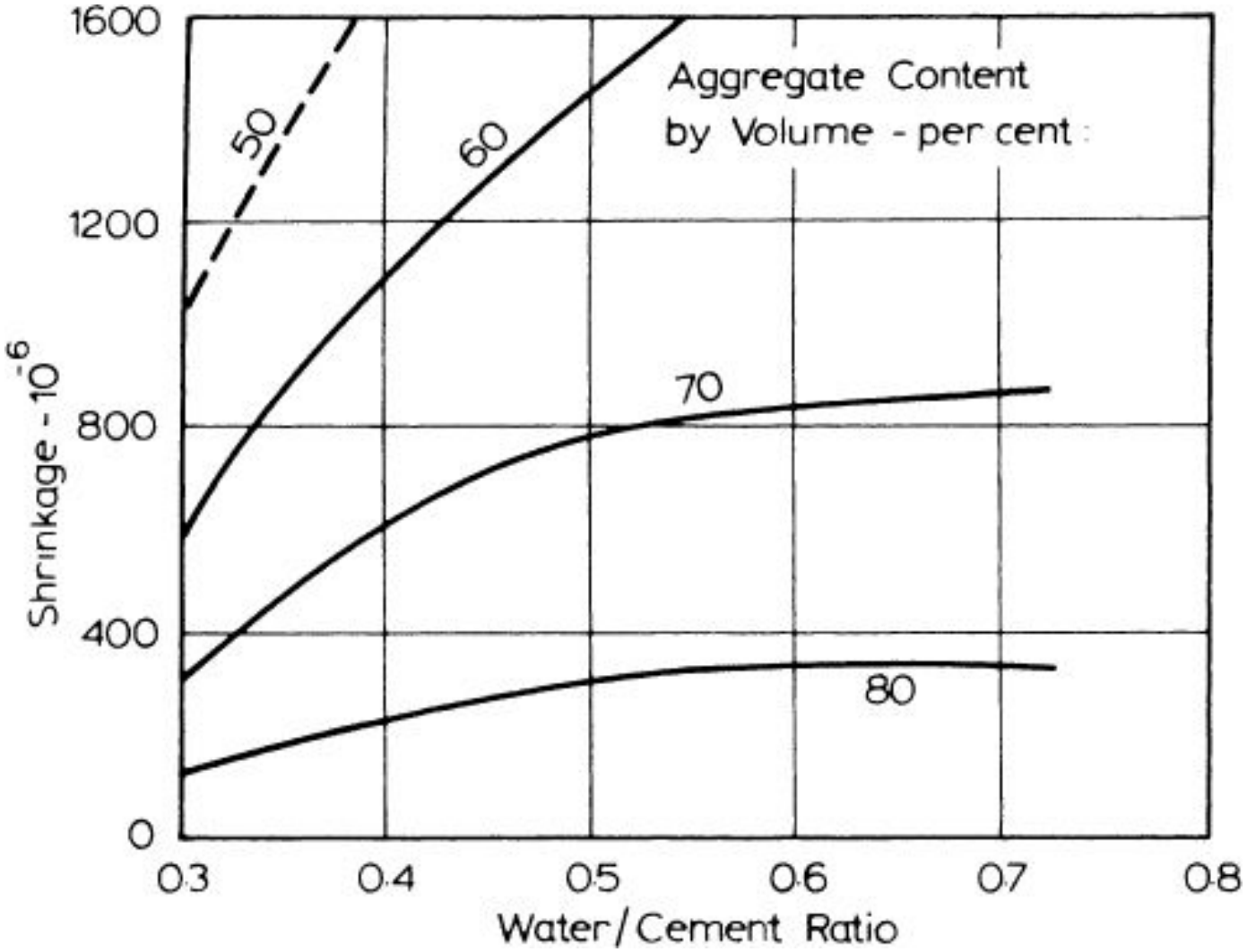


Fig. 9.11. Influence of water/cement ratio and aggregate content on shrinkage[9.48]

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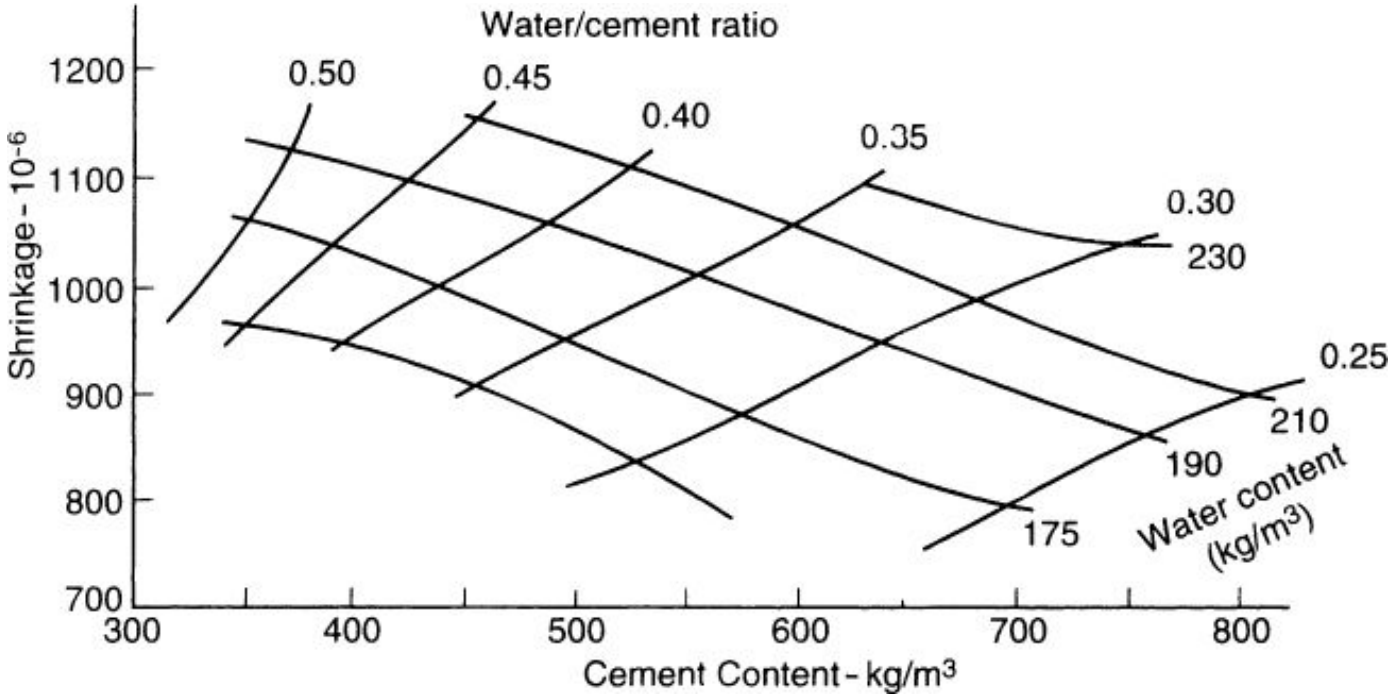


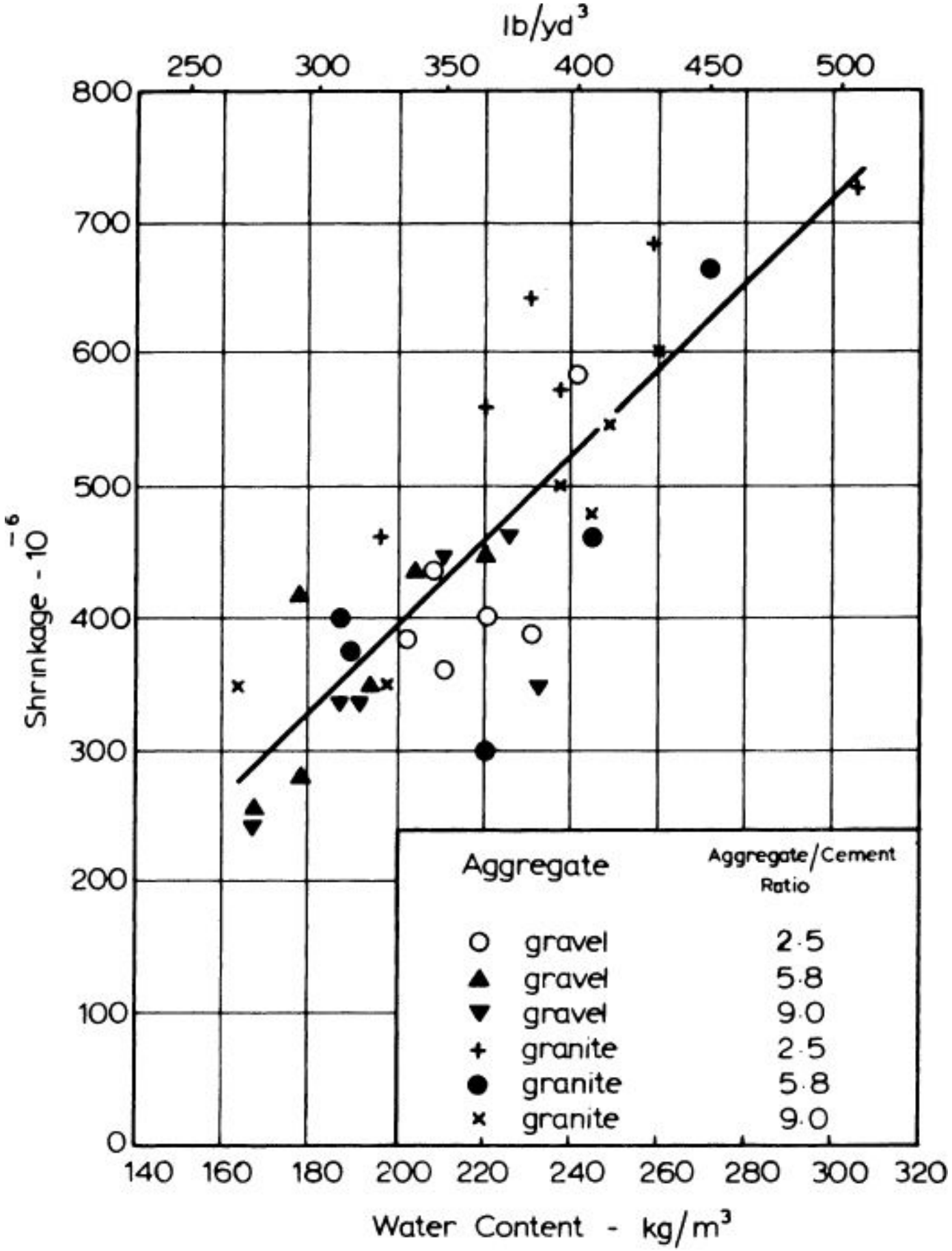
Fig. 9.12. The pattern of shrinkage as a function of cement content, water content, and water/cement ratio; concrete moist-cured for 28 days, thereafter dried for 450 days[9.76]

The water content of concrete affects shrinkage in so far as it reduces the volume of restraining aggregate. Thus, in general, the water content of a mix would indicate the order of shrinkage to be expected, following the general pattern of Fig. 9.13, but the water content *per se* is not a primary factor. In consequence, mixes having the same water content, but widely varying com-

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position, may exhibit different values of shrinkage.[9.82]

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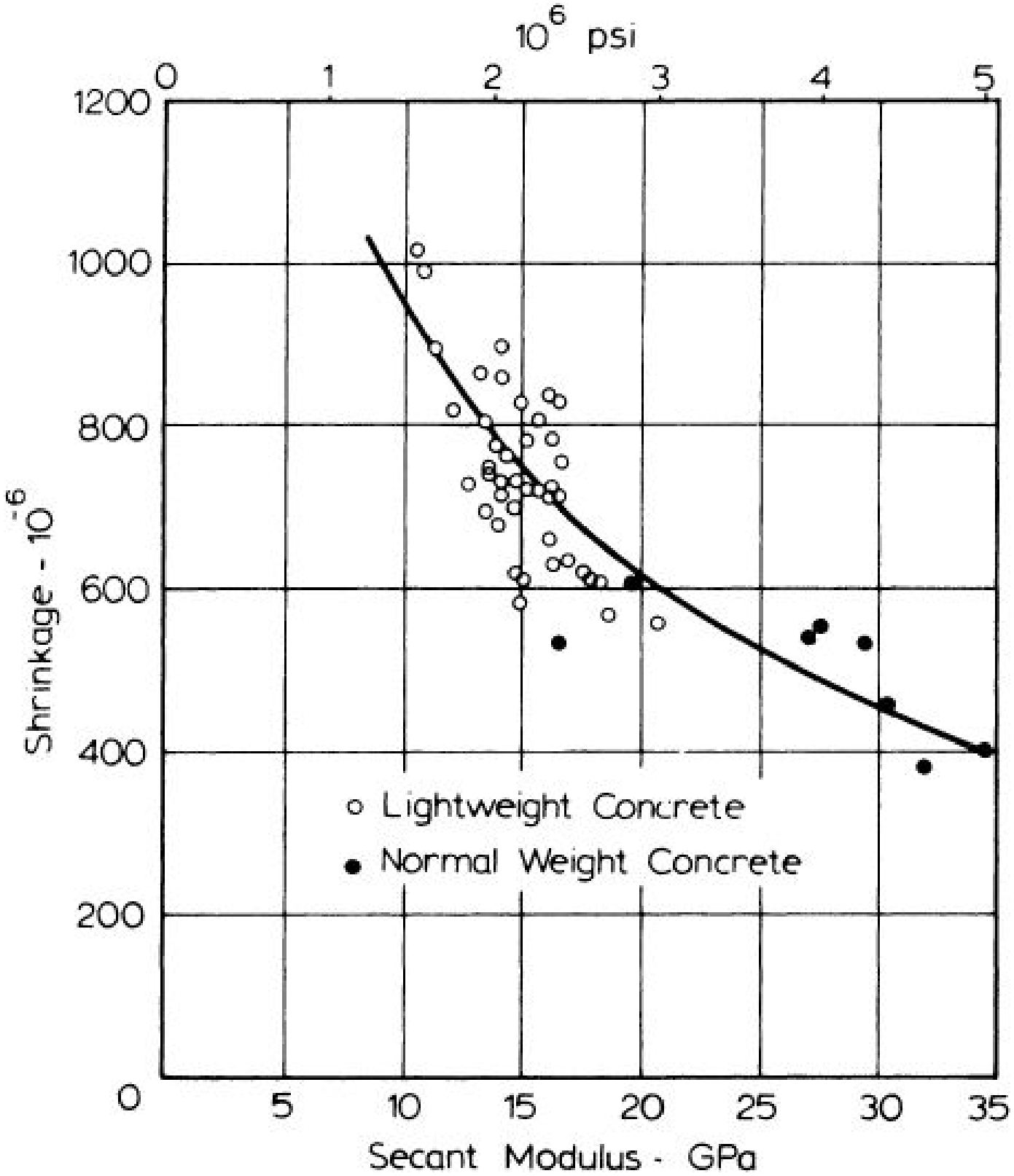


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Fig. 9.13. Relation between the water content of fresh concrete and drying shrinkage[9.25]

Let us now return to the restraining effect of the aggregate on shrinkage. The elastic properties of aggregate determine the degree of restraint offered; for example, steel aggregate leads to shrinkage one-third less, and expanded shale to one-third more, than ordinary aggregate.[9.6] This influence of aggregate was confirmed by Reichard[9.49] who found a correlation between shrinkage and the modulus of elasticity of concrete, which depends on the compressibility of the aggregate used (Fig. 9.14). The presence of clay in aggregate lowers its restraining effect on shrinkage and, because clay itself is subject to shrinkage, clay coatings on aggregate can increase shrinkage by up to 70 per cent.[9.18]

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Fig. 9.14. Relation between drying shrinkage after 2 years and secant modulus of elasticity of concrete (at a stress/strength ratio of 0.4) at 28 days[9.49]

Even within the range of ordinary aggregates, there is a considerable variation in shrinkage of the resulting concrete (Fig. 9.15). The usual natural aggregate itself is not normally subject to shrinkage, but there exist rocks which shrink on drying up to 900×10^{-6}; this is about the same magnitude as shrinkage of concrete made with non-shrinking aggregate. Shrinking aggregates are widespread in parts of Scotland but they exist also elsewhere. They are mainly *some* dolerites and basalts, and also some sedimentary rocks such as greywacke and mudstone. On the other hand, granite, limestone, and quartzite have been consistently found to be non-shrinking.

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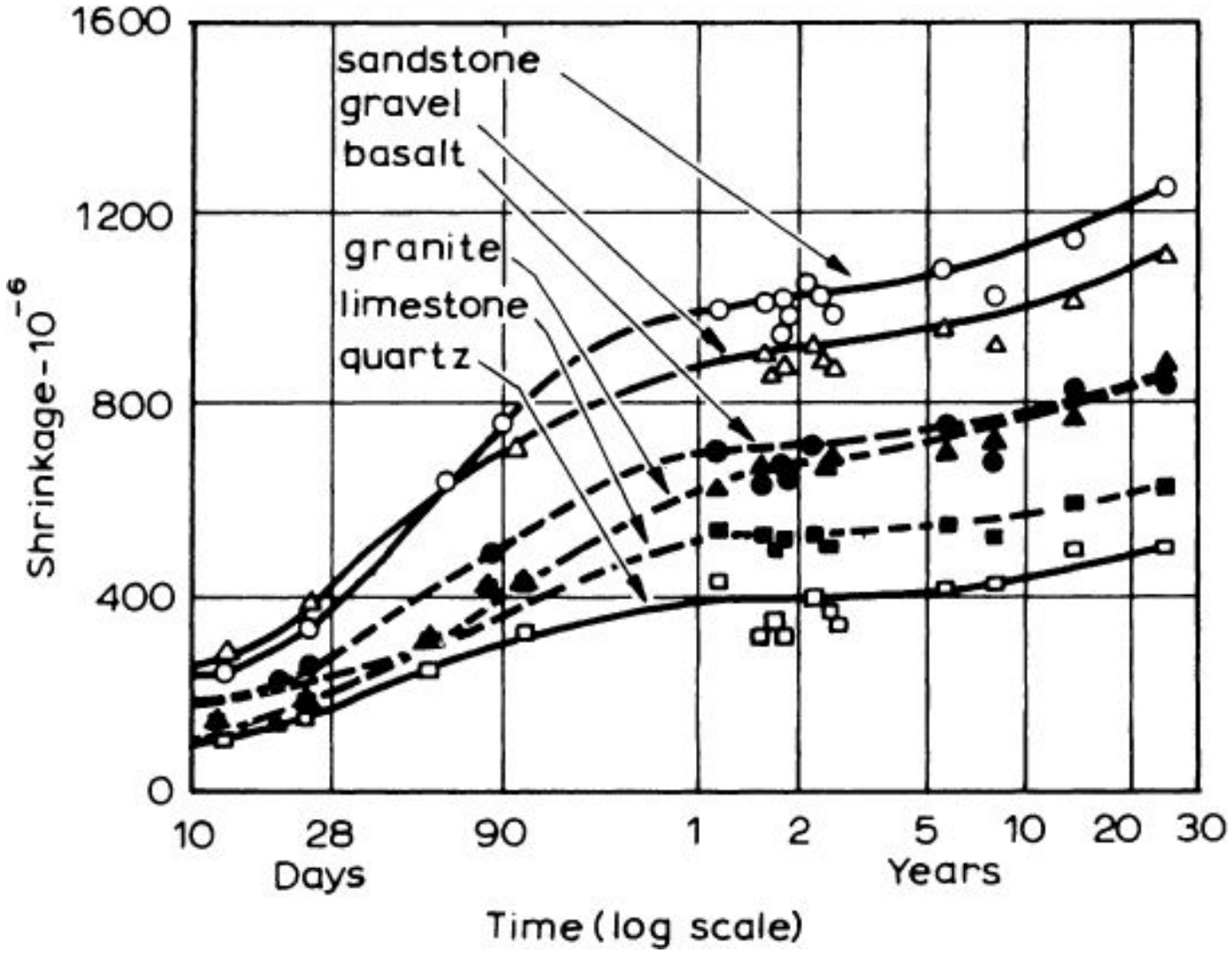


Fig. 9.15. Shrinkage of concretes of fixed mix proportions but made with different aggregates, and stored in air at 21 °C (70 °F) and a relative humidity of 50 per cent.[9.24] Time reckoned since end of wet curing at the age of 28 days

Concrete made with shrinking aggregate, and which therefore exhibits high shrinkage, may

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lead to serviceability problems in structures due to excessive deflection or warping (curling); if high shrinkage leads to cracking, durability of the structure may also be impaired. For these reasons, it is useful to determine the shrinkage of any suspect aggregate; a test method is prescribed by BS 812-120 : 1983 in which the shrinkage of concrete of fixed proportions and containing the given aggregate is determined by drying at 105 °C. The test is not intended to be used routinely. In this connection, it is useful to note that shrinking rocks usually have also high absorption, and this can be treated as a warning sign that the aggregate should be carefully investigated for its shrinkage properties. One possible way of dealing with such aggregates is to blend high- and low-shrinkage aggregates.

Lightweight aggregate usually leads to higher shrinkage, largely because the aggregate, having a lower modulus of elasticity, offers less restraint to the potential shrinkage of the cement paste. Those lightweight aggregates that have a large

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proportion of fine material smaller than 75 μm (No. 200) sieve have still higher shrinkage, as the fines lead to a larger void content.

The properties of cement have little influence on the shrinkage of concrete, and Swayze[9.26] has shown that a higher shrinkage of neat cement paste does not necessarily mean a higher shrinkage of concrete made with a given cement. Fineness of cement is a factor only in so far as particles coarser than, say, 75 μm (No. 200) sieve, which hydrate comparatively little, have a restraining effect similar to aggregate. Otherwise, contrary to some earlier suggestions, finer cement does not increase shrinkage of concrete made with normal[9.26,9.41] or lightweight[9.106] aggregate, although shrinkage of neat cement paste is increased.[9.40] The chemical composition of cement is now believed not to affect shrinkage except that cements deficient in gypsum exhibit a greatly increased shrinkage[9.27] because the initial framework established in setting determines the sub-

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sequent structure of the hydrated paste[9.22] and thus influences also the gel/space ratio, strength, and creep. An optimum gypsum content from the standpoint of retardation of cement is somewhat lower than that leading to least shrinkage.[9.28] For any given cement, the range of gypsum contents which is satisfactory for shrinkage is narrower than that for setting time.

Shrinkage of concrete made with high-alumina cement is of the same magnitude as when Portland cement is used, but it takes place much more rapidly.[9.19]

Including either fly ash or ground granulated blastfurnace slag in the mix increases shrinkage. Specifically, at a constant water/cement ratio, a higher proportion of fly ash or slag in the blended cement leads to higher shrinkage by some 20 per cent with the former material, and by up to 60 per cent at very high contents of slag.[9.71] Silica fume increases the long-term shrinkage.[9.81]

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Water-reducing admixtures *per se* probably cause a small increase in shrinkage. Their main effect is indirect in that the use of an admixture may result in a change in the water content or in the cement content of the mix, or in both, and it is the combined action of those changes that influences shrinkage.[9.71] Superplasticizers have been found[9.71] to increase shrinkage by some 10 to 20 per cent. However, the changes in the observed shrinkage are too small to be accepted as reliable and generally valid.

From the preceding statements, it can be expected that shrinkage of very-high-strength concrete, which contains a superplasticizer, is simply the outcome of the relevant and opposing factors: a very low water/cement ratio and concomitant high self-desiccation, which lead to low shrinkage, and a high cement content, which leads to high shrinkage. Thus, the usual approach to estimating shrinkage applies also to very-high-strength concrete. However, the more rigid struc-

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ture of such concrete restrains the magnitude of the effective shrinkage.

Entrainment of air has been found to have no effect on shrinkage.[9.29] Added calcium chloride increases shrinkage, generally between 10 and 50 per cent,[9.30] probably because a finer gel is produced and possibly because of greater carbonation of the more mature specimens with calcium chloride.[9.50]

Influence of curing and storage conditions

Shrinkage takes place over long periods: some movement has been observed even after 28 years[9.24] (Fig. 9.16), but a part of the long-term shrinkage is likely to be due to carbonation. Figure 9.16 (in which time is plotted on a logarithmic scale) shows that the rate of shrinkage decreases rapidly with time.

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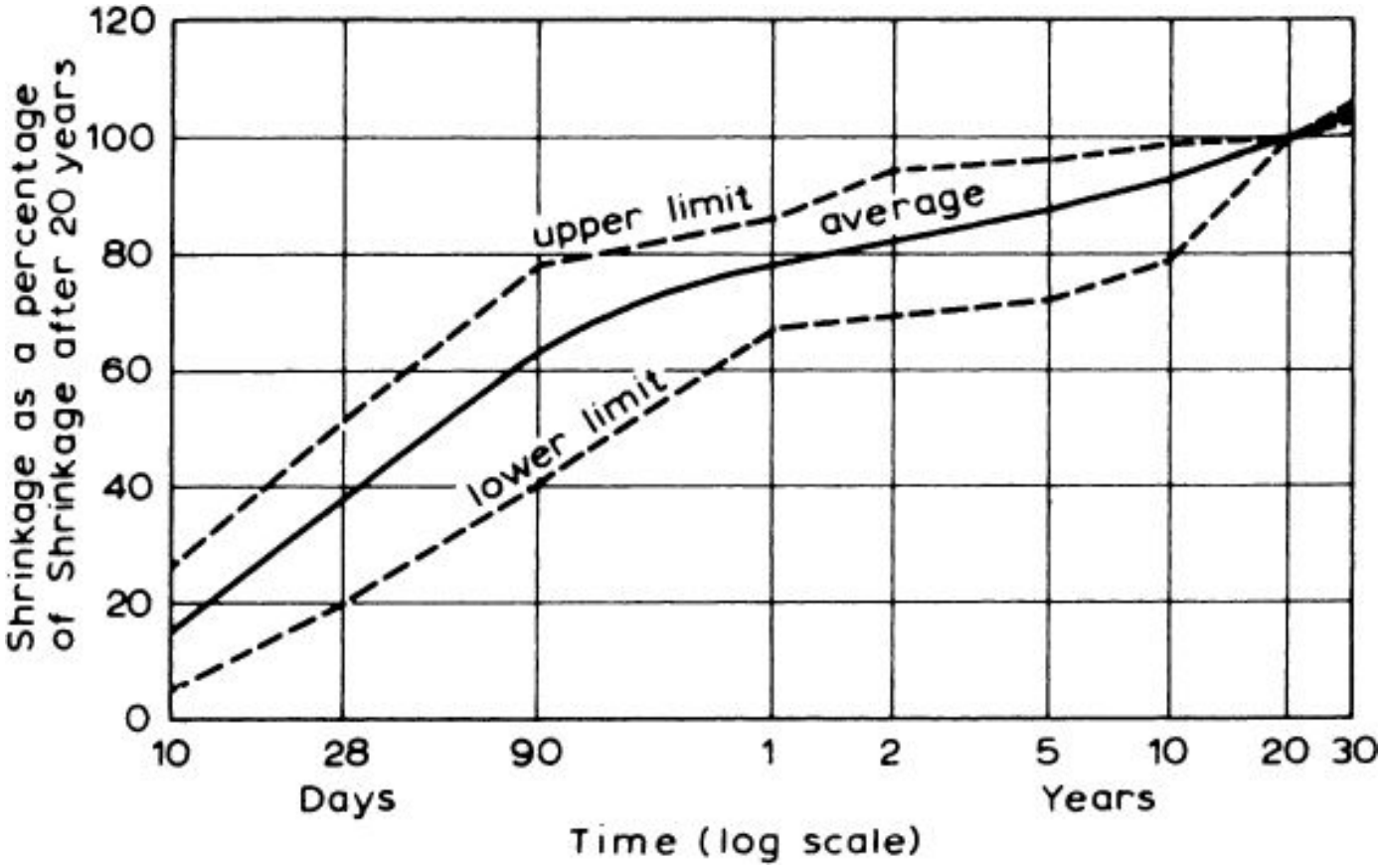


Fig. 9.16. Range of shrinkage–time curves for different concretes stored at relative humidities of 50 and 70 per cent[9.24]

Prolonged moist curing delays the advent of shrinkage, but the effect of curing on the magnitude of shrinkage is small, though rather complex. As far as neat cement paste is concerned, the greater the quantity of hydrated cement the smaller is the volume of unhydrated cement particles which restrain the shrinkage: thus prolonged curing could be expected to lead to greater

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shrinkage,[9.18] but the hydrated cement paste contains less water and becomes stronger with age and is able to attain a larger fraction of its shrinkage tendency without cracking. However, in concrete, if cracking takes place, e.g. around aggregate particles, the overall shrinkage, measured on a concrete specimen, apparently decreases. Well-cured concrete shrinks more rapidly[9.40] and, therefore, the relief of shrinkage stresses by creep is smaller; also, the concrete, being stronger, has an inherent low creep capacity. These factors may outweigh the higher tensile strength of well-cured concrete and may lead to cracking. In view of this, it is not surprising that contradictory results on the effects of curing on shrinkage have been reported, but in general the length of the curing period is not an important factor in shrinkage.

The magnitude of shrinkage is largely independent of the rate of drying except that transferring concrete directly from water to a very low humidity can lead to fracture. Rapid drying out does not allow a relief of stress by creep and may

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lead to more pronounced cracking. However, neither wind nor forced convection have any effect on the rate of drying of hardened concrete (except during very early stages) because the moisture conductivity of concrete is so low that only a very small rate of evaporation is possible: the rate cannot be increased by movement of air.[9.51] This has been confirmed experimentally.[9.52] (See p. 319 for evaporation from fresh concrete.)

The relative humidity of the medium surrounding the concrete greatly affects the magnitude of shrinkage, as shown for instance in Fig. 9.17. The same figure illustrates also the greater absolute magnitude of shrinkage compared with swelling in water: swelling is about six times smaller than shrinkage in air of relative humidity of 70 per cent or eight times smaller than shrinkage in air at 50 per cent.

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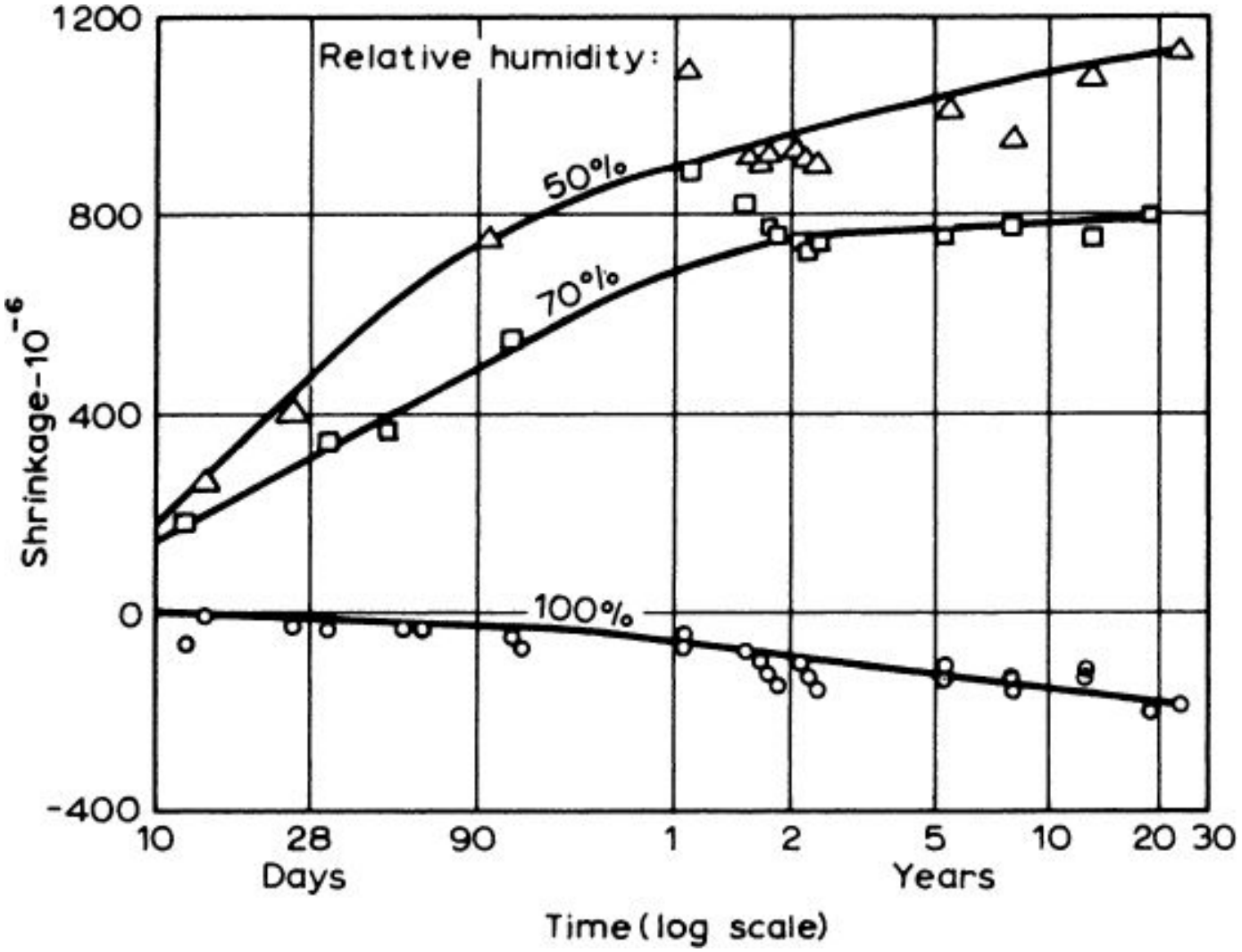


Fig. 9.17. Relation between shrinkage and time for concretes stored at different relative humidities.[9.24] Time reckoned since end of wet curing at the age of 28 days

We see thus that concrete placed in 'dry' (un-saturated) air shrinks, but it swells in water or air with a relative humidity of 100 per cent. This would indicate that the vapour pressure within

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the cement paste is always less than the saturated vapour pressure, and it is logical to expect that there is an intermediate humidity at which the paste would be in hygral equilibrium. In fact, Lorman[9.31] found this humidity to be 94 per cent, but in practice equilibrium is possible only in small and practically unrestrained specimens.

When it is desired to estimate shrinkage at a given relative humidity on the basis of a known value of shrinkage at some other relative humidity, the relation of ACI 209R-92[9.80] can be used. This is shown in Fig. 9.18, which includes also the relation proposed by Hansen and Almudaiheem.[9.72] The latter indicates a relative shrinkage lower than that given by ACI 209R-92 at relative humidities above 50 per cent. Hansen and Almudaiheem[9.72] also give values of relative shrinkage in the range of relative humidity of 11 to 40 per cent, for which ACI 209R-92 does not give any specific values.

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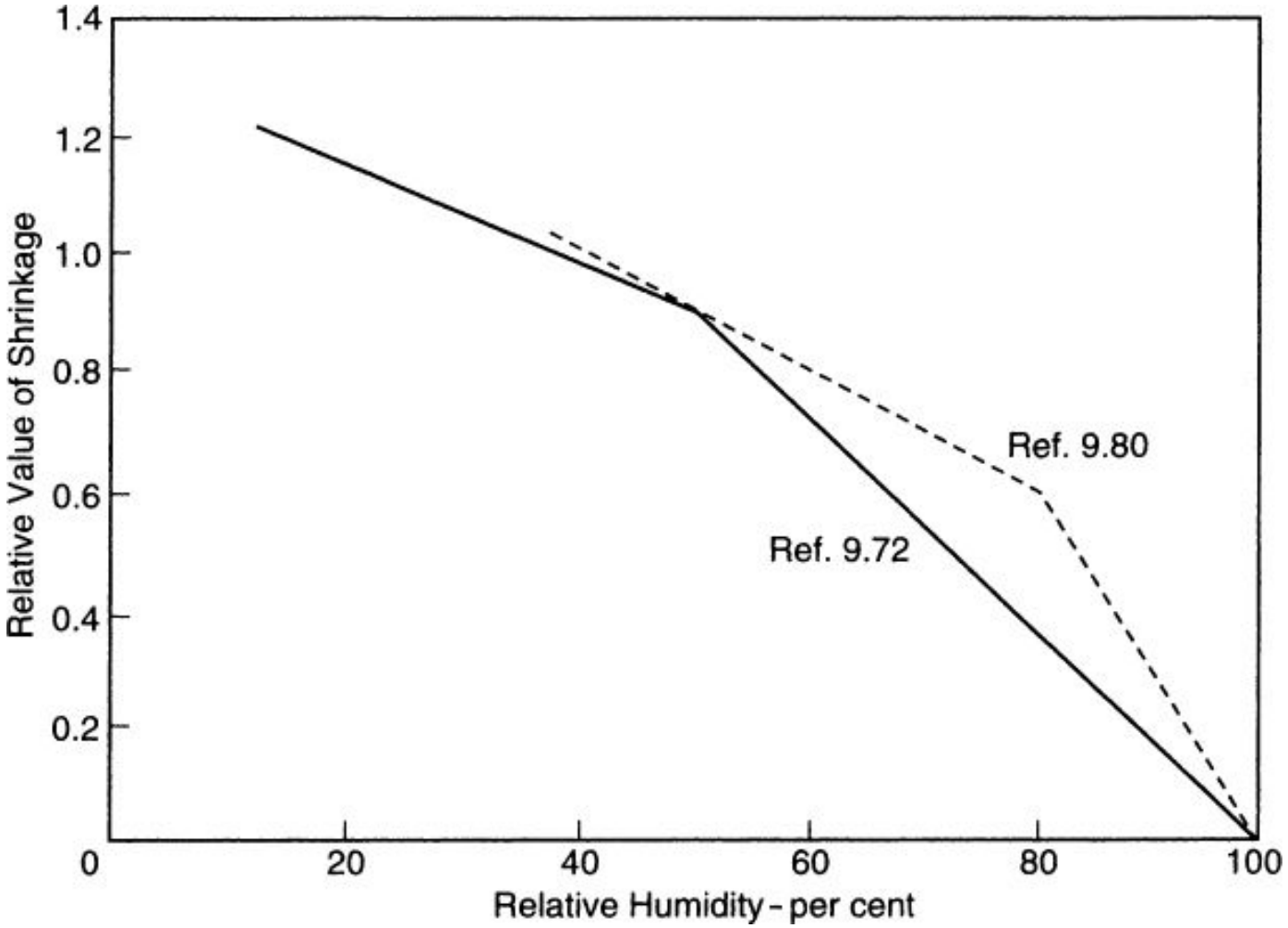


Fig. 9.18. Relative value of shrinkage as a function of ambient relative humidity according to ACI 209R-92[9.80] and Hansen and Almudaiheem[9.72]

Prediction of shrinkage

According to ACI 209R-92,[9.80] the development of shrinkage with time follows the equation

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$$s_t = \frac{t}{35 + t} s_{ult}$$

where s_t = shrinkage after t days since the end of 7-day moist curing

s_{ult} = ultimate shrinkage, and

t = time in days since the end of moist curing.

Prediction of the development of shrinkage by the above equation is subject to considerable variability, but the equation can be used to estimate ultimate shrinkage of a wide range of moist-cured concretes. It can be seen that one-half of the ultimate shrinkage is expected to occur after 35 days' drying. For steam-cured concrete, the value of 35 in the denominator is replaced by 55, and time t is reckoned from the end of steam curing at 1 to 3 days.

ACI 209R-92[9.80] gives a general expression for the prediction of shrinkage by modifying a standard value by a number of coefficients which

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allow for various factors. The error involved in such an approach must be expected to be large.

Various expressions for shrinkage are discussed by Neville *et al.*[9.84] These expressions can be used to estimate long-term shrinkage from short-term tests on the actual concrete. Such tests are necessary for a reasonably accurate prediction of shrinkage.

A method for determination of short-term shrinkage is prescribed in BS 1881-5 : 1984: the specimens are dried for a specified period under prescribed conditions of temperature and humidity. The shrinkage occurring under these conditions is about the same as that after a long exposure to air with a relative humidity of approximately 65 per cent,[9.19] and is therefore in excess of the shrinkage encountered outdoors in the British Isles. The magnitude of shrinkage can be determined using a measuring frame fitted with a micrometer gauge or a dial gauge reading to 10^{-5} strain, or by means of an extensometer or strain gauges. The American test method is prescribed

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by ASTM C 157-93; the air movement past the test specimens is carefully controlled and the relative humidity is maintained at 50 per cent. The ISO method is described in BS ISO 1920 : 2009.

Differential shrinkage

It was mentioned earlier that the potential shrinkage of neat cement paste is restrained by the aggregate. In addition, some restraint arises also from non-uniform shrinkage within the concrete member itself. Moisture loss takes place only at the surface so that a moisture gradient is established in the concrete specimen, which is thus subjected to differential shrinkage. The potential shrinkage is compensated by the strains due to internal stresses, tensile near the surface and compressive in the core. When drying takes place in an unsymmetrical manner, warping (curling) can result.

It may be useful to point out that the values of shrinkage generally quoted are those of *free shrinkage*, or *potential shrinkage*, that is, contrac-

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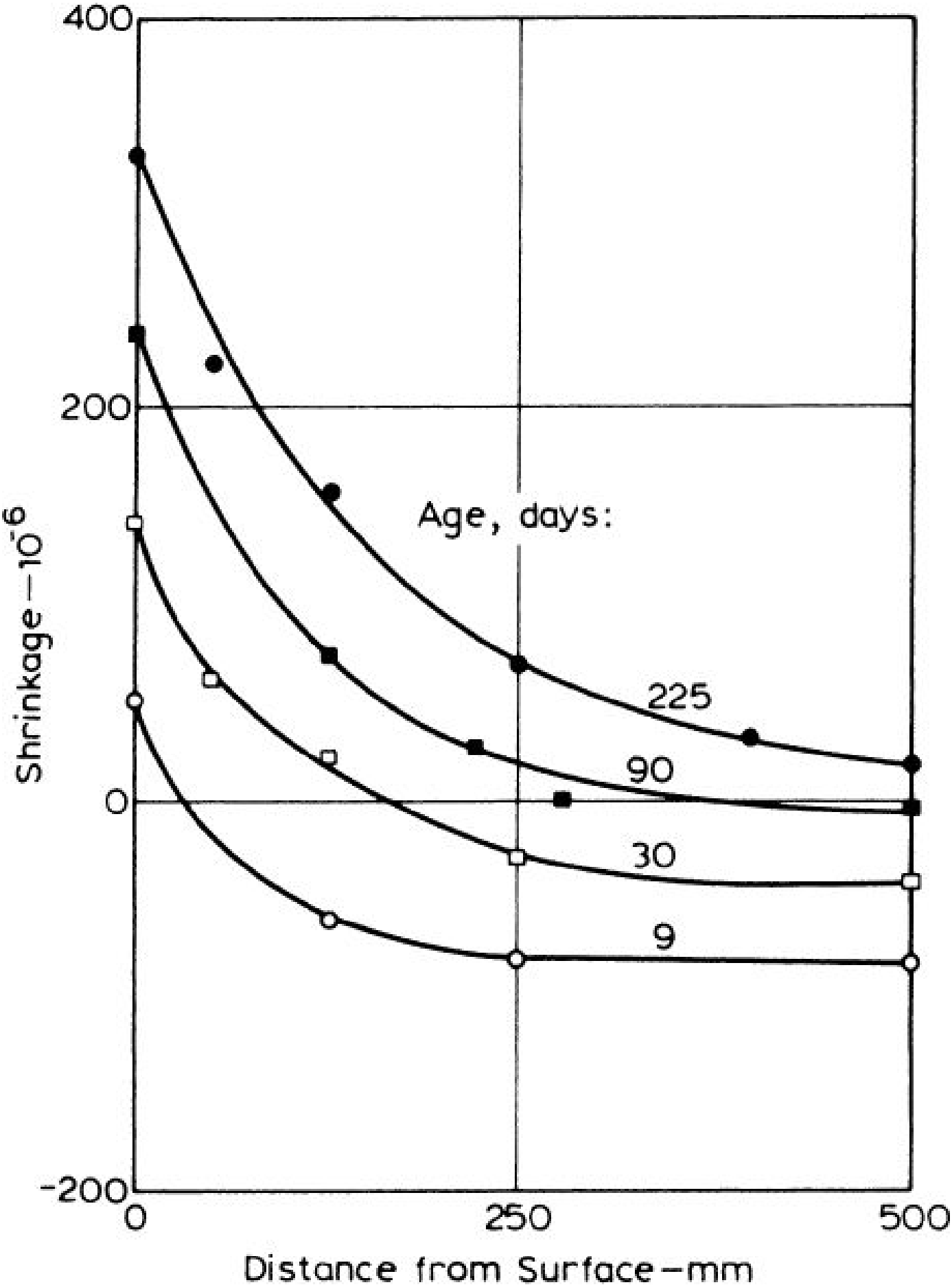
tion unrestrained either internally or by external constraints on a structural member. In considering the effect of the constraining forces on the actual shrinkage, it is important to realize that the induced stresses are modified by relaxation, which may prevent the development of cracking, as discussed on p. 442. Because relaxation occurs only slowly, it may prevent cracking when shrinkage develops slowly; however, the same magnitude of shrinkage occurring rapidly may well induce cracking. It is *shrinkage cracking* that is of paramount interest.

The progress of shrinkage extends gradually from the drying surface into the interior of the concrete but does so only extremely slowly. Desiccation was observed to reach the depth of 75 mm (3 in.) in one month but only 600 mm (2 ft) after 10 years.[9.14] Data[9.55] of L'Hermite are shown in Fig. 9.19; initial swelling in the interior can be seen. Ross[9.32] found the difference between shrinkage in a mortar slab at the surface and at a depth of 150 mm (6 in.) to be 470 ×

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10^{-6} after 200 days. If the modulus of elasticity of mortar is 21 GPa (3×10^6 psi) the differential shrinkage would induce a stress of 10 MPa (1400 psi); because the stress arises gradually it is relieved by creep but, even so, surface cracking may result.

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Fig. 9.19. Progress of shrinkage with time as a function of distance from drying surface (no drying possible in other directions). (Shrinkage values corrected for temperature differences)[9.55]

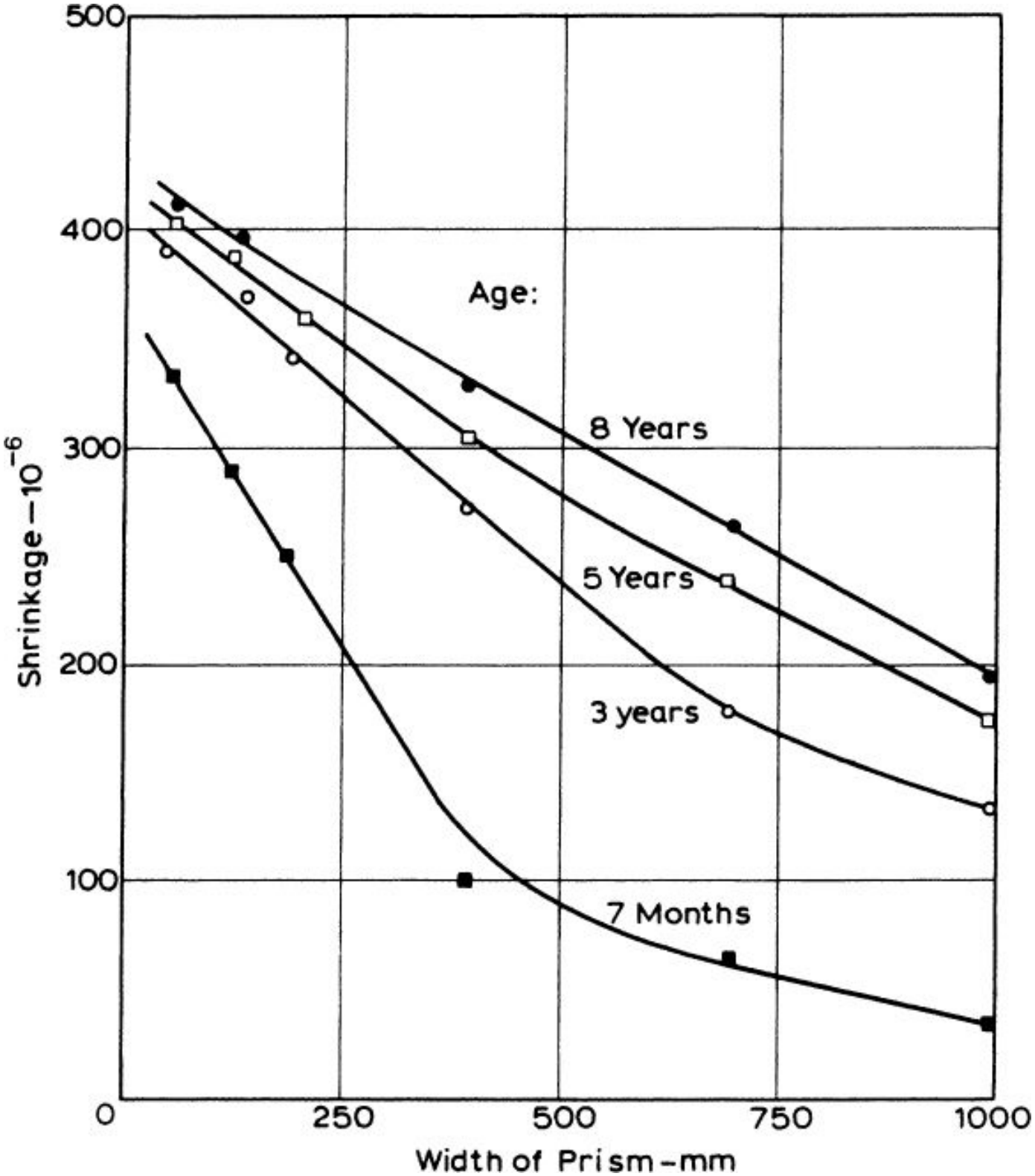
Because drying takes place at the surface of concrete, the magnitude of shrinkage varies considerably with the size and shape of the specimen, being a function of the surface/volume ratio.[9.32] A part of the size effect may also be due to the pronounced carbonation shrinkage of small specimens (see p. 444). Thus, for practical purposes, shrinkage cannot be considered as purely an inherent property of concrete without reference to the size of the concrete member.

Many investigations have, in fact, indicated an influence of the size of the specimen on shrinkage. The observed shrinkage decreases with an increase in the size of the specimen but, above some value, the size effect is small initially, although pronounced later (Fig. 9.20). The shape

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of the specimen also appears to enter the picture but, as a first approximation, shrinkage can be expressed as a function of the volume/surface ratio of the specimen. There appears to be a linear relation between this ratio and the logarithm of shrinkage[9.53] (Fig. 9.21). Furthermore, the ratio is linearly related to the logarithm of time required for half the shrinkage to be achieved. The latter relation applies to concretes made with different aggregates, so that, whereas the magnitude of shrinkage is affected by the type of aggregate used, the rate at which the final value of shrinkage is reached is not influenced.[9.53] It has been argued[9.16,9.83] that theoretically the ultimate shrinkage is independent of the size of the concrete element but, for realistic periods, it must be accepted that shrinkage is smaller in larger elements.

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Fig. 9.20. Relation between axial shrinkage and width of concrete prisms of square cross-section and length/width ratio of 4 (drying allowed at all surfaces)[9.55]

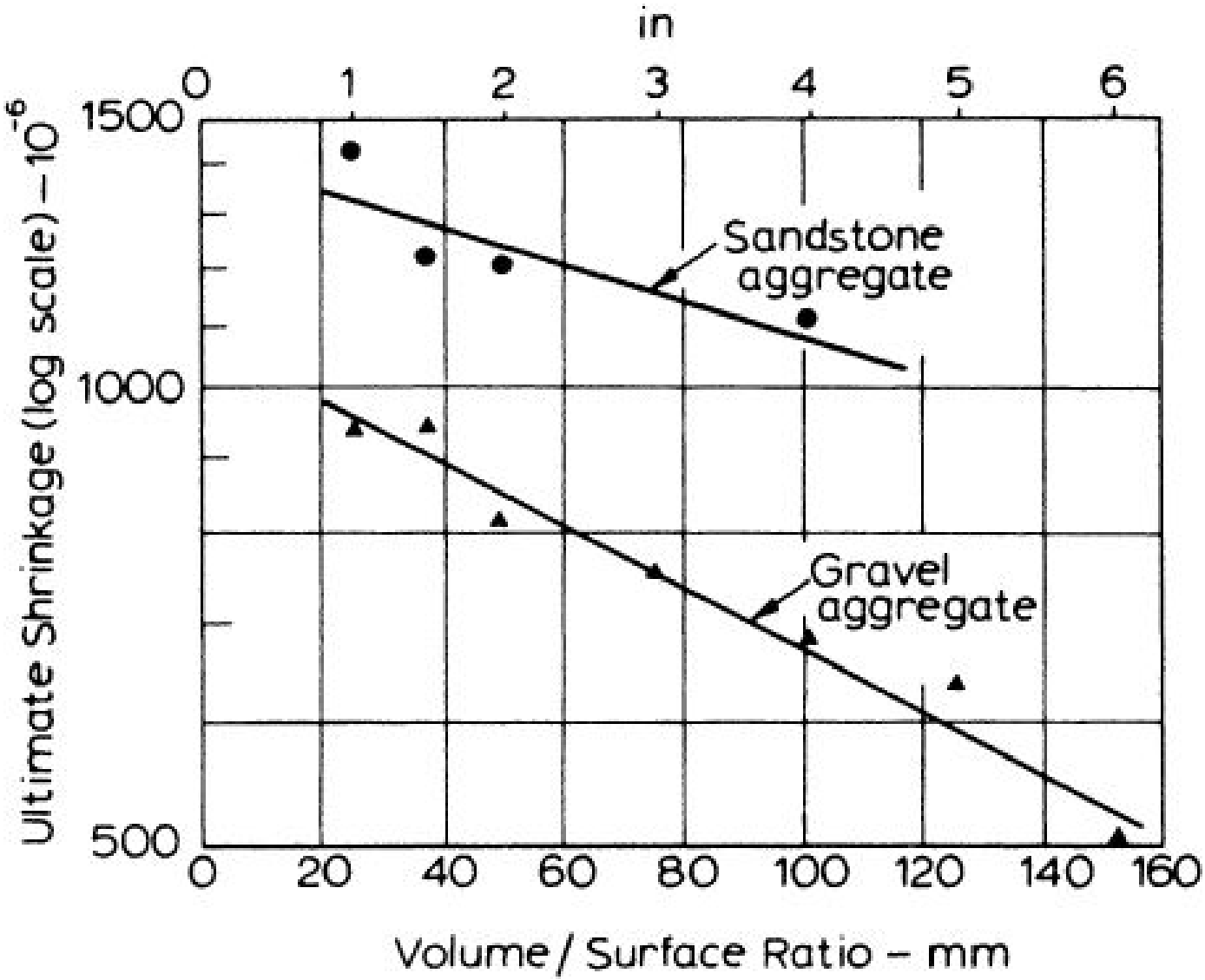


Fig. 9.21. Relation between ultimate shrinkage and volume/surface ratio[9.53]

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The effect of shape is secondary. I-shaped specimens exhibit less shrinkage than cylindrical ones of the same volume/surface ratio, the difference being 14 per cent on the average.[9.53] The difference, which can be explained in terms of variation in the mean distance that the water has to travel to the surface, is thus not significant for design purposes.

Shrinkage-induced cracking

As mentioned in connection with differential shrinkage, the importance of shrinkage in structures is largely related to cracking. Strictly speaking, we are concerned with the *cracking tendency* because the advent or absence of cracking depends not only on the potential contraction but also on the extensibility of concrete, its strength, and its degree of restraint to the deformation that may lead to cracking.[9.54] Restraint in the form of reinforcing bars or a gradient of stress increases extensibility of concrete in that it allows it to develop strain well beyond that corresponding to

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maximum stress. A high extensibility of concrete is generally desirable because it permits concrete to withstand greater volume changes.

The schematic pattern of crack development when stress is relieved by creep is shown in Fig. 9.22. Cracking can be avoided only if the stress induced by the free shrinkage strain, reduced by creep, is at all times smaller than the tensile strength of the concrete. Thus, time has a two-fold effect: the strength increases, thereby reducing the danger of cracking but, on the other hand, the modulus of elasticity also increases so that the stress induced by a given shrinkage becomes larger. Furthermore, the creep relief decreases with age so that the cracking tendency becomes greater. A minor practical point is that, if the cracks due to restrained shrinkage form at an early stage, and moisture subsequently has access to the crack, many of the cracks will become closed by autogenous healing.

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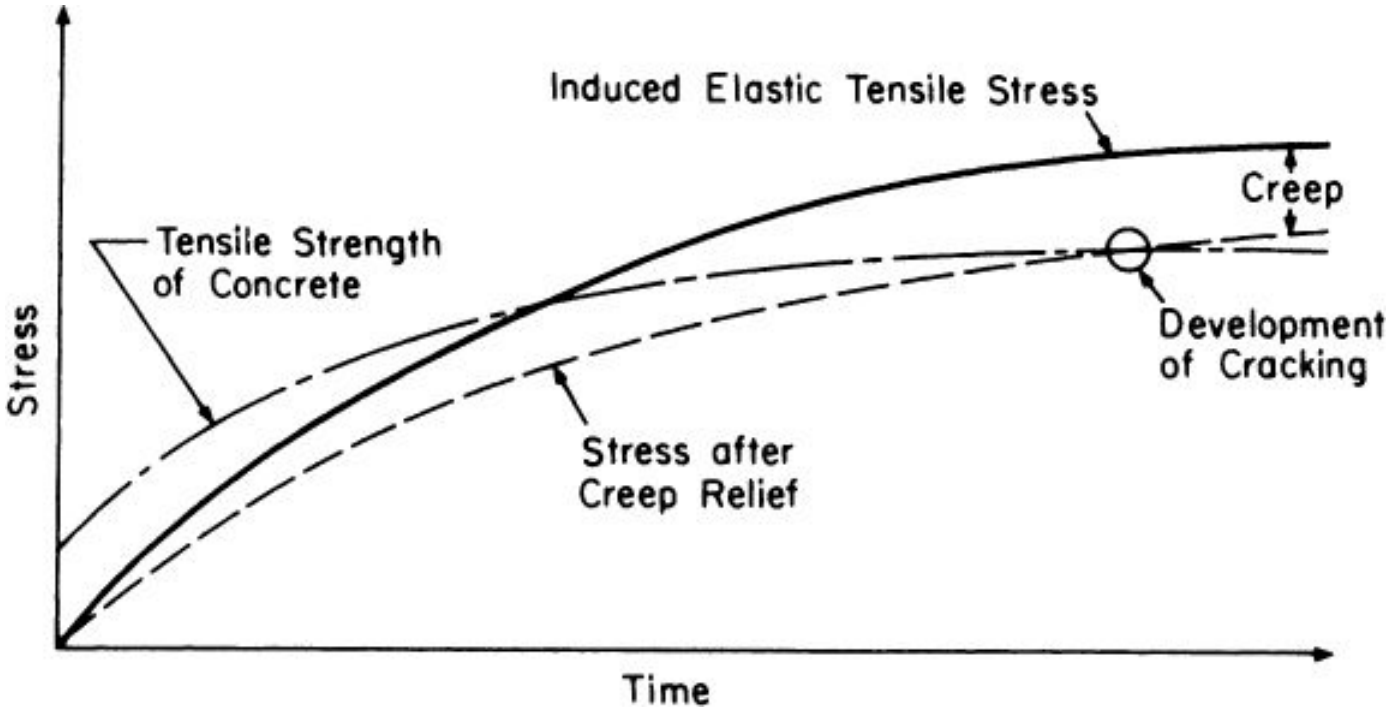


Fig. 9.22. Schematic pattern of crack development when tensile stress due to restrained shrinkage is relieved by creep

One of the most important factors in cracking is the water/cement ratio of the mix because its increase tends to increase shrinkage and, at the same time, to reduce the strength of the concrete. An increase in the cement content also increases shrinkage and, therefore, the cracking tendency, but the effect on strength is positive. This applies to drying shrinkage. Carbonation, although it produces shrinkage, reduces subsequent moisture movement, and therefore is advantageous from

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the standpoint of cracking tendency. On the other hand, the presence of clay in aggregate leads both to higher shrinkage and to greater cracking.

The use of admixtures may influence the cracking tendency through an interplay of effects on hardening, shrinkage, and creep. Specifically, retarders may allow more shrinkage to be accommodated in the form of plastic shrinkage (see p. 424) and also probably increase the extensibility of concrete, and therefore reduce cracking. On the other hand, if concrete has attained rigidity too rapidly, it cannot accommodate the would-be plastic shrinkage and, having a low strength, cracks.

The temperature at the time of placing determines the dimensions of concrete at the moment when it ceases to deform plastically (i.e. without loss of continuity). A subsequent drop in temperature will produce potential contraction. Thus, placing concrete in hot weather means a high cracking tendency. Steep temperature or moisture gradients produce severe internal restraints and

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thus represent a high cracking tendency. Likewise, restraint by the base of a member, or by other members, may lead to cracking.

These are some of the factors to be considered. Actual cracking and failure depend on the combination of factors, and indeed it is rarely that a single adverse factor is responsible for cracking of concrete. Early age thermal cracking is considered by CIRIA.[9.160]

There exists no standard test to assess cracking due to restrained shrinkage, but the use of a ring-shaped concrete specimen restrained by an internal steel ring can be informative with respect to the comparative resistance of different concretes to cracking.[9.78,9.79] Cracking of concrete due to various causes is considered in Chapter 10.

Moisture movement

If concrete which has been allowed to dry in air of a given relative humidity is subsequently placed in water (or at a higher humidity) it will

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swell. Not all initial drying shrinkage is, however, recovered, even after prolonged storage in water. For the usual range of concretes, the irreversible part of shrinkage represents between 0.3 and 0.6 of the drying shrinkage,[9.14] the lower value being more common.[9.25] The absence of fully reversible behaviour is probably due to the introduction of additional bonds within the gel during the period of drying, when closer contact between the gel particles is established. If the cement paste has hydrated to a considerable degree before drying, it will be less affected by the closer configuration of the gel when dry; in fact, neat cement paste, water-cured for six months and then dried, was found to have no residual shrinkage on rewetting.[9.33] On the other hand, if drying is accompanied by carbonation, the cement paste becomes insensitive to moisture movement so that the residual shrinkage is increased.[9.14]

The influence on moisture movement of curing before drying and of carbonation during dry-

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ing may explain why there is no simple relation between the magnitude of moisture movement and shrinkage.

Figure 9.23 shows the moisture movement, expressed as linear strain, of cement paste subjected to alternating storage in water and in air at a relative humidity of 50 per cent.[9.33] The magnitude of the moisture movement varies with the range of humidity and composition of concrete (Table 9.4). Lightweight concrete has a higher moisture movement than concrete made with ordinary aggregate.

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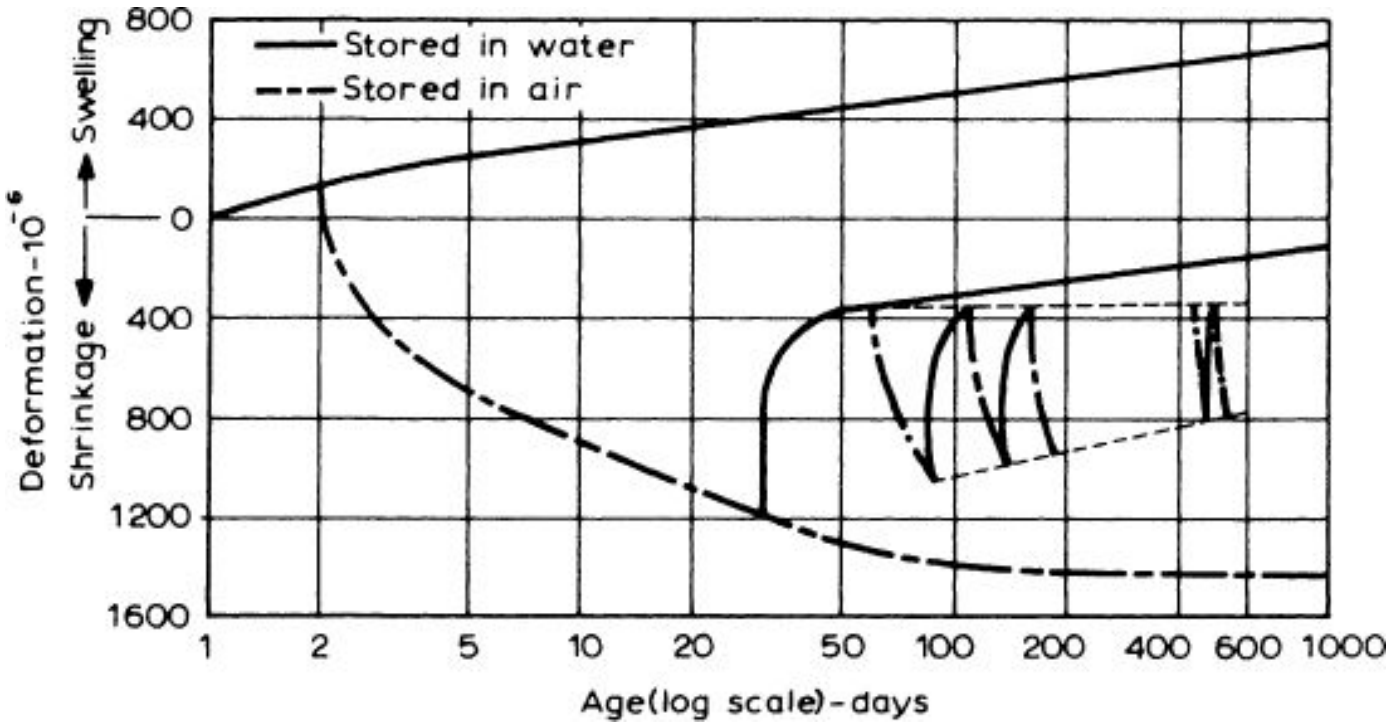


Fig. 9.23. Moisture movement of a 1:1 cement : pulverized basalt mix stored alternately in water and in air at 50 per cent relative humidity; cycle period 28 days[9.33]

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Table 9.4. Typical Values of Moisture Movement of Mortar and Concrete Dried at 50 °C (122 °F) and Immersed in Water[9.19]

Mix proportions by mass	*Moisture movement (linear strain) 10^{-6}*
Neat cement	1000
1:1 mortar	400
1:2 mortar	300
1:3 mortar	200
1:2:4 concrete	300

For a given concrete, there is a gradual reduction in the moisture movement during succeeding cycles, probably due to the creation of additional bonds within the gel.[9.22] If the water storage periods are of sufficient duration, the continued hydration of cement results in some additional swelling so that there is a net increase in dimensions superimposed on the reversible movement due to drying and wetting.[9.19] (In Fig. 9.23 this would be shown by a slight rise in the upper dotted line.)

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Carbonation shrinkage

In addition to shrinkage upon drying, the surface zone of concrete undergoes shrinkage due to carbonation, and some of the experimental data on drying shrinkage include the effects of carbonation. Drying shrinkage and carbonation shrinkage are, however, quite distinct in nature.

The process of carbonation is discussed in Chapter 10 and, at this stage, our concern is limited to carbonation shrinkage. However, it should be noted that, because carbon dioxide is fixed by the hydrated cement paste, the mass of the latter increases. Consequently, the mass of concrete also increases. When concrete dries and carbonates simultaneously, the increase in mass on carbonation may at some stage give the misleading impression that the drying process has reached the stage of constant mass, i.e. equilibrium (see Fig. 9.24). Such an interpretation of test data must clearly be guarded against.[9.58]

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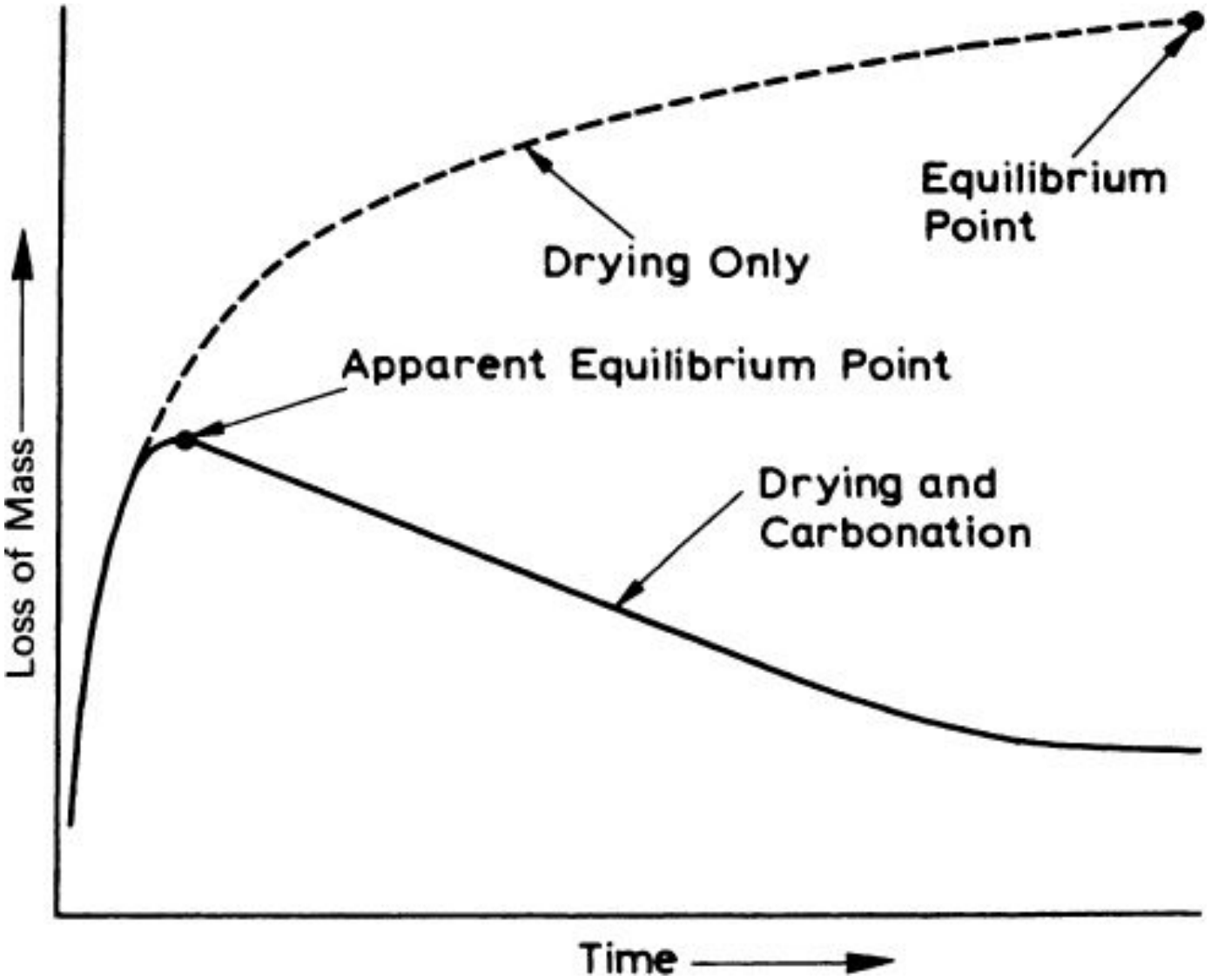


Fig. 9.24. Loss of mass of concrete due to drying and carbonation[9.58]

Carbonation shrinkage is probably caused by the dissolving of crystals of $Ca(OH)_2$ while under a compressive stress (imposed by the drying shrinkage) and depositing of $CaCO_3$ in spaces free from stress; the compressibility of the hydrated cement paste is thus temporarily increased.

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If carbonation proceeds to the stage of dehydration of C-S-H, this also produces carbonation shrinkage.[9.104]

Figure 9.25 shows the drying shrinkage of mortar specimens dried in CO_2-free air at different relative humidities, and also the shrinkage after subsequent carbonation. Carbonation increases the shrinkage at intermediate humidities, but not at 100 per cent or 25 per cent. In the latter case, there is insufficient water in the pores within the cement paste for CO_2 to form carbonic acid. On the other hand, when the pores are full of water, the diffusion of CO_2 into the paste is very slow; it is also possible that the diffusion of calcium ions from the paste leads to precipitation of $CaCO_3$ with a consequent clogging of surface pores.[9.37]

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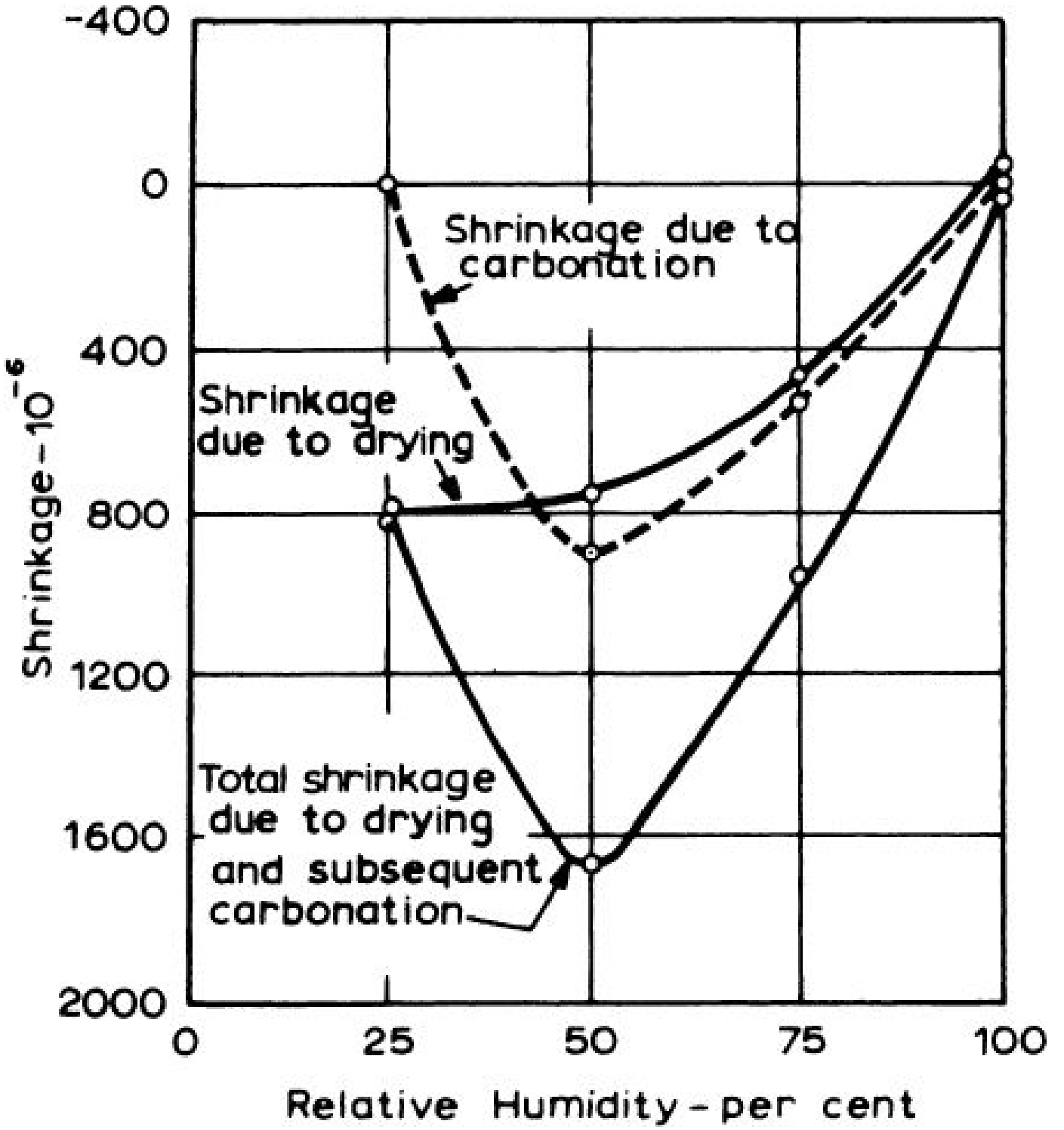


Fig. 9.25. Drying shrinkage and carbonation shrinkage of mortar at different relative humidities[9.37]

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The sequence of drying and carbonation greatly affects the total magnitude of shrinkage. Simultaneous drying and carbonation produces lower total shrinkage than when drying is followed by carbonation (Fig. 9.26) because, in the former case, a large part of the carbonation occurs at relative humidities above 50 per cent: under such conditions carbonation shrinkage is reduced (Fig. 9.25). Carbonation shrinkage of high-pressure steam-cured concrete is very small.

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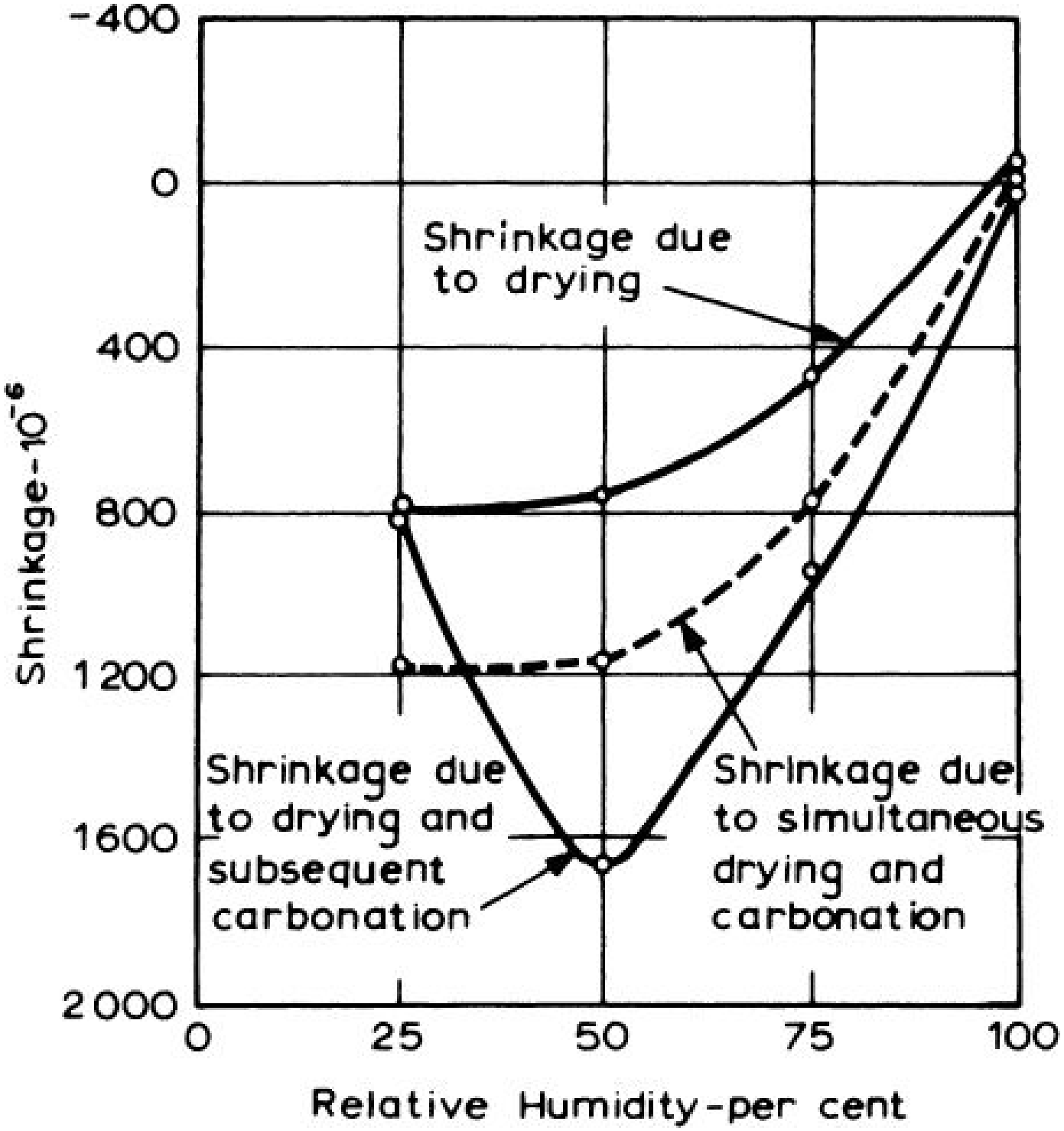


Fig. 9.26. Influence of the sequence of drying and carbonation of mortar on shrinkage[9.37]

When concrete is subjected to alternating wetting and drying in air containing CO_2, shrinkage

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due to carbonation (during the drying cycle) becomes progressively more apparent. The total shrinkage at any stage is greater than if drying took place in CO_2-free air,[9.37] so that carbonation increases the magnitude of irreversible shrinkage and may contribute to crazing of exposed concrete. Crazing is a form of shallow cracking induced by the restrained shrinkage of the surface zone against the non-shrinking interior of the concrete.

However, carbonation of concrete prior to exposure to alternating wetting and drying reduces the moisture movement, sometimes by nearly a half.[9.38] A practical application of this is to precarbonate precast products immediately after demoulding by exposing them to flue gases. Concrete with a small moisture movement is then obtained, but the humidity conditions during carbonation have to be carefully controlled. Various techniques of carbonation of concrete products are described in ACI 517.2R-87.[9.96]

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Shrinkage compensation by the use of expansive cements[*]

[*] This section was substantially published in ref. 9.105.

The discussion of drying shrinkage earlier in this chapter should have made it clear that shrinkage is probably one of the least desirable properties of concrete. When shrinkage is restrained, it may lead to shrinkage cracking, which mars the appearance of concrete and makes it more vulnerable to attack by external agents, thus adversely affecting durability. But even unrestrained shrinkage is harmful: adjacent concrete elements shrink away from one another, thus opening 'external cracks'. Shrinkage is also responsible for a part of the loss of the initial stress in the tendons in prestressed concrete.

It is not surprising, therefore, that many attempts have been made to develop a cement which, on hydration, would counteract the deformation induced by shrinkage. In special cases,

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even a net expansion of concrete on hardening may be advantageous. Concrete containing such an expansive cement expands in the first few days of its life, and a form of prestress is obtained by restraining this expansion with steel reinforcement: steel is put in tension and concrete in compression. Restraint by external means is also possible. Such concrete is known as *shrinkage-compensating concrete.*

It is also possible to use expansive cement in order to produce *self-stressing concrete*, in which the restrained expansion, remaining after most of the shrinkage has occurred, is high enough to induce a significant compressive stress in concrete[9.3] (up to about 7 MPa (1000 psi)).

Expansive cement, although considerably more expensive than Portland cement, is valuable in concrete structures in which a reduction in cracking is of importance, for instance, bridge decks, pavement slabs, and liquid storage tanks.

It is worth making it clear that the use of expansive cement does not prevent the development

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of shrinkage. What happens is that the restrained early expansion balances approximately the subsequent normal shrinkage; this is shown in Fig. 9.27. Usually, a small residual expansion is aimed at because, as long as some compressive stress in concrete is retained, shrinkage cracking will not develop.

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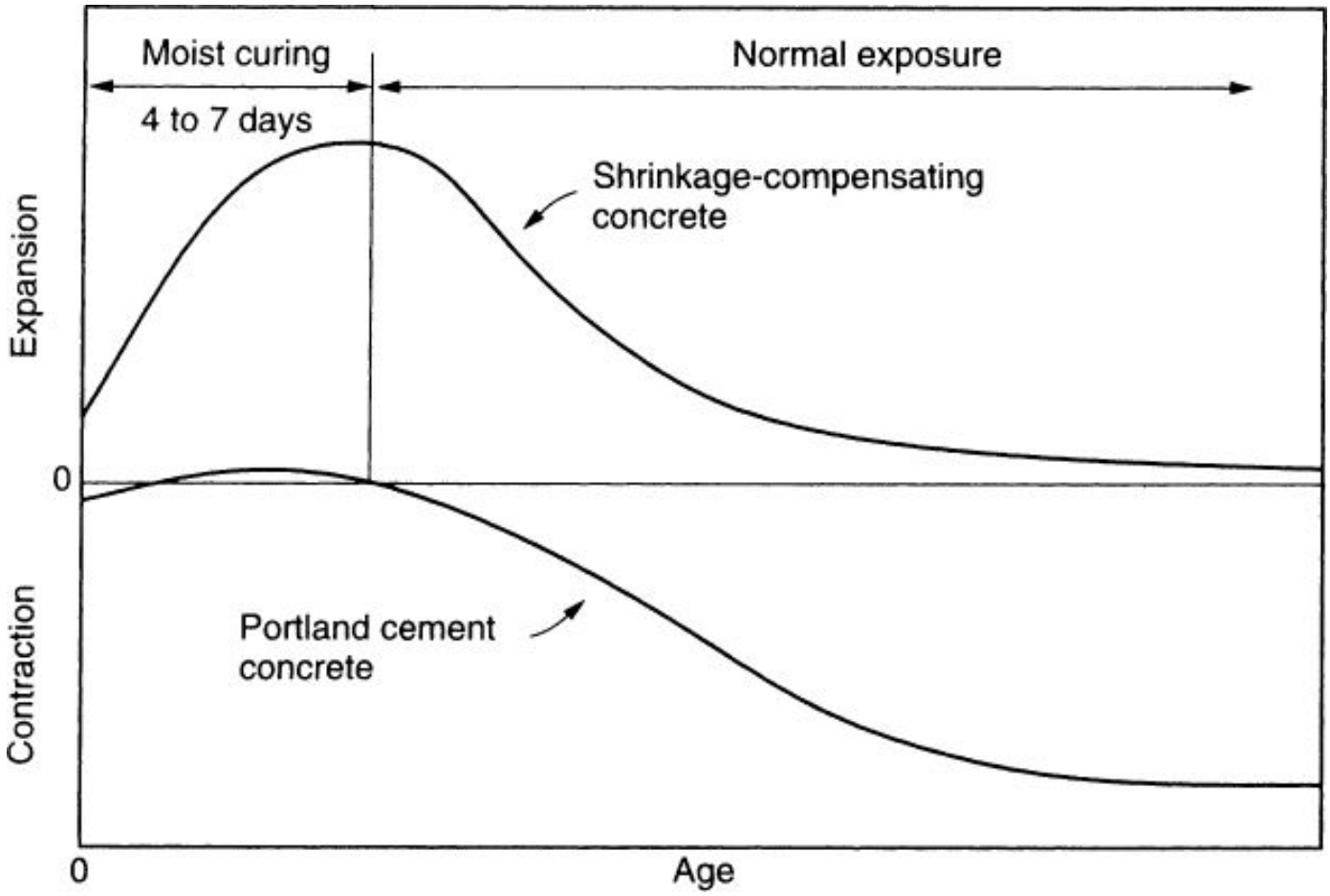


Fig. 9.27. Diagrammatic representation of length changes of shrinkage-compensating and Portland cement concretes (based on ref. 9.91)

Types of expansive cements

Early development of expansive cements took place in Russia and in France, where Lossier[9.2] used a mixture of Portland cement, an expanding agent, and a stabilizer. The expanding agent was obtained by burning a mixture of gypsum, baux-

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ite, and chalk, which form calcium sulfate and calcium aluminate (mainly C_5A_3). In the presence of water, these compounds react to form calcium sulfoaluminate hydrate (ettringite), with an accompanying expansion of the cement paste. The stabilizer, which is blastfurnace slag, slowly takes up the excess calcium sulfate and brings expansion to an end.

Nowadays, three main types of expansive cement are produced, but only one, Type K, is commercially available in the United States. ASTM C 845-04 classifies expansive cements, collectively referred to as Type E-1, according to the expansive agent used with Portland cement and calcium sulfate. In each case, the agent is a source of reactive aluminate which combines with the sulfates in the Portland cement to form expansive ettringite; for instance, in Type K cement, the reaction is

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$$4CaO \cdot 3Al_2O_3 \cdot SO_3 + 8[CaO \cdot SO_3 \cdot 2H_2O] + 6[CaO \cdot H_2O] + 74H_2O \rightarrow 3[3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O].$$

The resulting compound is known as ettringite.

Calcium sulfate reacts rapidly with $4CaO \cdot 3Al_2O_3 \cdot SO_3$ because it exists in a separate form,[9.85] unlike C_3A which is part of the Portland cement clinker.

Whereas the formation of ettringite in mature concrete is harmful (see p. 511), a controlled formation of ettringite in the early days after placing of concrete is used to achieve a shrinkage-compensating effect.

The three types of expansive cement recognized by ACI 223R-93[9.91] and by ASTM C 845-04, are:

Type K which contains $4CaO \cdot 3Al_2O_3 \cdot SO_3$ and uncombined CaO,

Type M which contains calcium aluminates CA and $C_{12}A_7$, and

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Type S	which contains C_3A in excess of the amount normally present in Portland cement.

In addition, in Japan, there is produced an expansive cement which uses specially processed calcium oxide[9.8] to produce free-lime expansion. This cement is called Type O.

Type K cement is produced by integral burning of the components or by intergrinding. It is also possible, as is done in Japan,[9.8] to add the expansive component at the concrete batching plant.

Special expansive cements, containing high-alumina cement, for particular purposes where an extremely high expansion is required, can also be produced.[9.92]

Shrinkage-compensating concrete

The expansion of cement paste resulting from the formation of ettringite begins as soon as water has been added to the mix, but only *restrained*

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expansion is beneficial, and no restraint is offered while concrete is in the plastic state or while it has negligible strength. For this reason, prolonged mixing[9.86] and delay before placing of concrete containing expansive cement should be avoided.

On the other hand, delayed expansion in concrete in service may prove disruptive, as is the case with external sulfate attack (see p. 511). It is, therefore, important that ettringite formation ceases after several days, and this happens when either SO_3 or $A1_2O_3$ has become exhausted.

ASTM C 845-04 prescribes a maximum 7-day expansion of mortar of between 400×10^{-6} and 1000×10^{-6}; the 28-day expansion must not exceed the 7-day expansion by more than 15 per cent. The latter value is a check on delayed expansion.

Because the formation of ettringite requires a large amount of water, wet curing of concrete made with expansive cement is necessary for full

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benefits of the use of such cement to be reaped.[9.87]

Information on the use of expansive cements so as to obtain shrinkage-compensating concrete is given in ACI 223R-93[9.91] but some features of this type of concrete merit mention here. Its water requirement is about 15 per cent higher than when Portland cement only is used. However, as some of this additional water becomes combined very early, the strength of concrete is little affected.[9.91] Another way of representing the situation is to say that, at the same water/cement ratio, concrete made with Type K expansive cement has a 28-day compressive strength some 25 per cent higher than concrete made with Portland cement only.[9.4,9.85]

At a given water content, the workability of expansive cement concrete is lower and the slump loss is greater.[9.86]

The usual admixtures can be used in shrinkage-compensating concrete but trial mixes

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are necessary because some admixtures, especially air-entraining ones, may not be compatible with certain expansive cements.[9.55,9.86]

Because expansive cement has a large content of calcium sulfate, which is softer than Portland cement clinker, the cement has a high specific surface, typically 430 kg/m^2. Excessive fineness, by promoting rapid hydration, may lead to premature expansion,[9.91] which is ineffective because very young concrete is unable to offer restraint. The expansion is greater the higher the cement content of the concrete and the higher the modulus of elasticity of the aggregate[9.3] because the aggregate offers restraint to the expansion of the cement paste. ASTM 878-09 prescribes a test method for restrained expansion of shrinkage-compensating concrete. This test can be used to study the effects of various factors on expansion.

Silica fume can be incorporated into shrinkage-compensating concrete in order to control excessive expansion.[9.90] Tests on Type K ce-

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ment paste[9.89] have shown that silica fume in the mix accelerates expansion but the expansion stops before $CaO \cdot 3Al_2O_3 \cdot SO_3$ has been used up, probably due to a lowering of the pH. The absence of long-term expansion is desirable and shortening the wet-curing period to 4 days is convenient.

If, following the expansive reactions, the cement is undersulfated, the concrete is vulnerable to sulfate attack (see p. 509); this may be the case with Type M and Type S cements.[9.4]

Creep of concrete*

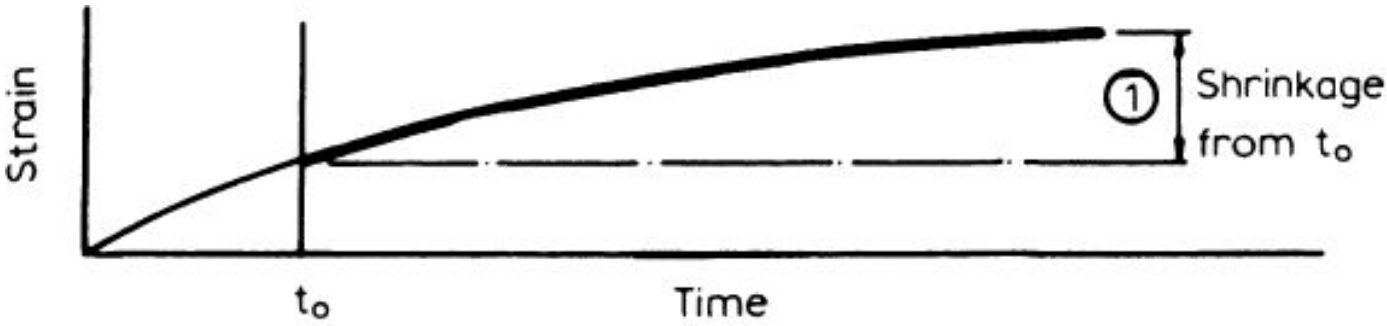
*For fuller treatment of this topic, see A. M. Neville, W. Dilger and J. J. Brooks, *Creep of Plain and Structural Concrete* (London, Construction Press, Longman Group, 1983).

We have seen that the relation between stress and strain for concrete is a function of time: the gradual increase in strain with time under load is

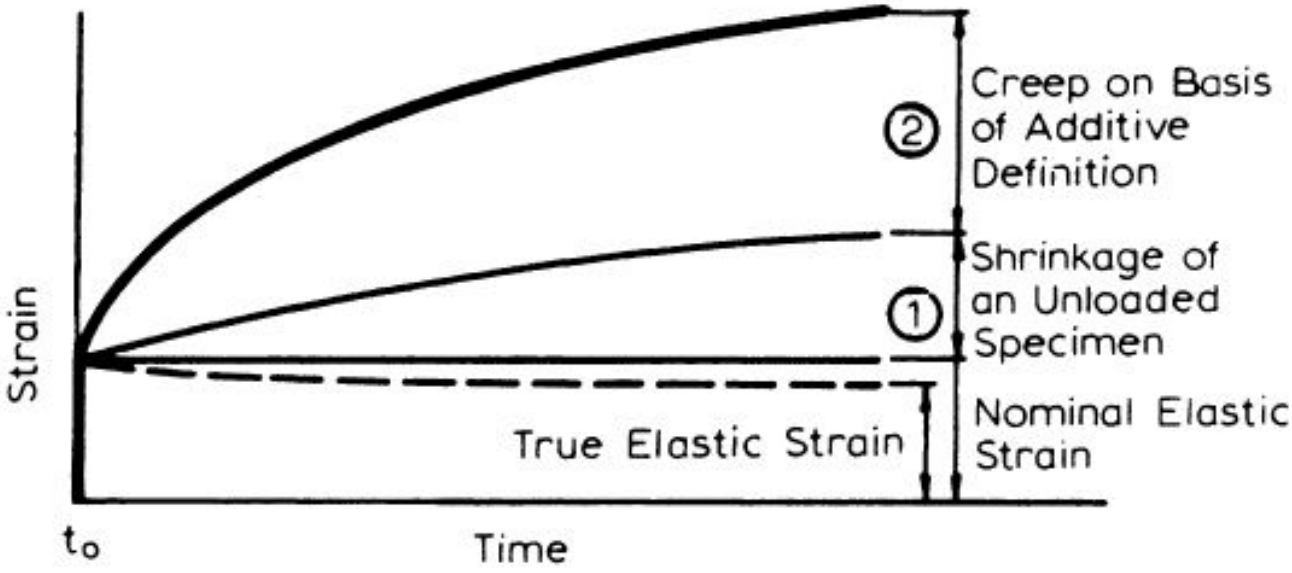
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due to creep. Creep can thus be defined as the increase in strain under a sustained stress (Fig. 9.28) and, because this increase can be several times as large as the strain on loading, creep is of considerable importance in structures.

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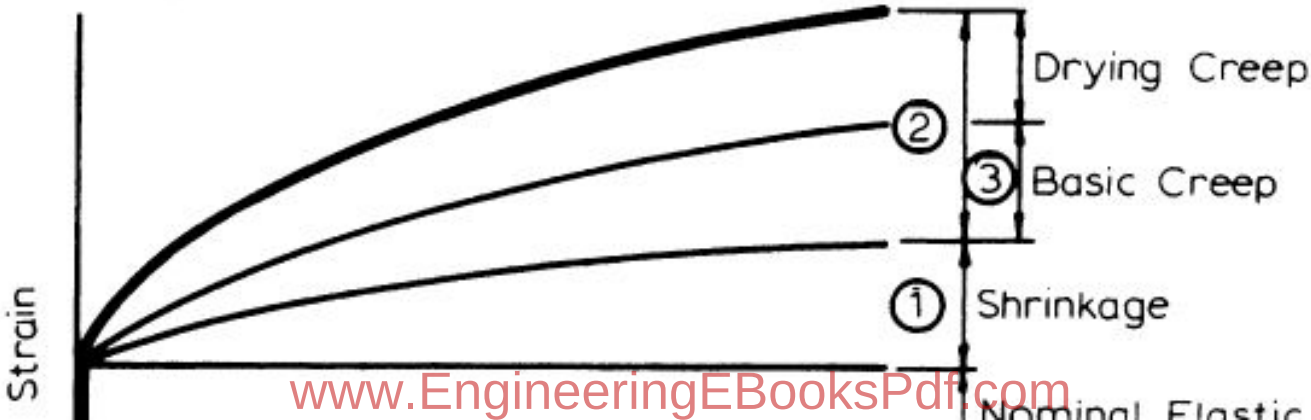
a) Shrinkage of an Unloaded Specimen



b) Change in Strain of Loaded and Drying Specimen



c) Creep of a Loaded Specimen in Hygral Equilibrium with the Ambient Medium



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Fig. 9.28. Time-dependent deformations in concrete subjected to a sustained load

Creep may also be viewed from another standpoint: if the restraint is such that a stressed concrete specimen is subjected to a constant strain, creep will manifest itself as a progressive decrease in stress with time.[9.107] This form of relaxation is shown in Fig. 9.29.

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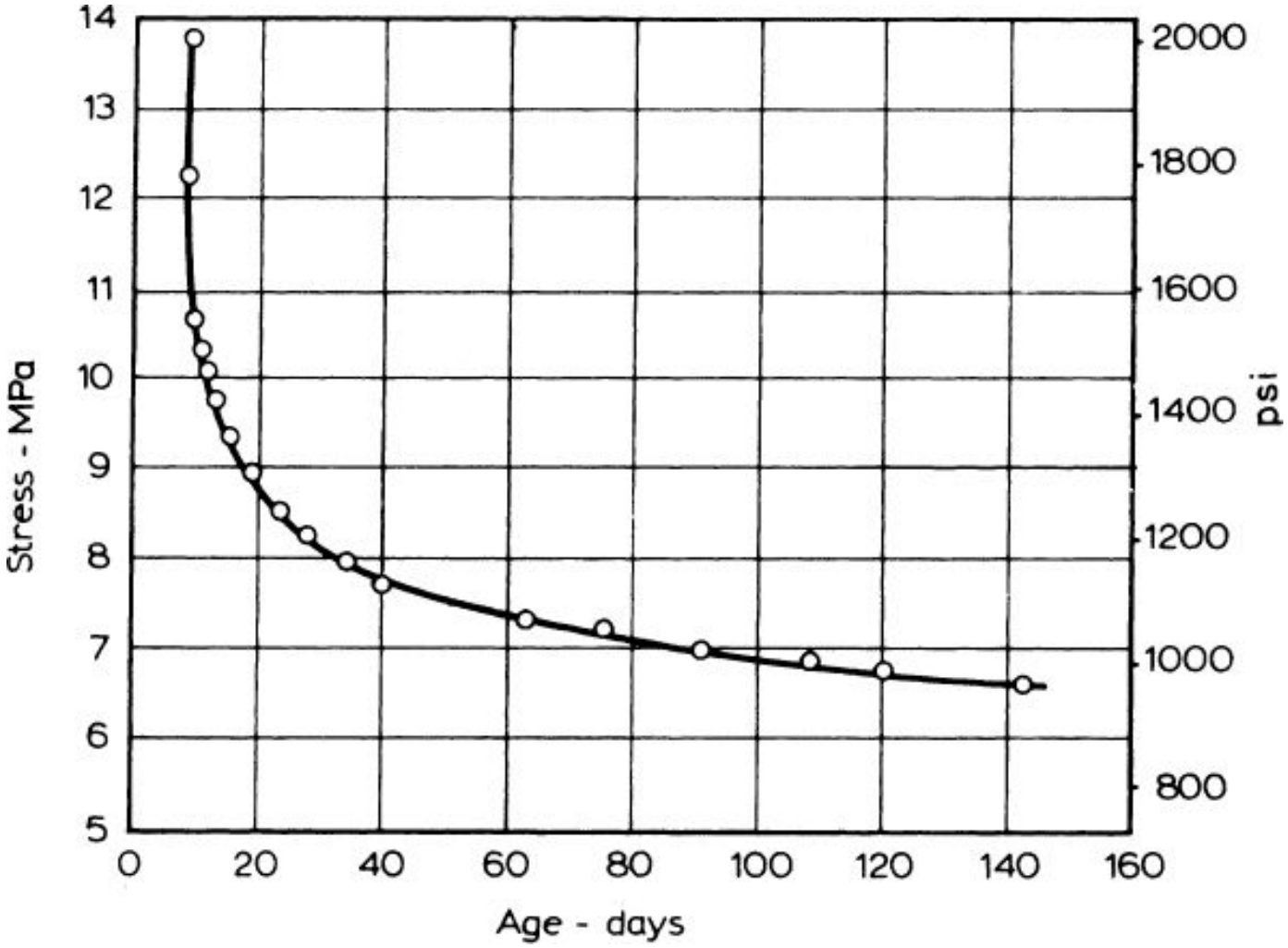


Fig. 9.29. Relaxation[9.107] of stress under a constant strain of 360×10^{-6}

Under normal conditions of loading, the instantaneous strain recorded depends on the speed of application of the load and includes thus not only the elastic strain but also some creep. It is difficult to differentiate accurately between the immediate elastic strain and early creep, but this is not of practical importance as it is the total

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strain induced by the application of load that matters. Because the modulus of elasticity of concrete increases with age, the elastic deformation gradually decreases and, strictly speaking, creep should be taken as strain in excess of the elastic strain at the time at which creep is being determined (Fig. 9.28). Often, the modulus of elasticity is not determined at different ages, and creep is simply taken as an increase in strain above the *initial* elastic strain. This alternative definition, although theoretically less correct, does not introduce a serious error and is often more convenient to use except in rigorous analysis.

So far, we have considered the creep of concrete stored under such conditions that no shrinkage or swelling takes place. If a specimen is drying while under load, it is usually assumed that creep and shrinkage are additive; creep is thus calculated as the difference between the total time-deformation of the loaded specimen and the shrinkage of a similar unloaded specimen stored under the same conditions through the same peri-

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od (Fig. 9.28). This is a convenient simplification but, as shown on p. 460, shrinkage and creep are not independent phenomena to which the principle of superposition can be applied, and in fact the effect of shrinkage on creep is to increase the magnitude of creep. In the case of many actual structures, however, creep and shrinkage occur simultaneously and the treatment of the two together is, from the practical standpoint, often convenient.

For this reason, and also because the great majority of the available data on creep were obtained on the assumption of the additive properties of creep and shrinkage, the discussion in this chapter will, for the most part, consider creep as a deformation in excess of shrinkage. However, where a more fundamental approach is warranted, distinction will be made between creep of concrete under conditions of no moisture movement to or from the ambient medium (*true* or *basic creep*) and the additional creep caused by dry-

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ing (*drying creep*). The terms and definitions involved are illustrated in Fig. 9.28.

If a sustained load is removed, the strain decreases immediately by an amount equal to the elastic strain at the given age, generally lower than the elastic strain on loading. This *instantaneous recovery* is followed by a gradual decrease in strain, called *creep recovery* (Fig. 9.30). The shape of the creep recovery curve is rather like that of the creep curve, but the recovery approaches its maximum value much more rapidly.[9.108] The recovery of creep is not complete, and creep is not a simply reversible phenomenon, so that any sustained application of load, even only over a period of a day, results in a residual deformation. Creep recovery is of importance in predicting deformation of concrete under a stress which varies with time.

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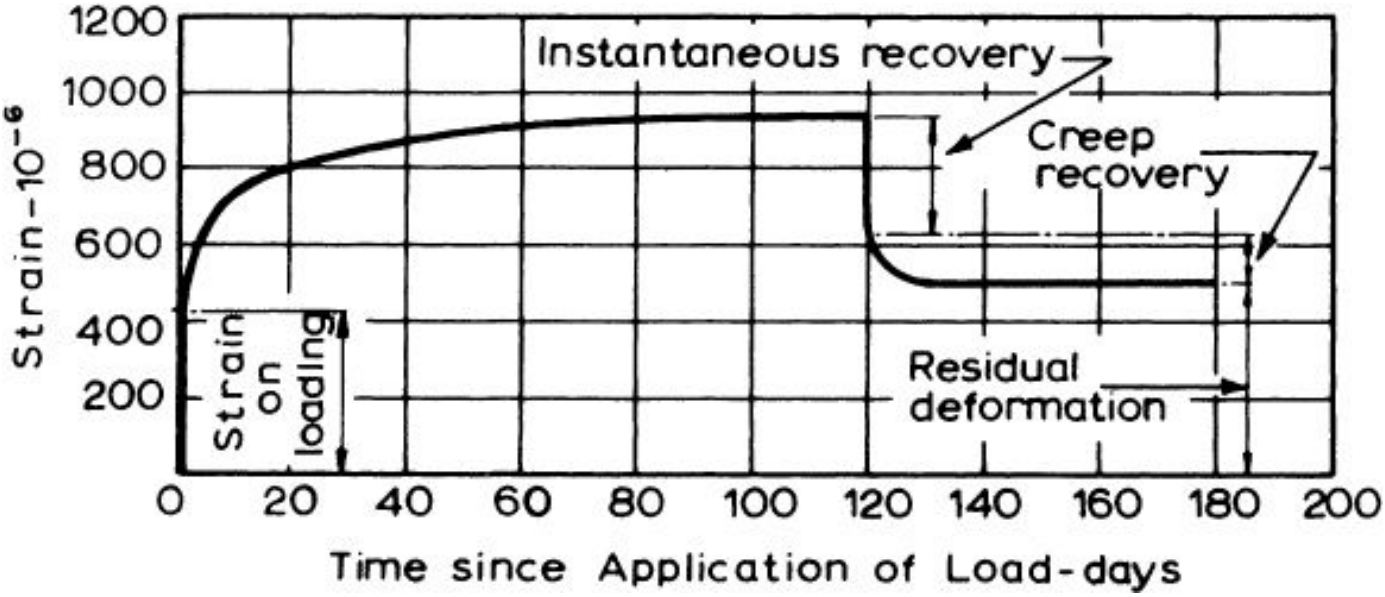


Fig. 9.30. Creep and creep recovery of a mortar specimen, stored in air at a relative humidity of 95 per cent, subjected to a stress of 14.8 MPa (2150 psi) and then unloaded[9.108]

Factors influencing creep

In most investigations, creep has been studied empirically in order to determine how it is affected by various properties of concrete. A difficulty in interpreting many of the available data arises from the fact that, in proportioning concrete, it is not possible to change one factor without altering also at least one other. For instance, the richness and the water/cement ratio of a mix of a given

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workability vary at the same time. Certain influences are, however, apparent.

Some of these arise from the intrinsic properties of the mix, others from external conditions. First of all, it should be noted that it is really the hydrated cement paste which undergoes creep, the role of the aggregate in concrete being primarily that of restraint; the usual normal weight aggregates are not liable to creep under the stresses existing in concrete. The situation is thus similar to that in the case of shrinkage (see p. 430). Creep is, therefore, a function of the volumetric content of cement paste in concrete, but the relation is not linear. It has been shown[9.109] that creep of concrete, c, the volumetric content of aggregate, g, and the volumetric content of unhydrated cement, u, are related by

$$\log \frac{c_p}{c} = \alpha \log \frac{1}{1-g-u}$$

where c_p is creep of neat cement paste of the same quality as used in concrete, and

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$$\alpha = \frac{3(1-\mu)}{1+\mu+2(1-2\mu_a)\dfrac{E}{E_a}}.$$

Here, μ_a = Poisson's ratio of aggregate, μ = Poisson's ratio of surrounding material (concrete), E_a = modulus of elasticity of aggregate, and E = modulus of elasticity of the surrounding material. This relation applies to concrete made both with normal weight aggregate and with lightweight aggregate.[9.110]

Figure 9.31 illustrates the relation between creep of concrete and its aggregate content (the volume of unhydrated cement being ignored). It may be noted that, in the majority of the usual mixes, the variation in the aggregate content is small, but an increase in the aggregate content by volume from 65 to 75 per cent can decrease creep by 10 per cent.

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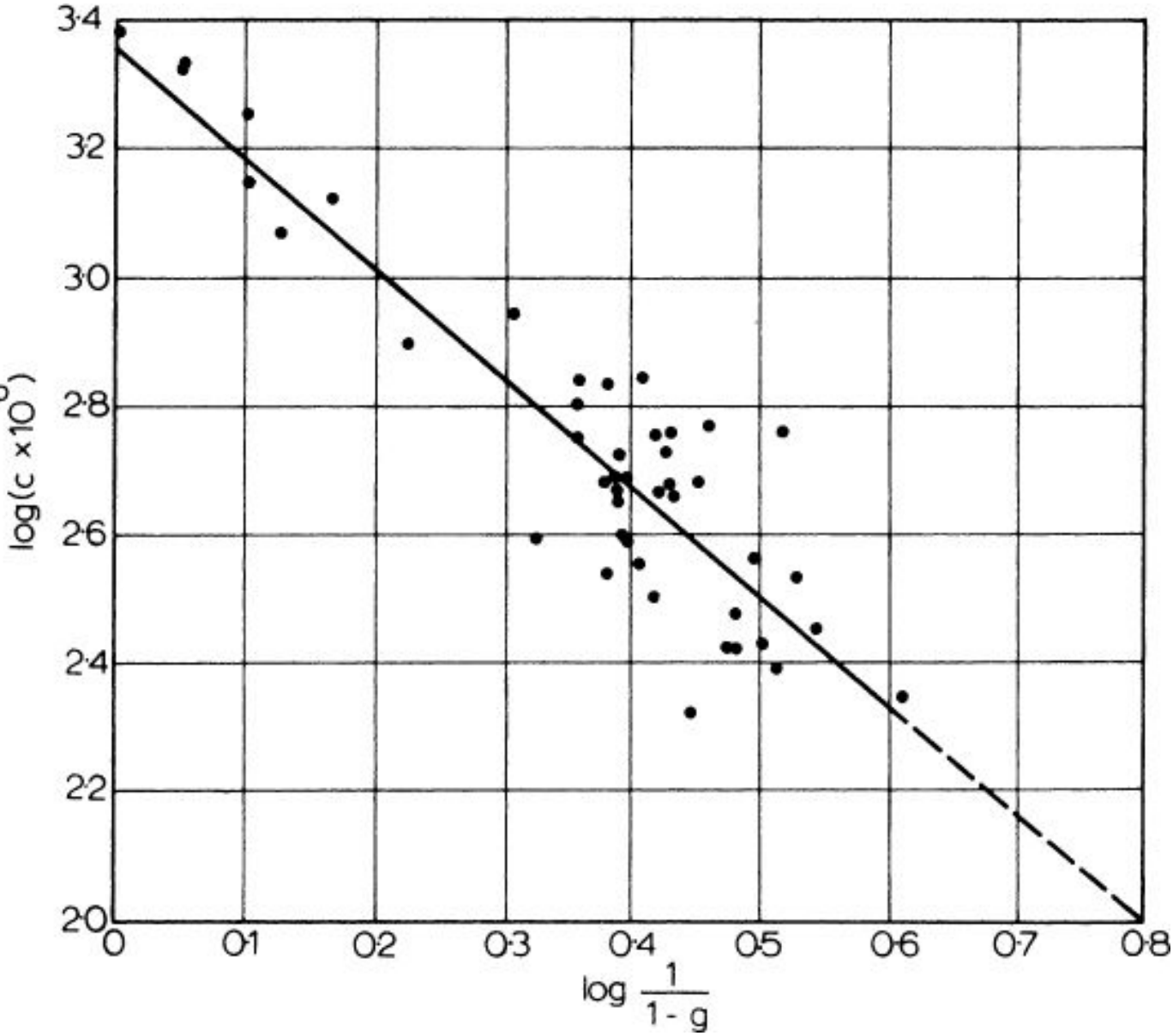


Fig. 9.31. Relation between creep *c* after 28 days under load and content of aggregate *g* for wet-stored specimens loaded at the age of 14 days to a stress/strength ratio of 0.50[9.109]

The grading, maximum size, and shape of the aggregate have been suggested as factors in creep. However, their main influence lies in the

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effect that they have directly or indirectly on the aggregate content,[9.109] providing that full consolidation of concrete has been achieved in all cases.

There are certain physical properties of aggregate which influence the creep of concrete. The modulus of elasticity of aggregate is probably the most important factor. The higher the modulus the greater the restraint offered by the aggregate to the potential creep of the hydrated cement paste; this is evident from the expression for α above.

Porosity of aggregate has also been found to influence the creep of concrete but, because aggregates with a higher porosity generally have a lower modulus of elasticity, it is possible that porosity is not an independent factor in creep. On the other hand, it can be visualized that the porosity of aggregate, and even more so its absorption, play a direct role in the transfer of moisture within concrete; this transfer may be associated with creep in that it produces conditions conducive to the development of drying creep. This may be

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the explanation for the high initial creep occurring with some lightweight aggregates batched in a dry condition.

Because of the great variation in aggregate within any mineralogical and petrological type, it is not possible to make a general statement about the magnitude of creep of concrete made with aggregates of different types. However, the data of Fig. 9.32 are of considerable importance: after 20 years' storage at a relative humidity of 50 per cent, concrete made with sandstone aggregate exhibited creep more than twice as great as concrete made with limestone. An even greater difference between the creep strains of concretes made with different aggregates was found by Rüsch *et al.*[9.111] After 18 months under load at a relative humidity of 65 per cent, the maximum creep was five times the minimum value, the aggregates in the increasing order of creep being: basalt; quartz; gravel, marble and granite; and sandstone.

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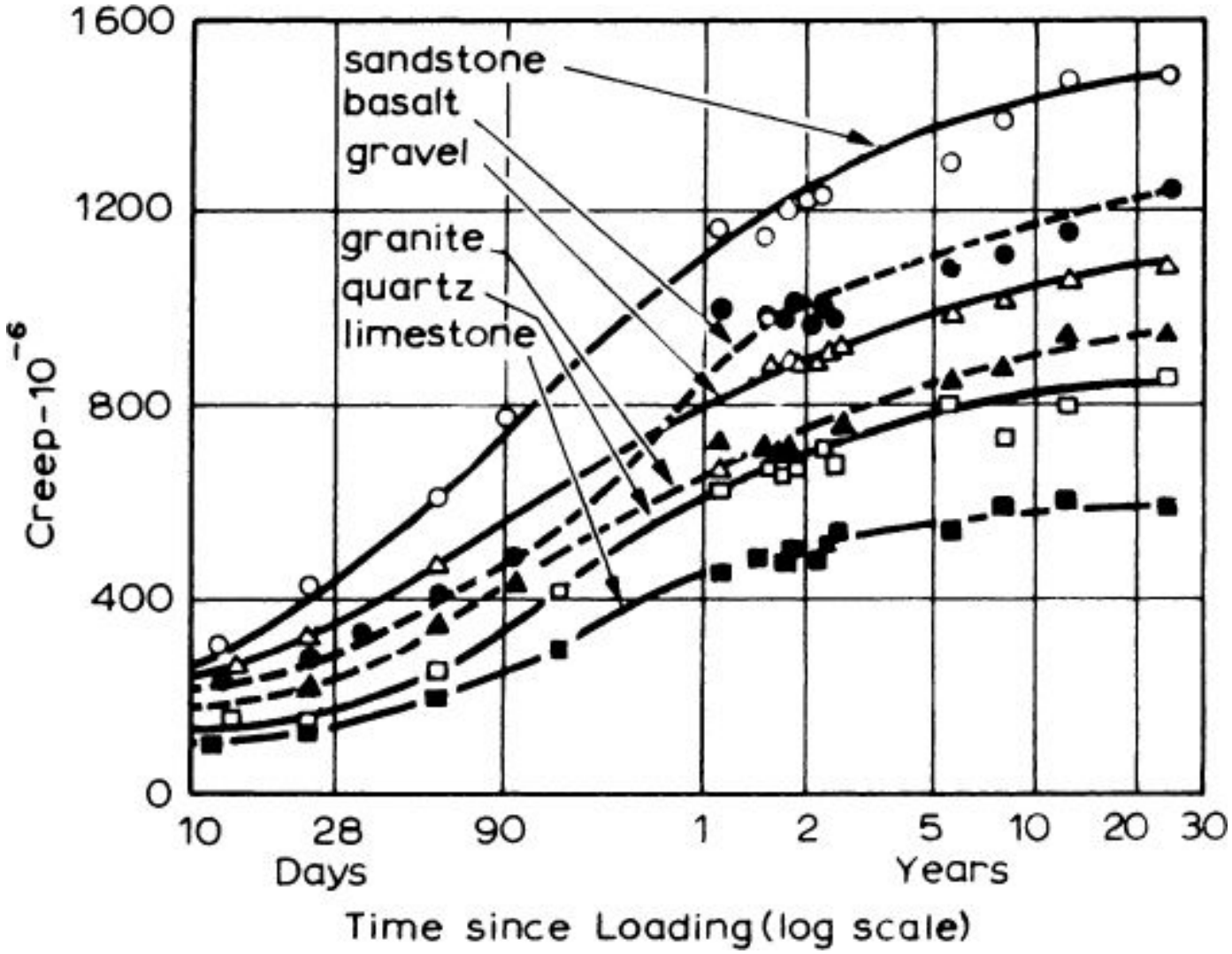


Fig. 9.32. Creep of concretes of fixed proportions but made with different aggregates, loaded at the age of 28 days, and stored in air at 21 °C (70 °F) and a relative humidity of 50 per cent[9.24]

There is no fundamental difference between normal and lightweight aggregates as far as the creep properties are concerned, and the higher

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creep of concretes made with lightweight aggregates reflects only the lower modulus of elasticity of that aggregate. The rate of creep of lightweight aggregate concrete decreases with time more slowly than in normal weight concrete. As a general rule, it can be stated that creep of structural quality lightweight aggregate concrete is about the same as that of concrete made with ordinary aggregate. (It is important in any comparison that the aggregate contents do not differ widely between the lightweight and ordinary concretes.) Furthermore, because the elastic deformation of lightweight aggregate concrete is usually larger than in ordinary concrete, the ratio of creep to elastic deformation is smaller for lightweight aggregate concrete.[9.112]

Influence of stress and strength

At this stage, it may be appropriate to consider the influence of stress on creep. There is a direct proportionality between creep and the applied stress,[9.113] with a possible exception of speci-

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mens loaded at a very early age. There is no lower limit of proportionality because concrete undergoes creep even at a very low stress. The upper limit of proportionality is reached when severe microcracking develops in concrete; this occurs at a stress, expressed as a fraction of strength, which is lower in a more heterogeneous material. Thus, the limit in concrete is usually between 0.4 and 0.6, but occasionally as low as 0.3 or as high as 0.75; the latter value applies to high strength concrete.[9.66] In mortar, the limit is in the region of 0.80 to 0.85.[9.112]

It appears safe to conclude that, within the range of stresses in structures in service, the proportionality between creep and stress holds good, and creep expressions assume this to be the case. Creep recovery is also proportional to the stress previously applied.[9.114]

Above the limit of proportionality, creep increases with an increase in stress at an increasing rate, and there exists a stress/strength ratio above which creep produces *time failure*. This stress/

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strength ratio is in the region of 0.8 to 0.9 of the short-term static strength. Creep increases the total strain until this reaches a limiting value corresponding to the ultimate strain of the given concrete. This statement implies a limiting strain concept of failure, at least in the hardened cement paste (see p. 294).

The strength of concrete has a considerable influence on creep: within a wide range, creep is inversely proportional to the strength of concrete at the time of application of the load. This is indicated, for instance, in the data of Table 9.5. It is thus possible to express creep as a linear function of the stress/strength ratio[9.115] (Fig. 9.33). This proportionality has been widely confirmed. It may not be a fundamental relationship but it is a most convenient one because, in practice, the strength of concrete is specified and the stress under a sustained load is calculated by the designer. For this reason, the stress/strength ratio approach is thought more practical than the consideration of type of cement, water/cement ratio,

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and age. In our approach, while we recognize the role of the water/cement ratio, we utilize the fact that, for the same stress/strength ratio, creep is sensibly independent of the water/cement ratio. Likewise, we ignore the age as such, its influence being mainly in increasing the strength of concrete. It may be appropriate to note that even very old concrete undergoes creep, as demonstrated by tests on 50-year-old concrete.[9.116]

Table 9.5. Ultimate Specific Creep of Concretes of Different Strengths Loaded at the Age of 7 Days

Compressive strength of concrete		*Ultimate specific creep*		*Product of specific creep and strength* 10^{-3}
MPa	*psi*	10^{-6} *per MPa*	10^{-6} *per psi*	
14	2000	203	1.40	2.8
28	4000	116	0.80	3.2
41	6000	80	0.55	3.3
55	8000	58	0.40	3.2

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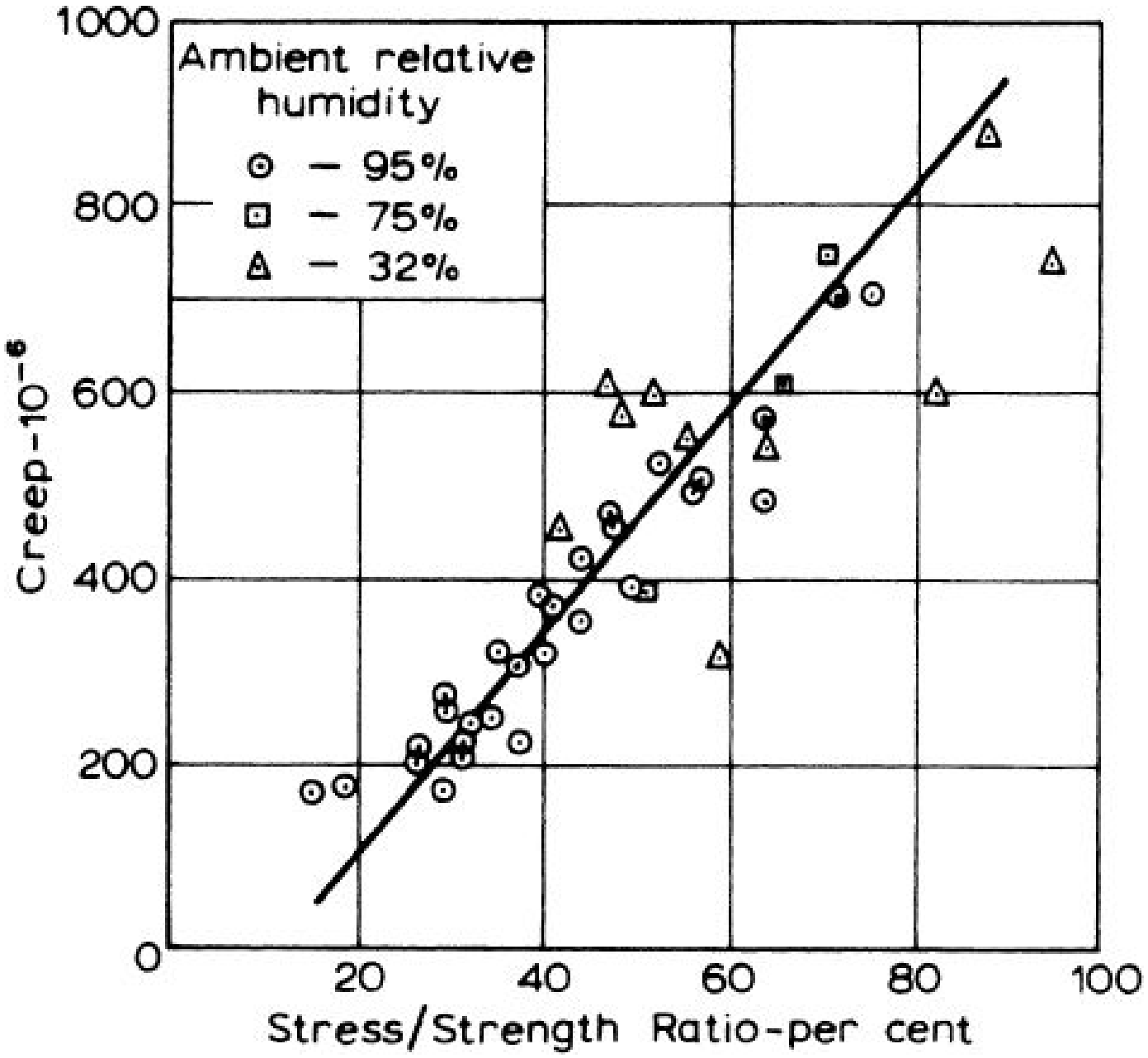


Fig. 9.33. Creep of mortar specimens cured and stored continuously at different humidities[9.117]

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Influence of properties of cement

The type of cement affects creep in so far as it influences the strength of the concrete at the time of application of the load. For this reason, any comparison of creep of concretes made with different cements should take into account the influence of the type of cement on the strength of concrete at the time of application of the load. On this basis, both Portland cements of different types and high-alumina cement lead to sensibly the same creep,[9.123,9.124] but the rate of gain of strength has some effect as shown below.

Fineness of cement affects the strength development at early ages and thus influences creep. It does not seem, however, that fineness *per se* is a factor in creep: contradictory results may be due to the indirect influence of gypsum. The finer the cement the higher its gypsum requirement, so that re-grinding a cement in the laboratory without the addition of gypsum produces an improperly retarded cement, which exhibits high shrinkage and high creep.[9.28] Extremely fine cements, with a

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specific surface up to 740 kg/m^2, lead to a higher early creep but to lower creep after one or two years under load.[9.41] This is probably due to the high gain of strength of the finest cement with resultant rapid drop in the actual stress/strength ratio.[9.133]

The change in strength of concrete while under load is of importance in evaluating the preceding statement that creep is not influenced by the type of cement. For the same stress/strength ratio at the time of application of load, creep is smaller the greater the relative increase in strength beyond the time of application of load.[9.133] Thus, creep increases in order for: low heat, ordinary, and rapid-hardening cements. There is no doubt, however, that for a constant applied stress (not stress/strength ratio) at a fixed (early) age, creep increases in order for: rapid-hardening, ordinary, and low heat cements. These two statements bring out clearly the need for a full qualification of information about factors in creep.

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The influence on creep of the strength of concrete at the time of application of load applies also when different cementitious materials are used. Otherwise, *quantitative* generalizations about creep of concretes containing fly ash or ground granulated blastfurnace slag are not possible because published literature reports investigations, each of which used specific and differing test conditions. Such data cannot be used to predict the creep of concrete at the structural design stage. All we can say reliably is that the *pattern* of development of creep and of creep recovery is not affected by the presence of Class C or Class F fly ash,[9.144,9.153] ground granulated blastfurnace slag,[9.151] or silica fume, or even combinations of these materials.

There may, however, be some influences on creep of the structure of the hydrated cement arising from inclusion of the various cementitious materials. The influence on drying creep, where permeability and diffusivity of the hydrated cement paste are relevant, may be different from the

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influence on basic creep. For instance, the use of blastfurnace slag leads to a lower basic creep but a higher drying creep.[9.14,9.125,9.152] It should be remembered that the various cementitious materials have different rates of hydration and therefore of gain of strength while the concrete is under load. The rate of gain of strength affects creep; this was referred to earlier in this section.

An example of the influence of hydration on creep is offered by tests of Buil and Acker,[9.150] who found that silica fume has no effect on basic creep but significantly reduces the drying creep. The explanation is likely to lie in the fact that the hydration reactions of silica fume reduce the amount of water available for movement out of the gel. Generally, because of the long-term hydration, and therefore increase in strength under sustained load, of concrete containing fly ash or ground granulated blastfurnace slag, the long-term rate of creep is reduced in such concrete.

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Creep of concrete made with expansive cement is larger than when Portland cement only is included in the mix.[9.156]

Water-reducing and set-retarding admixtures have been found to increase the basic creep in many, but not all, cases.[9.134,9.135] There are indications that lignosulfonate-based admixtures lead to a larger increase than carboxylic-acid-based admixtures.[9.71] For drying creep, no reliable pattern of influence of these admixtures has been established.[9.71] The same situation exists with respect to superplasticizers.[9.71] From this rather unsatisfactory situation, it follows that, if creep is of importance in a given structure, the influence of any admixture to be used should be carefully checked.

There are some general comments that should be made about differences in creep reported by various researchers. In a number of investigations, the differences in creep reported are of about the same magnitude as the scatter of results

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for any one set of tests. It is not reasonable, therefore, to accept these differences as significant and they cannot be used as a basis for prediction. Tests involving actual materials are necessary. These tests, which must be performed under conditions expected to apply in service, can be of short duration. Extrapolation using the expressions discussed on p. 470 can be used to estimate long-term creep.

Reverting to the relation between creep and the stress/strength ratio, we can note that because, for a *given* mix, strength and modulus of elasticity are related to one another, creep and modulus of elasticity are also related. Figure 9.34 shows experimental values of creep at any time t, against the ratio of the modulus of elasticity at the time t to the modulus at the time of application of load;[9.118] the ages at which the load was applied and at which creep was determined varied widely, but one mix only was used. The modulus at the time of application of the load gives an indication

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of the strength at that time, and the increase in the modulus reflects the duration of the load.

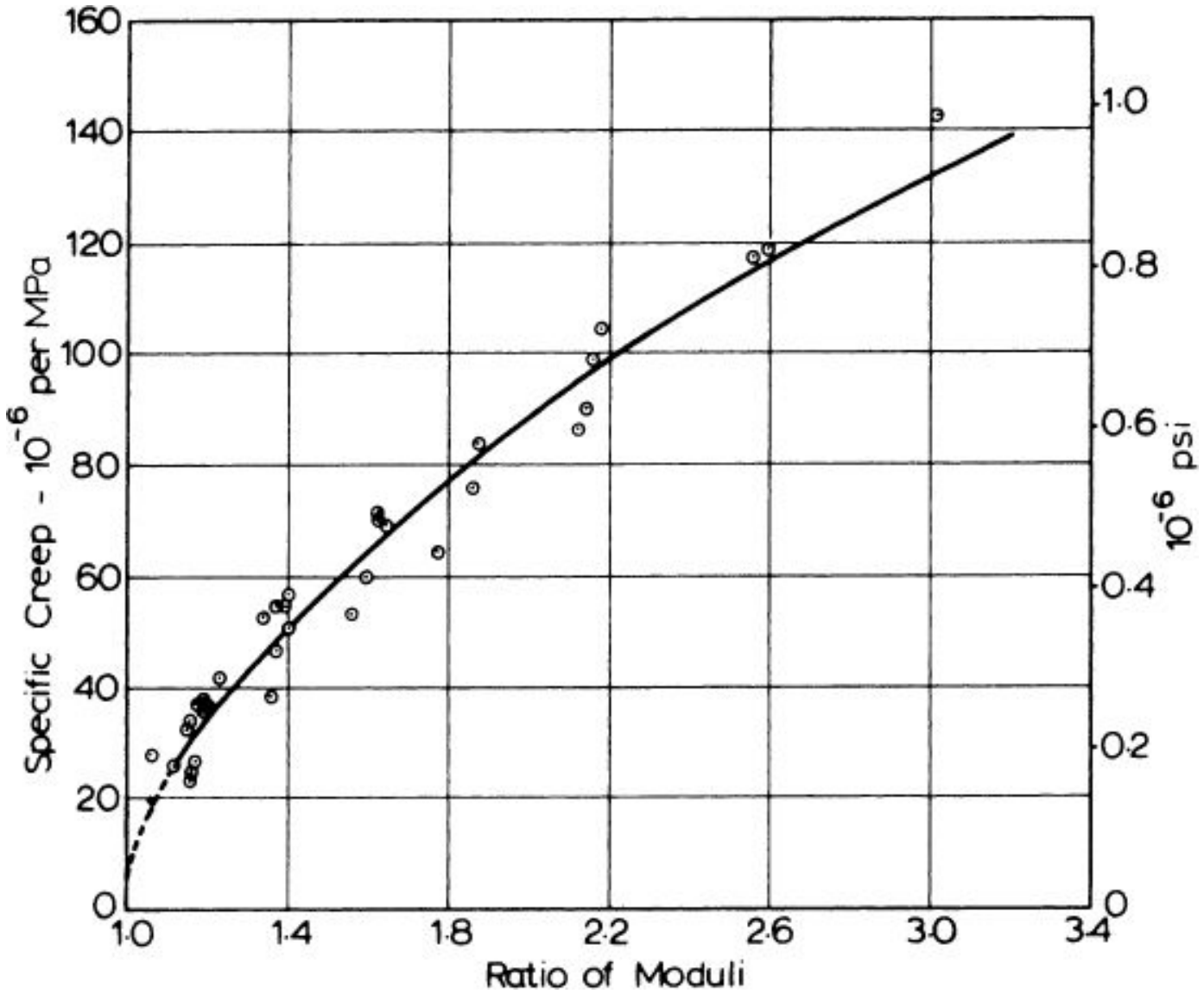


Fig. 9.34. Relation between creep at any time *t* and ratio of the modulus of elasticity of concrete at time *t* to the modulus at the time of application of the load; various concretes, ages at loading, and periods under load[9.118]

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Influence of ambient relative humidity

One of the most important external factors influencing creep is the relative humidity of the air surrounding the concrete. Taking a broad view, we can say that, for a given concrete, creep is higher the lower the relative humidity. This is illustrated in Fig. 9.35 for specimens cured at a relative humidity of 100 per cent and then loaded and exposed to different humidities. Such treatment results in a greatly varying shrinkage occurring in the different specimens during the early stages after the application of the sustained load. The rates of creep during that period vary correspondingly but, at later ages, the rates seem to be close to one another. Thus, drying while under load enhances creep of concrete, i.e. induces the additional drying creep (cf. Fig. 9.28). The influence of relative humidity is much smaller, or absent, in the case of specimens which have reached hygral equilibrium with the surrounding medium prior to the application of the load[9.117] (Fig. 9.35). Thus, in reality, it is not the relative

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humidity that influences creep but the process of drying, i.e. the occurrence of drying creep.

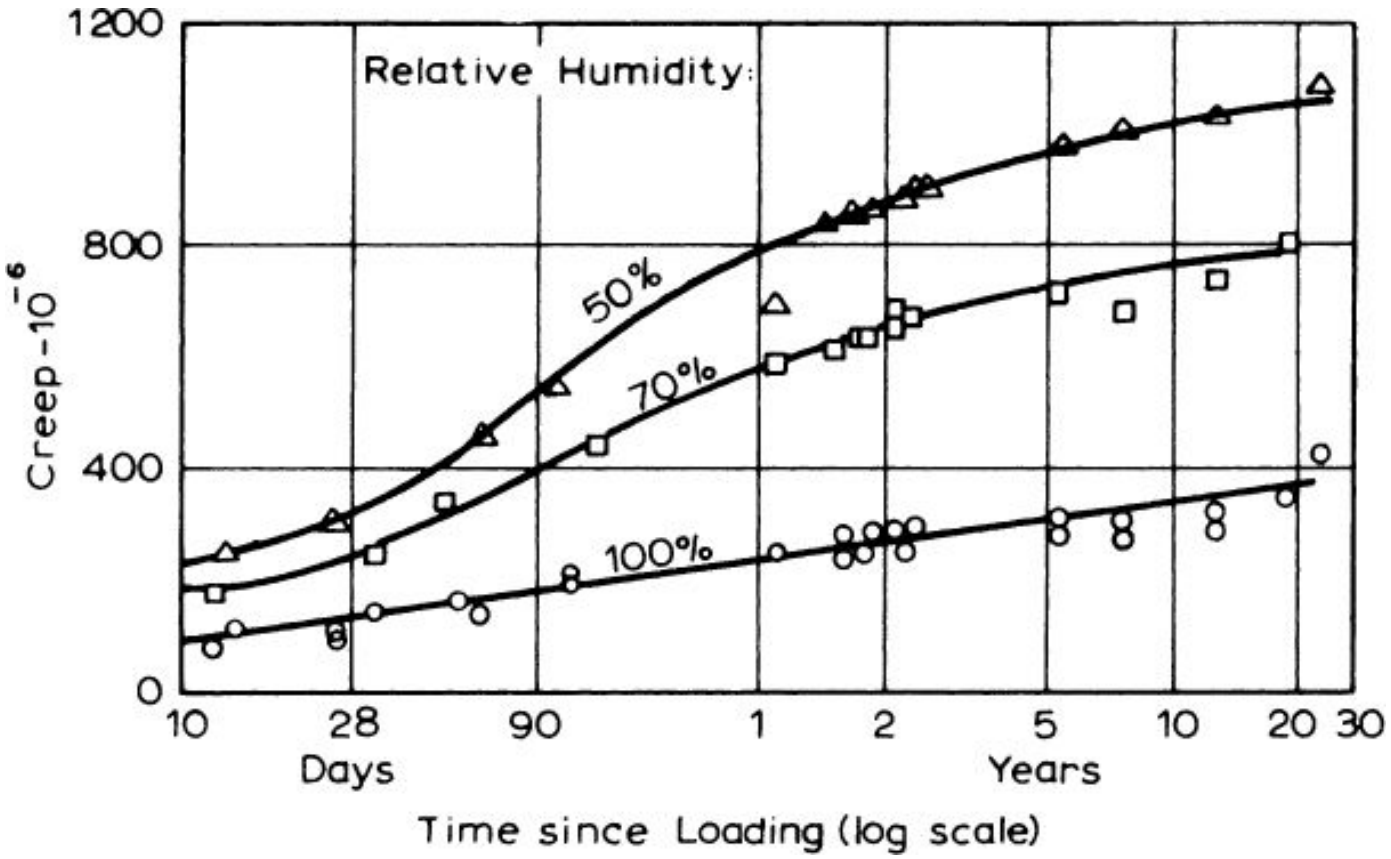


Fig. 9.35. Creep of concrete cured in fog for 28 days, then loaded and stored at different relative humidities[9.24]

Drying creep may be related to, or influenced by, the tensile stress induced in the outer part of a concrete specimen by restrained shrinkage and the resultant cracking.[9.149] The compressive stress arising from an applied compressive load

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cancels out this cracking.[9.148] Consequently, the actual shrinkage of a loaded specimen is larger than the measured shrinkage of a specimen which has undergone surface cracking. The approach of considering creep and shrinkage to be additive, therefore, assumes too small a value of shrinkage: the difference between this assumed shrinkage and the actual shrinkage in a loaded specimen represents drying creep. This hypothesis has not, however, been confirmed by tests on mortar[9.145] in which a large drying creep was observed in the absence of shrinkage cracking of unloaded companion specimens. Day and Illston[9.154] also found that very small specimens of hydrated cement paste undergo drying creep and concluded that drying creep is an intrinsic property of hydrated cement paste.

Bažant and Xi[9.157] suggested that, rather than drying creep, there exists stress-induced shrinkage caused by local movement of water between capillary pores and gel pores. However, until convincing evidence is available, the concept of dry-

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ing creep as defined in Fig. 9.28 should be retained.

At this stage, it is appropriate to note that concrete which exhibits high shrinkage shows generally also a high creep.[9.14] This does not mean that the two phenomena are due to the same cause, but they may both be linked to the same aspect of the structure of hydrated cement paste. It should not be forgotten that concrete cured and loaded at a constant relative humidity exhibits creep, and that creep produces no significant loss of water from the concrete to the surrounding medium;[9.120,9.121] nor is there any gain in mass during creep recovery.[9.121] (A small increase in mass occasionally observed during the period of creep or creep recovery may be due to carbonation.)

A further indication of the interrelation between shrinkage and creep is given in Fig. 9.36. Specimens which had been loaded for 600 days and then unloaded and allowed to recover their creep exhibited, on subsequent immersion in water, swelling proportional to the stress which

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had been removed over two years previously. The residual deformation after swelling shows a similar proportionality.

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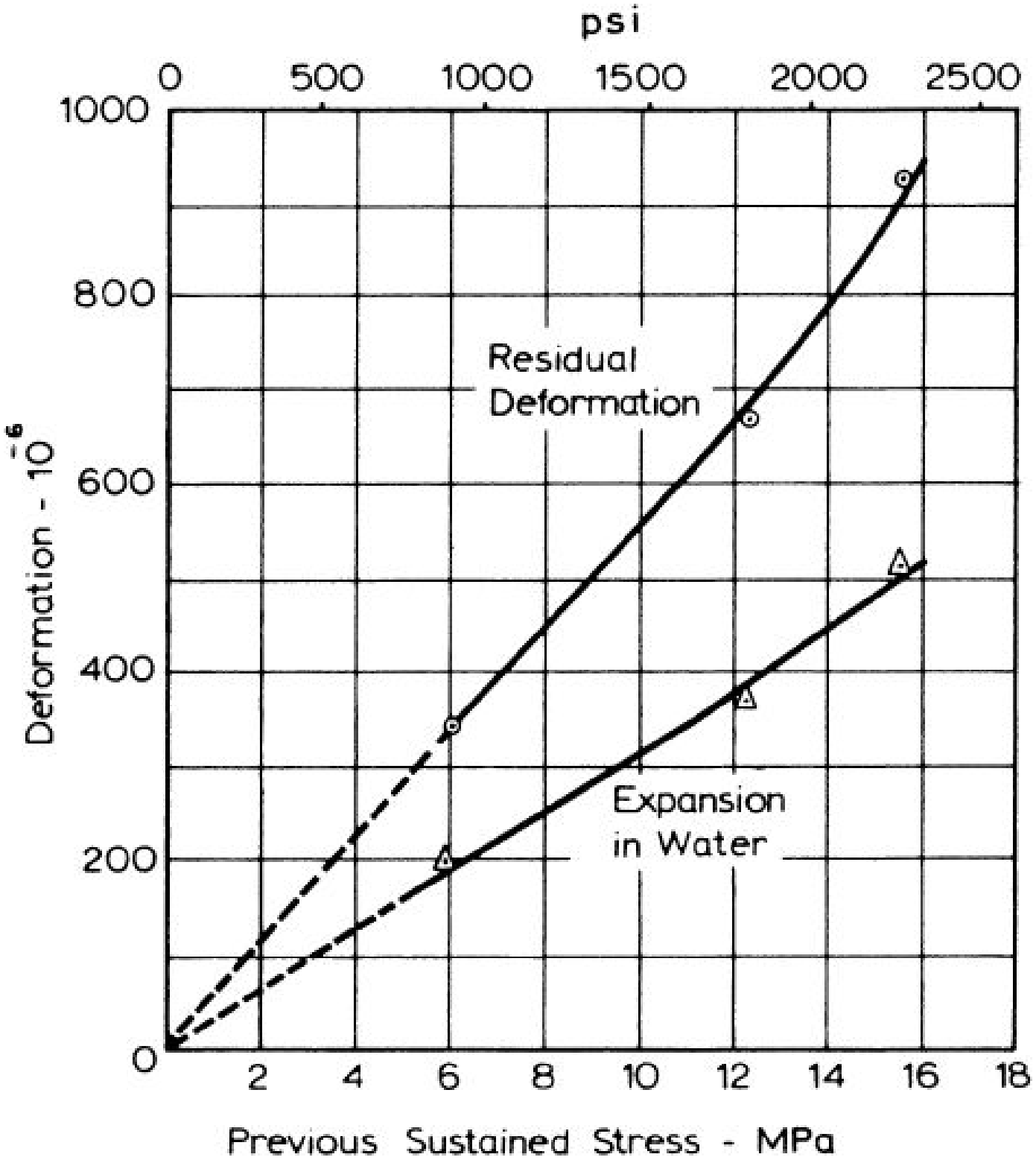


Fig. 9.36. Relation between original sustained stress and: (*a*) expansion in water, and (*b*) residual deformation of concrete[9.113]

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Figure 9.37 shows time deformation of loaded specimens stored alternately in water and in air with a relative humidity of 50 per cent. The ordinates represent the change in deformation from that existing after 600 days under load in air. It can be seen that, while in water, the loaded specimens show creep relative to the swelling of the unloaded specimen, but in air the change in deformation of all specimens is the same. The increase in creep on immersion of this old concrete in water may be due to the breaking of some of the bonds formed during the period of drying (cf. p. 443). Figure 9.38 shows the data of Fig. 9.37 plotted as a deformation relative to the deformation of the unloaded specimen. A practical conclusion from these observations is that alternating wetting and drying increases the magnitude of creep, so that results of laboratory tests may underestimate the creep under normal weather conditions.

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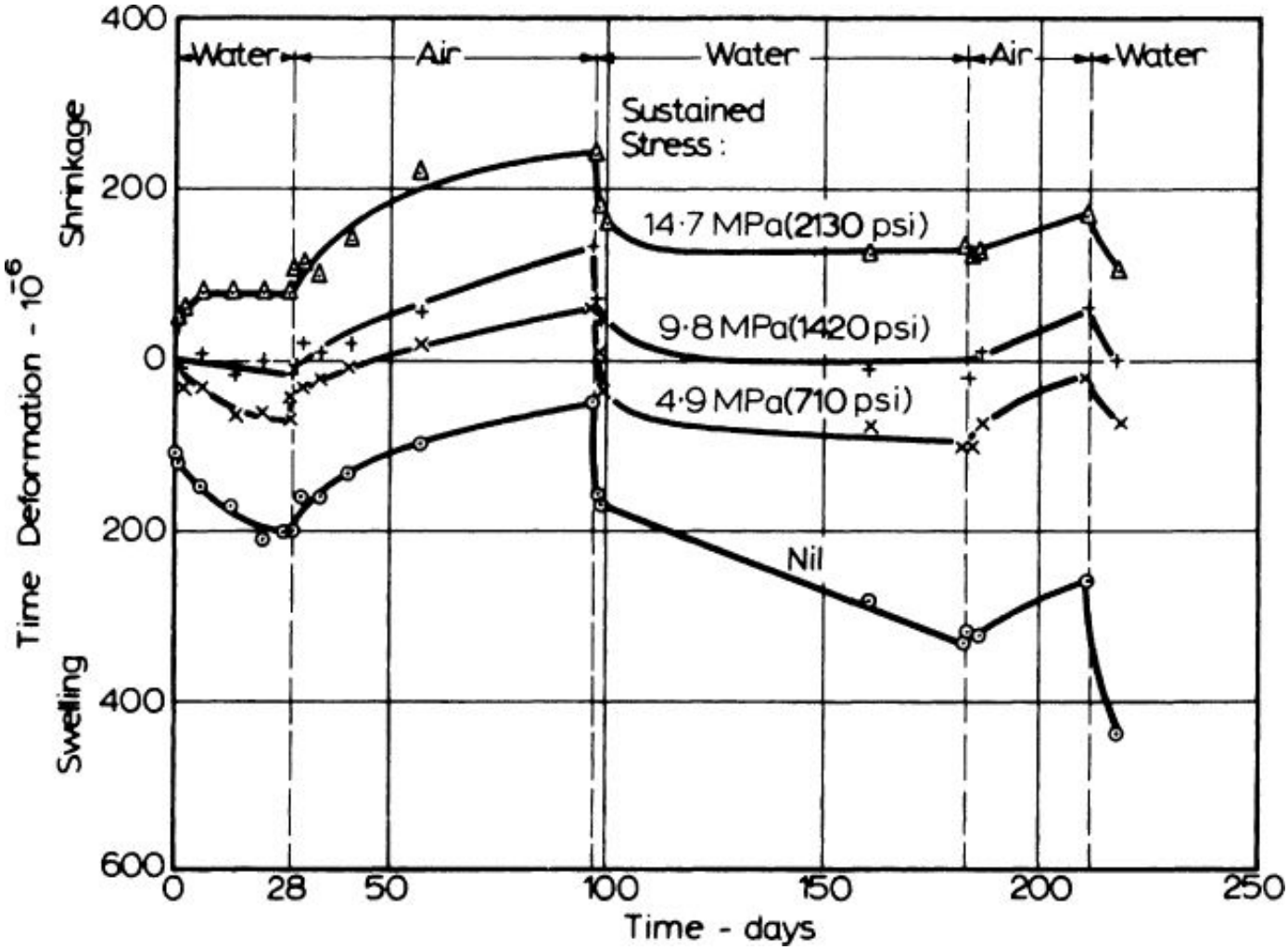


Fig. 9.37. Time deformation of concrete subjected to different stresses and stored alternately in water and in air at a relative humidity of 50 per cent.[9.14] Strains at origin of time (after 600 days under load in air):

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Stress, MPa	psi	Strain, 10^{-6}
0	0	280
4.9	710	1000
9.8	1420	1800
14.7	2130	2900

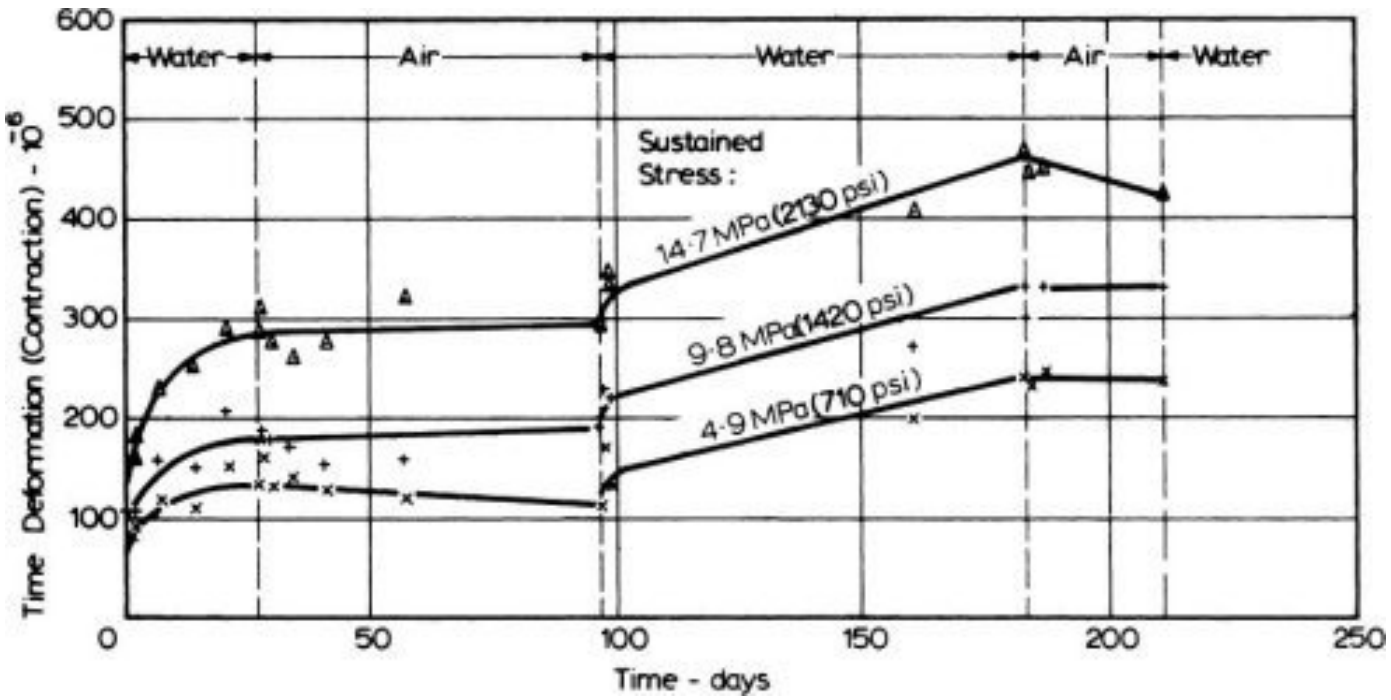


Fig. 9.38. Time deformation of loaded specimens of Fig. 9.37 plotted relative to the strain of the unloaded specimen[9.14]

Creep has been found to decrease with an increase in the size of the specimen. This may be due to the effects of shrinkage and to the fact that creep at the surface occurs under conditions of

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drying and is, therefore, greater than within the core of the specimen where the conditions approximate to mass curing. Even if, with time, drying reaches the core, it will have hydrated extensively and reached a higher strength, which leads to lower creep. In sealed concrete, no size effects can be present.

The size effect can best be expressed in terms of the volume/surface ratio of the concrete member; the relation is shown in Fig. 9.39. It can be seen that the actual shape of the specimen is of even lesser importance than in the case of shrinkage. Also, the decrease in creep with an increase in size is smaller than in the case of shrinkage (cf. Fig. 9.21). But the rates of gain of creep and of shrinkage are the same, indicating that both phenomena are the same function of the volume/surface ratio. These data apply to shrinkage and creep at 50 per cent relative humidity.[9.53]

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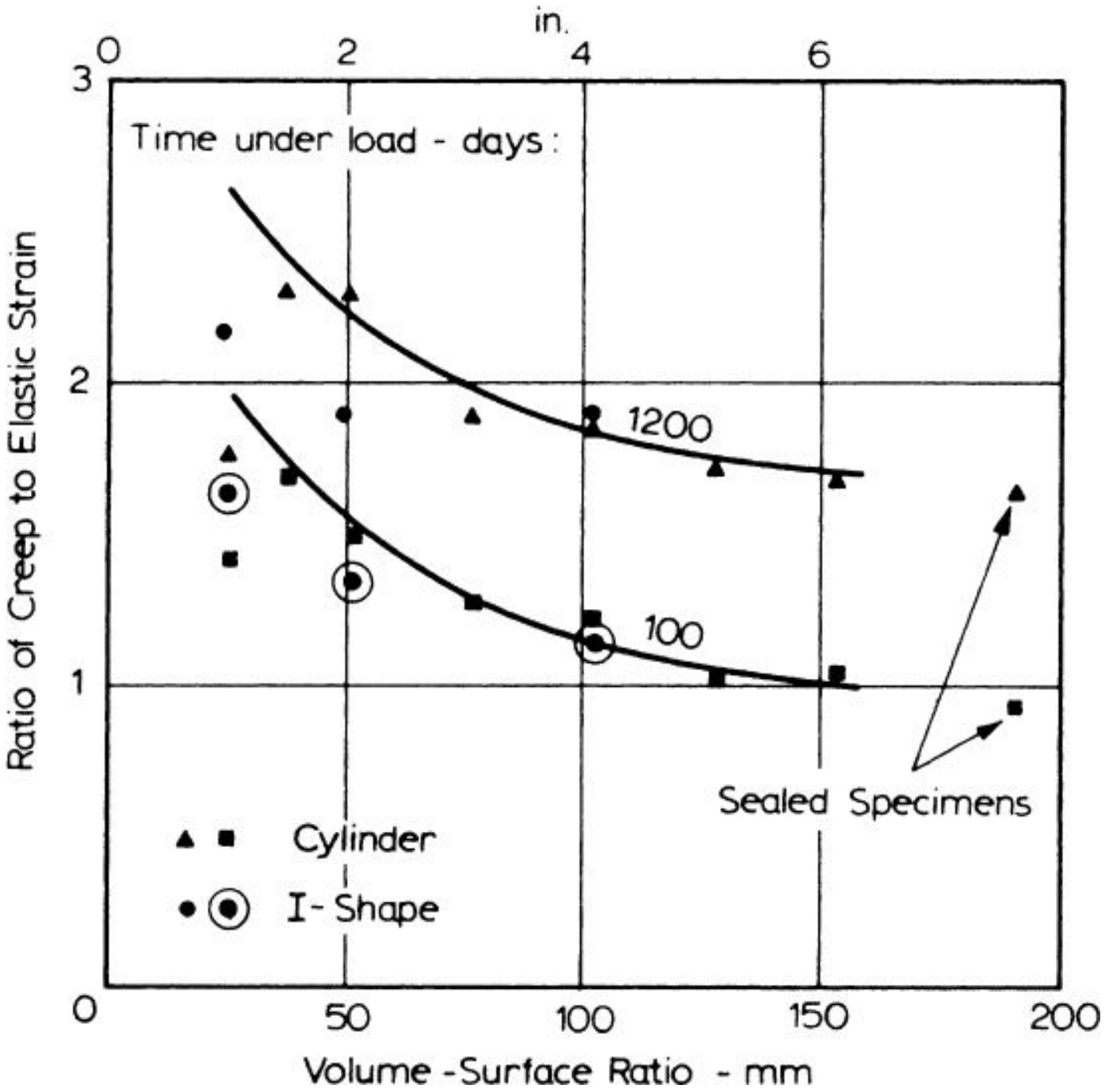


Fig. 9.39. Relation between ratio of creep to elastic strain and volume/surface ratio[9.53]

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Other influences

The influence of temperature on creep is of interest in prestressed concrete nuclear pressure vessels as well as in other types of structures, e.g. bridges. The rate of creep increases with temperature up to about 70 °C (160 °F) when, for a 1:7 mix with a water/cement ratio of 0.6, it is approximately 3.5 times higher than at 21 °C (70 °F). Between 70 °C (160 °F) and 96 °C (205 °F) the rate drops off to 1.7 times the rate at 21 °C (70 °F).[9.116] These differences in rate persist at least for 15 months under load. Figure 9.40 illustrates the progress of creep. This behaviour is believed to be due to desorption of water from the surface of the gel so that gradually the gel itself becomes the sole phase subject to molecular diffusion and shear flow; consequently the rate of creep decreases. It is also possible that a part of the increase in the creep of concrete loaded at elevated temperatures may be due to the lower strength of concrete at high temperatures[9.147] (see p. 361).

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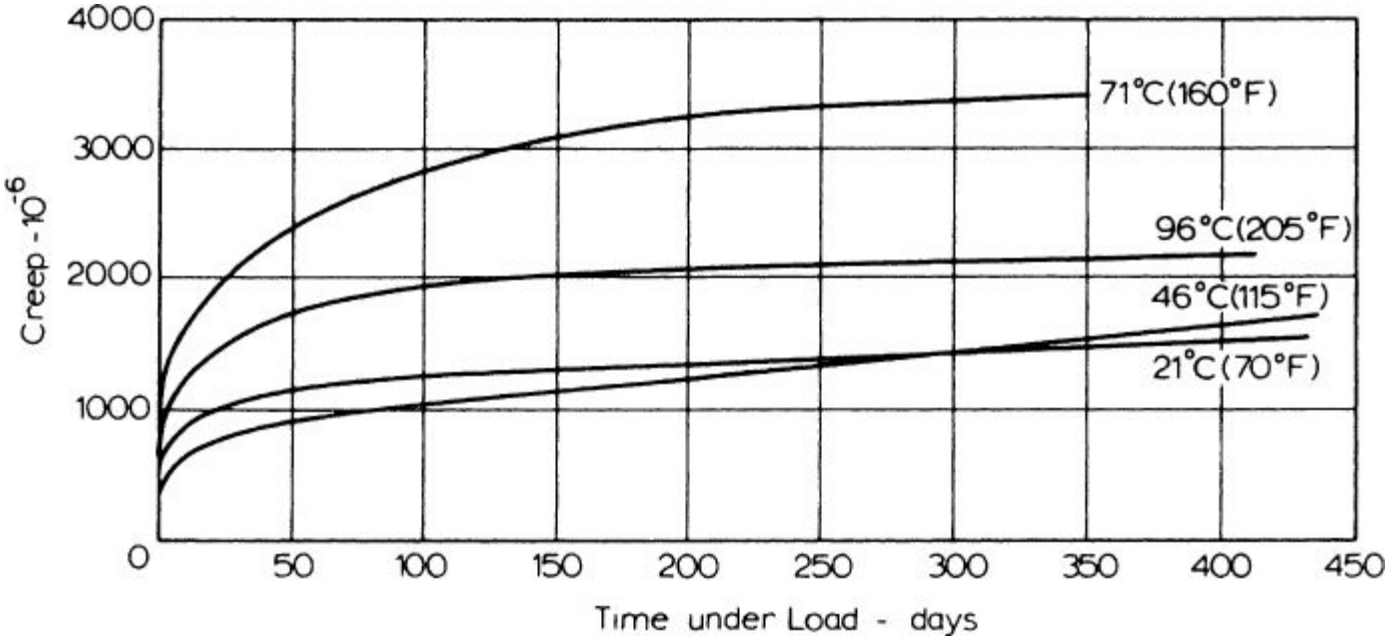


Fig. 9.40. Relation between creep and time under load for concretes stored at different temperatures (stress/strength ratio of 0.70)[9.116]

As far as low temperatures are concerned, freezing produces a higher initial rate of creep but it quickly drops to zero.[9.137] At temperatures between –10 and –30 °C (14 and –22 °F). creep is about one-half of the creep at 20 °C (68 °F).[9.155]

Creep of concrete over a wide range of temperatures is shown in Fig. 9.41.[9.136]

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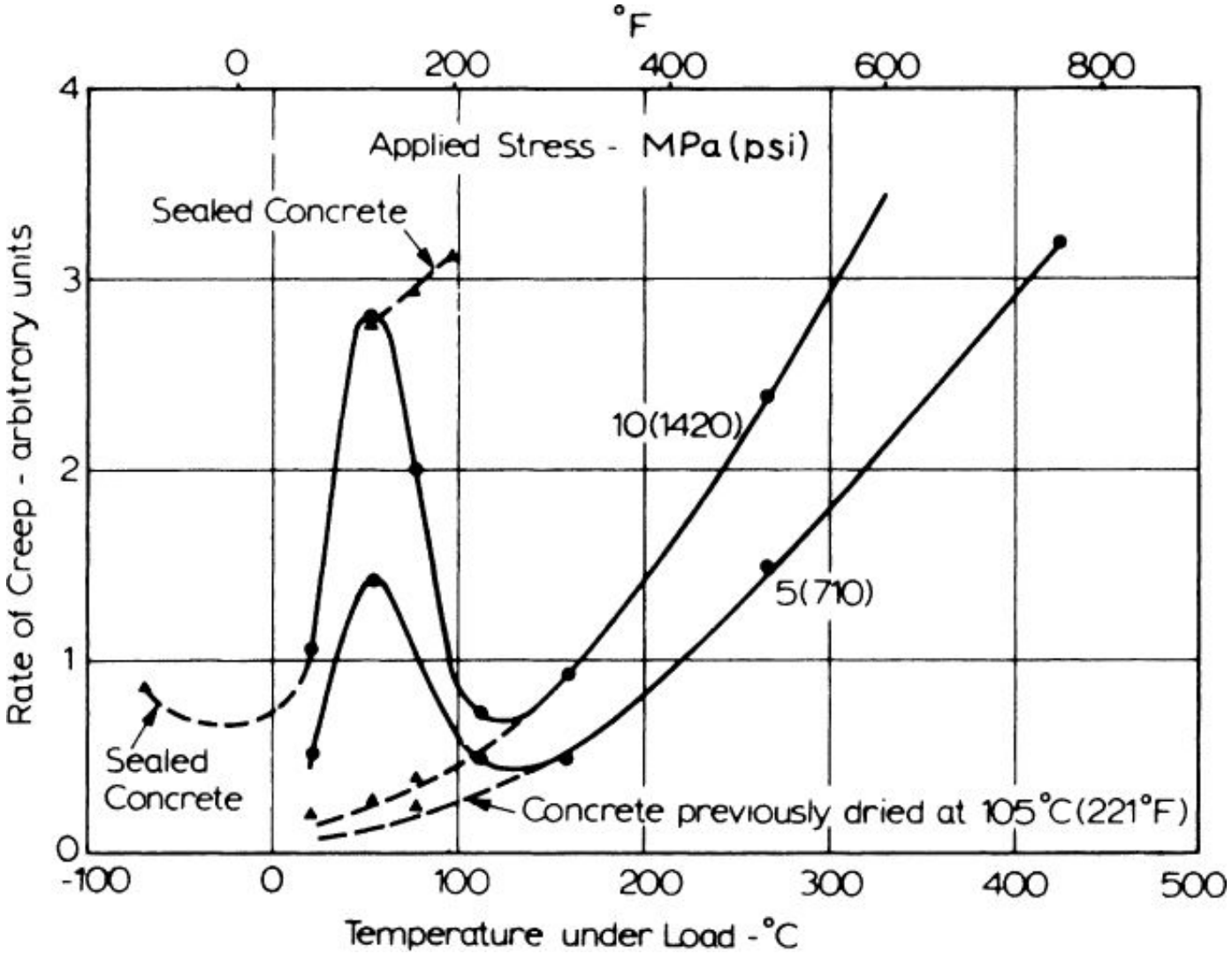


Fig. 9.41. Influence of temperature on rate of creep[9.136]

Most of the test data on creep have been obtained under a sustained constant stress but sometimes the actual load alternates between some limits. It has been found that an alternating load, with a given mean stress/strength ratio, leads to a larger time-dependent deformation than a static

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load corresponding to the same stress/strength ratio.[9.139] This is illustrated in Fig. 9.42 for the case when the alternating load varied between a stress/strength ratio 0.35 and 0.05 while the static load represented a stress/strength ratio of 0.35. The same figure shows also the deformation under a mean stress/strength ratio of 0.35 (varying between 0.45 and 0.25): the deformation is higher still. The deformation under cyclic loading is caused probably by the same mechanism as creep under a static load so that the use of the term 'creep' in both cases may be justified. It seems that cycling results in a higher rate of creep at early ages and also leads to a larger long-term value.[9.140] Thus, use of creep data from static tests may underestimate creep when the load is cyclic.

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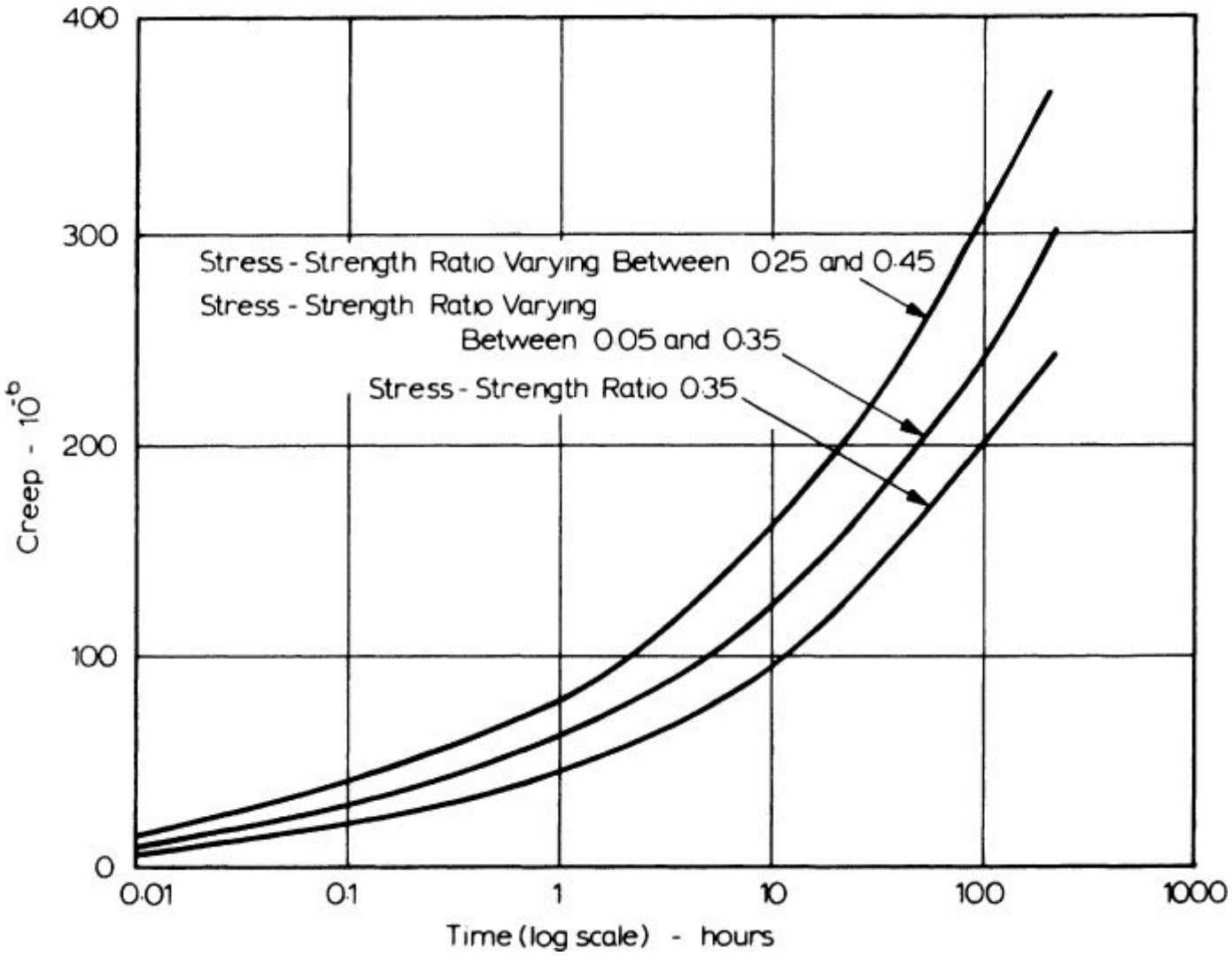


Fig. 9.42. Creep under alternating and static loading

The preceding discussion referred to uniaxial compression but creep also occurs in other loading situations, and information about creep behaviour under these conditions is especially helpful in establishing the nature of creep and in some design problems. Unfortunately, experimental data are limited, and in many cases quantitative

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evaluation and comparison with the behaviour in compression are not possible. For this reason, no more than broad qualitative statements will be made.

Creep of mass concrete in uniaxial tension is 20 to 30 per cent higher than under a compressive stress of equal magnitude. The difference depends upon age at loading and may be as high as 100 per cent for storage at a relative humidity of 50 per cent for concrete loaded at early ages. However, contradictory evidence also exists[9.101] so that reliable statements about creep in tension cannot be made. The shape of the creep–time curves in tension is broadly similar to that in compression, but the decrease in the rate of creep with time is much less pronounced in the former case because the increase in strength with age is lower. Drying enhances creep in tension just as it does in compression. In direct tension, time failure occurs in a manner similar to uniaxial compression, but the critical stress/strength ratio is probably only 0.7.[9.158]

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Creep occurs under torsional loading, and is affected by stress, water/cement ratio and ambient relative humidity in qualitatively the same manner as creep in compression. The creep–time curve is also of the same shape.[9.119] The ratio of creep to elastic deformation in torsion was found to be the same as for compressive loading.[9.138]

Under uniaxial compression, creep occurs not only in the axial direction but also in the normal directions. This is referred to as *lateral creep*. The resulting creep Poisson's ratio was considered on p. 423. From the fact that there is lateral creep induced by an axial stress, it follows that, under multiaxial stress, in any direction there is creep due to the stress applied in that direction and also creep due to the Poisson's ratio effect of creep strains in the two normal directions. There is evidence[9.45] that the superposition of creep strains due to each stress separately is not valid, so that creep under multiaxial stress cannot be simply predicted from uniaxial creep measurements. Specifically, creep under multiaxial com-

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pression is less than under a uniaxial compression of the same magnitude in the given direction (Fig. 9.43). But even under hydrostatic compression there is considerable creep.

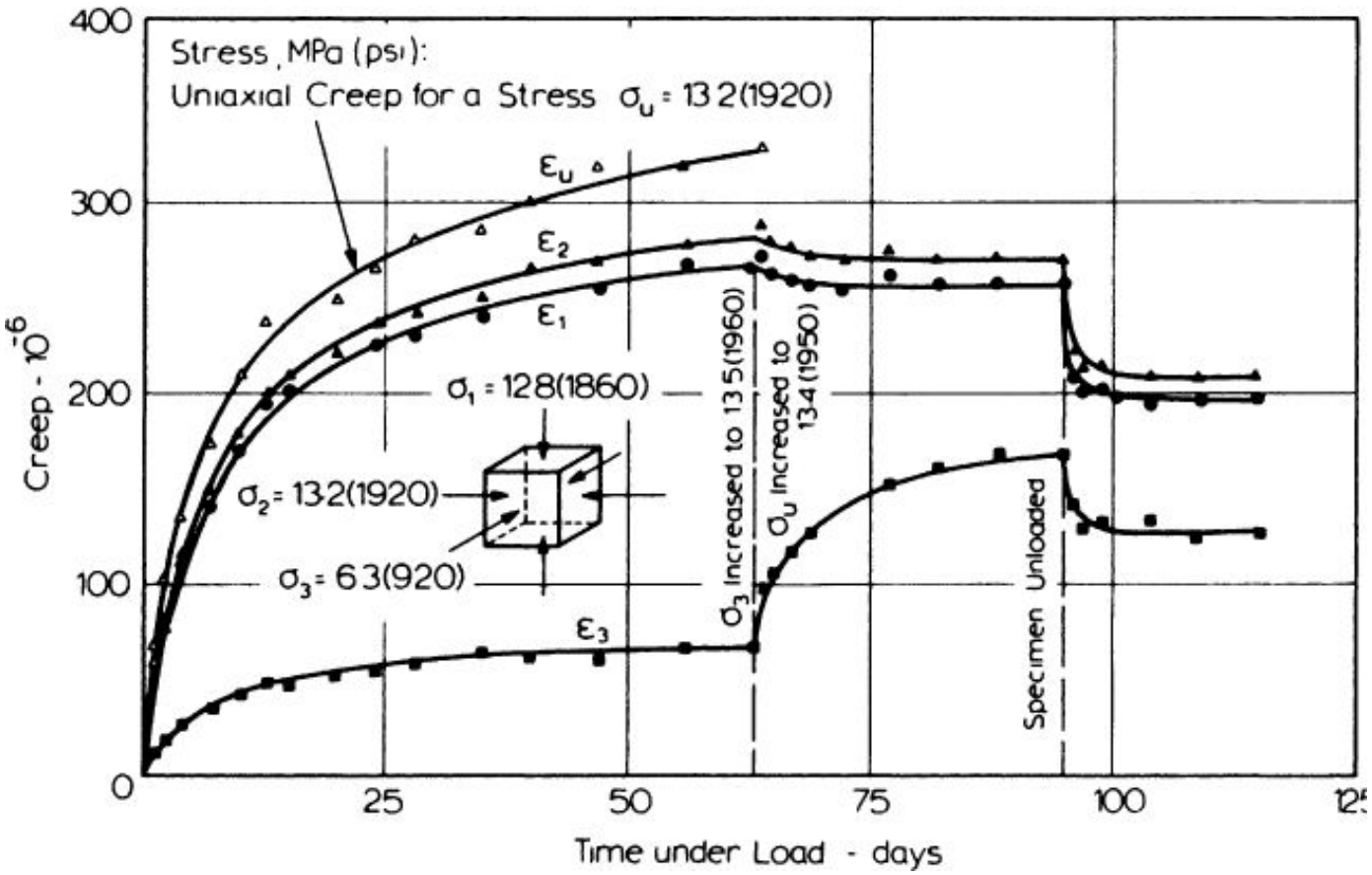


Fig. 9.43. Typical creep–time curves for concrete under triaxial compression

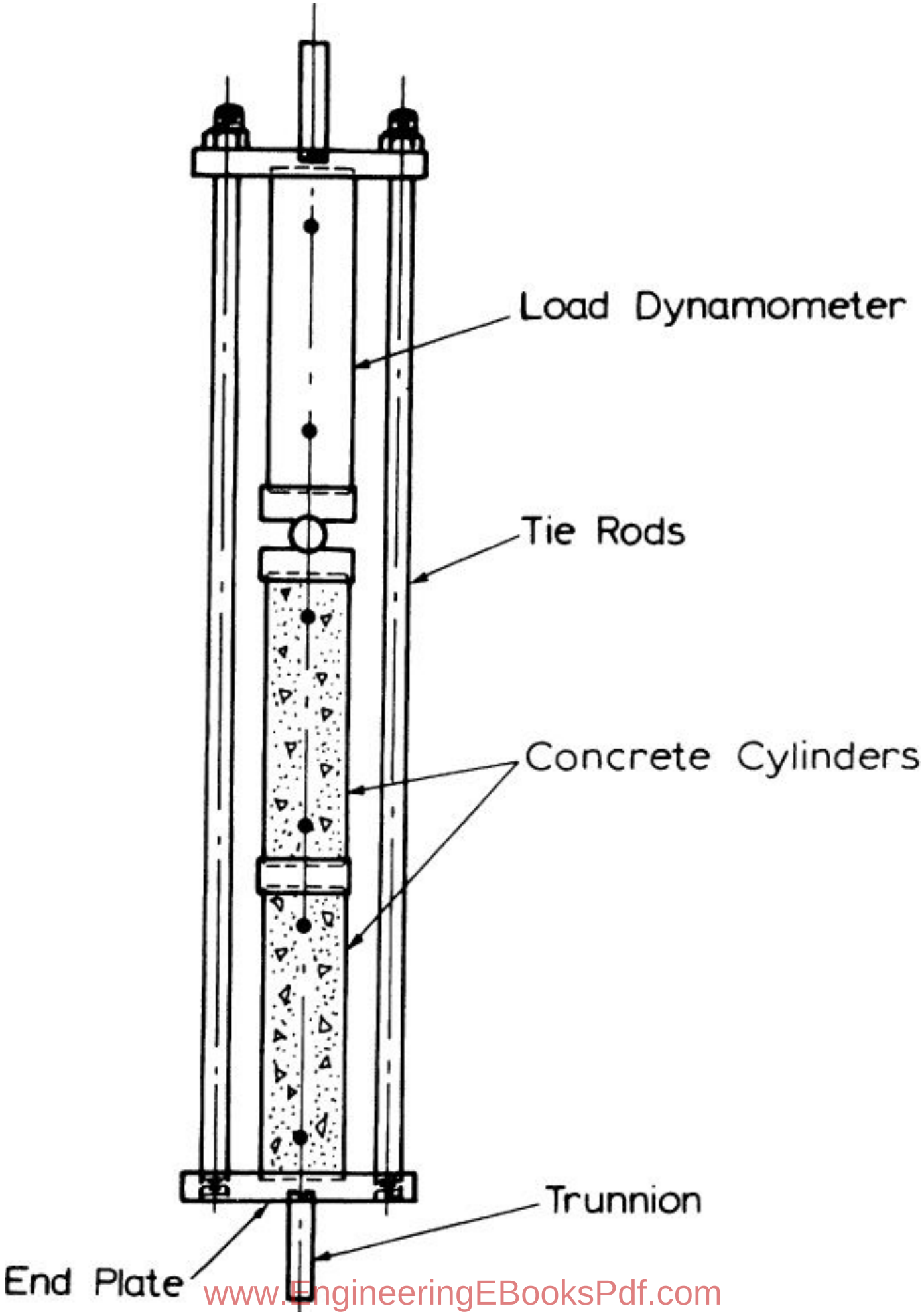
Relation between creep and time

Creep is usually determined by measuring the change with time in the strain of a specimen subjected to a constant stress and stored under ap-

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propriate conditions. ASTM C 512-02 describes a spring-loaded frame which maintains a constant load on a concrete test cylinder despite any change in its length. However, for comparative tests on concrete with untried aggregates or admixtures, an even simpler test apparatus can be used[9.141] (Fig. 9.44). Here, the load has to be adjusted from time to time, its value being determined by a dynamometer in series with the concrete specimens.

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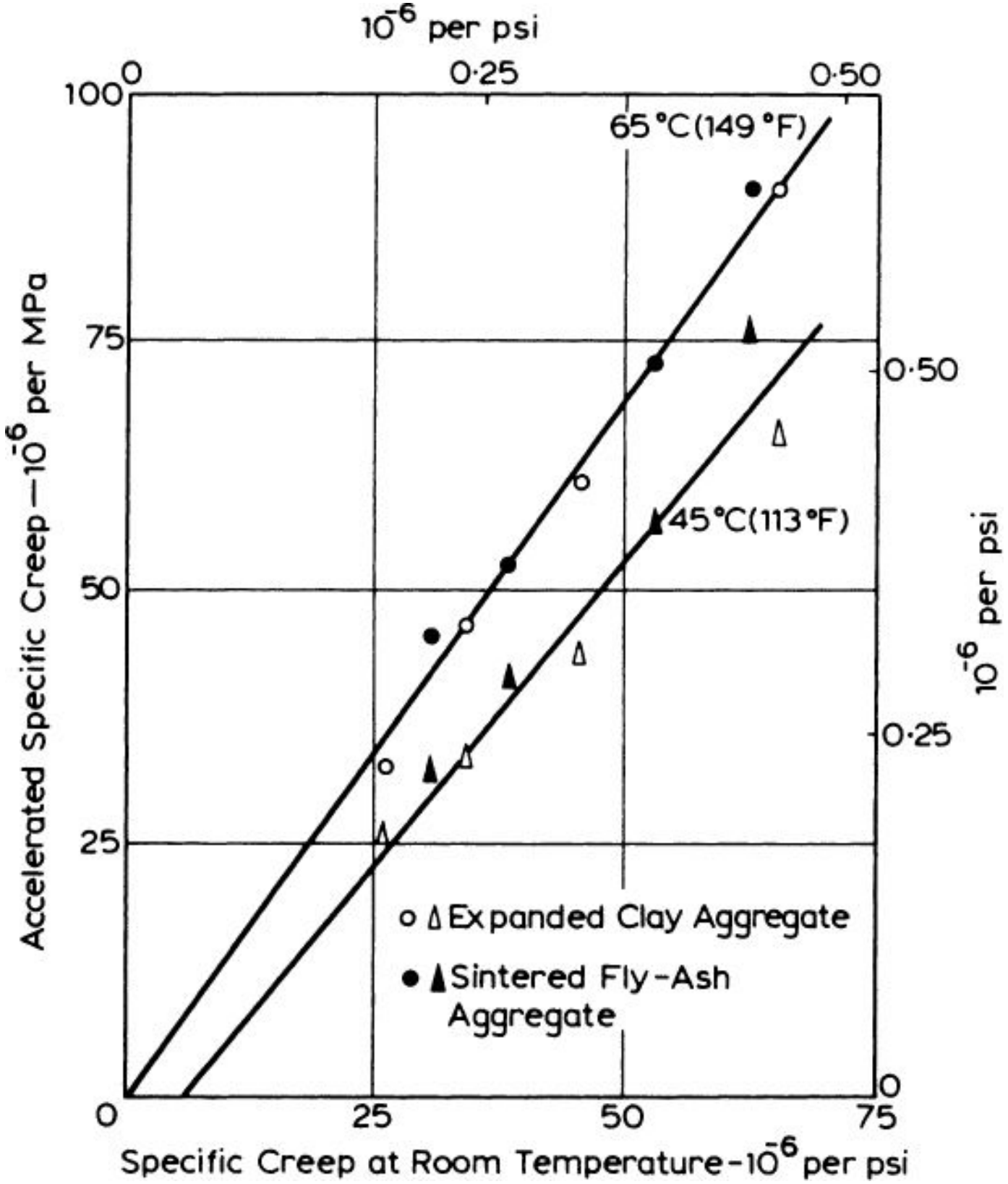


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Fig. 9.44. A simple test rig for the determination of creep of concrete under an approximately constant stress[9.141]

The apparatus of Fig. 9.44 can be used for accelerated creep tests by immersion in water at a temperature of between 45 and 65 °C. As mentioned earlier, a higher temperature leads to a higher creep so that, after 7 days, any difference between an unknown concrete and a reference concrete can be easily detected. This accelerated creep appears to be linearly related to the 100-day creep at normal temperature for a wide range of mixes and aggregates,[9.141] as shown in Fig. 9.45.

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Fig. 9.45. Relation between creep in the accelerated 7-day test at a higher temperature and 100-day creep at normal temperature for various concrete mixes[9.141]

Creep continues for a very long time, if not indefinitely, the longest determination to date indicating that a small increase in creep takes place after as long as 30 years[9.24] (Fig. 9.46); the tests were then discontinued because of interference by carbonation of the specimens. The rate of creep decreases, however, at a continuous rate, and it is generally assumed that creep tends to a limiting value after an infinite time under load; this has not, however, been proved.

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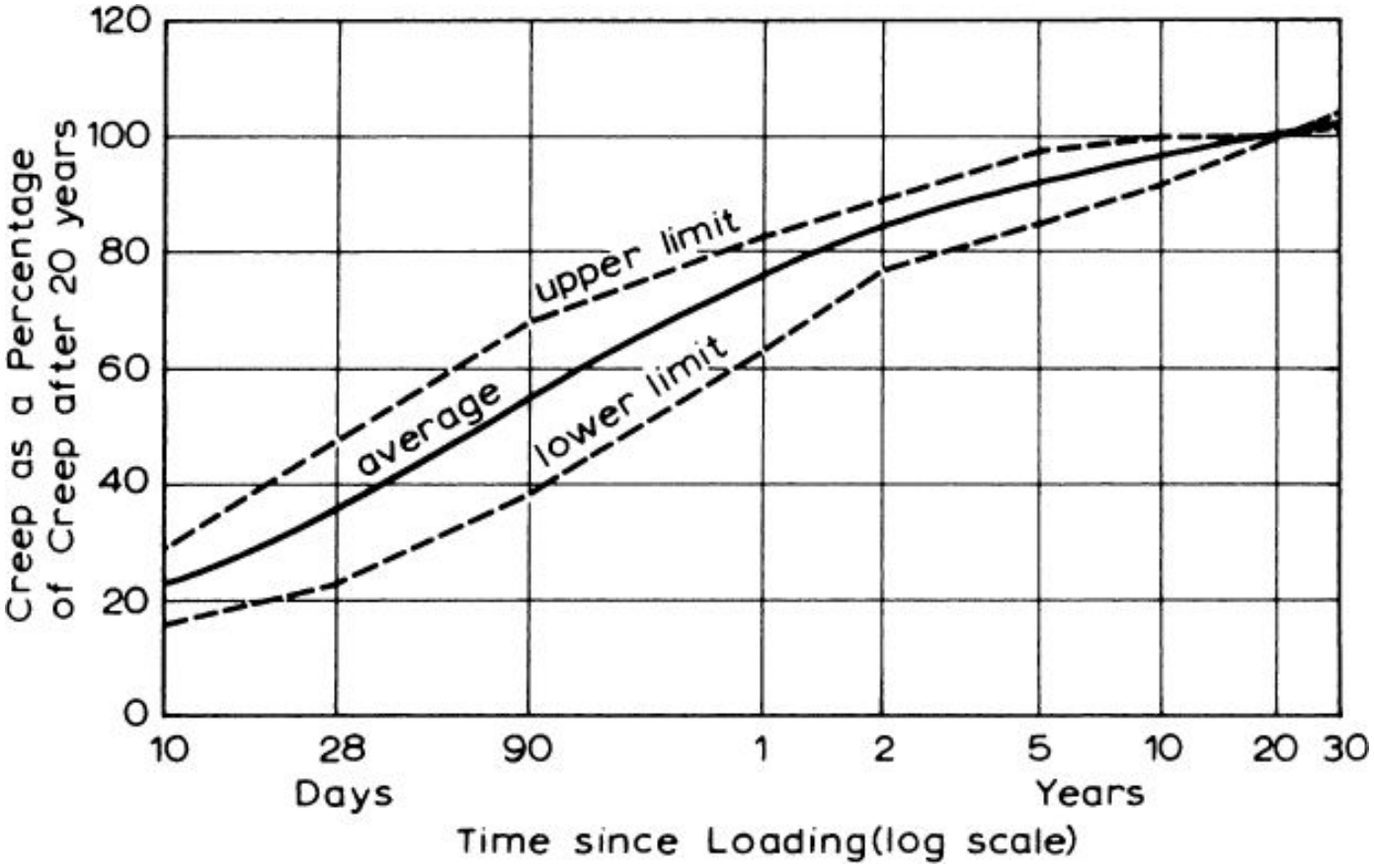


Fig. 9.46. Range of creep–time curves for different concretes stored at various relative humidities[9.24]

Figure 9.46 gives long-term measurements of Troxell *et al.*,[9.24] and it can be seen that, if creep after 1 year under load is taken as unity, then the average values of creep at later ages are:

1.14 after 2 years

1.20 after 5 years

1.26 after 10 years

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1.33 after 20 years, and

1.36 after 30 years.

These values show that ultimate creep may be in excess of 1.36 times the one-year creep, although for calculation purposes it is often assumed that the 30-year creep represents the ultimate creep.

Numerous mathematical expressions relating creep and time have been suggested. One of the most convenient is the hyperbolic expression, introduced by Ross[9.122] and by Lorman.[9.31] Ross expresses creep c after time t under load as

$$c = \frac{t}{a + bt}.$$

When $t = \infty$, then $c = 1/b$, i.e. $1/b$ is the limiting value of creep. The symbols a and b represent constants determined from experimental results: by plotting t/c against t, a straight line of slope b is obtained, and the intercept of the t/c axis is equal to a. The straight line should be drawn so as to pass through the points at later ages, there

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being generally some deviation from the straight line during the early period after the application of the load.

ACI 209R-92[9.80] uses a modified Ross expression, the main difference being the application of a power exponent of 0.6 to time t. ACI 209R-92 also offers values of coefficients to allow for various factors influencing creep.

The U.S. Bureau of Reclamation, which has made an extensive study of creep of concrete in dams, where only basic creep occurs, has found that creep can be represented by an expression of the type

$$c = F(K) \log_e (t + 1)$$

where K = age at which the load is applied,

$F(K)$ = a function representing the rate of creep deformation with time, and

t = time under load, in days.

$F(K)$ is obtained from a plot on semi-logarithmic paper.

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Sometimes, values of creep per unit stress are given, usually in units of 10^{-6} per MPa; this is known as *specific creep* or *unit creep*. Creep can also be expressed as a ratio of creep to the initial elastic deformation; this ratio is known as *creep coefficient* or *characteristic creep*. The merit of this approach is that it takes into account the elastic properties of aggregate, which influence creep and the elastic deformation of concrete in a similar manner.

All embracing, but complex, expressions for creep have been developed by Bažant and co-workers, who published also a somewhat simplified, but not simple, version of creep prediction expressions.[9.146]

The variety of creep expressions may seem bewildering but a reliable prediction of creep of any concrete under any conditions is not possible. Short-term tests, say of a 28-day duration under load, are necessary. Extrapolation is then possible. It has been found[9.142] that for periods under load up to five years, the power expression seems

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best to fit experimental data for basic creep and, for basic-plus-drying creep, a logarithmic-power function appears most appropriate. For the majority of concretes, regardless of the water/cement ratio or the type of aggregate, specific creep at the age of t days ($t > 28$), c_t, can be related to specific creep after 28 days under load, c_{28}, by the expressions:

basic creep: $c_t = c_{28} \times 0.50t^{0.21}$

total creep: $c_t = c_{28} \times (-6.19 + 2.15 \log_e t)^{0.38}$

where c_t = long-term specific creep in 10^{-6} per MPa.

Nature of creep

From Fig. 9.30, it is apparent that creep and creep recovery are related phenomena, but their nature is far from clear. The fact that creep is partly reversible suggests that it may consist of a partly reversible visco-elastic movement (consisting of a purely viscous phase and a purely elastic phase)

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and possibly also a non-reversible plastic deformation.

An *elastic* deformation is always recoverable on unloading. A *plastic* deformation is never recoverable, can be time-dependent, and there is no proportionality between plastic strain and the applied stress, or between stress and rate of strain. A *viscous* deformation is never recoverable on unloading, is always time-dependent, and there is always proportionality between the rate of viscous strain and the applied stress, and hence between stress and strain at a given time.[9.129] These various types of deformation can be summarized as shown in Table 9.6.

Table 9.6. Types of Deformation

Type of deformation	*Instantaneous*	*Time-dependent*
Reversible	Elastic	Delayed-elastic
Irreversible	Plastic set	Viscous

A possible treatment of the observed partial recovery of creep is by the principle of superpos-

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ition of strains, developed by McHenry.[9.126] This states that the strains produced in concrete at any time t by a stress increment applied at any time t_0 are independent of the effects of any stress applied either earlier or later than t_0. The stress increment is understood to mean either a compressive or a tensile stress, i.e. also a relief of load. It follows then that, if the compressive stress on a specimen is removed at age t_1, the resulting creep recovery will be the same as the creep of a similar specimen subjected to the same compressive stress at the age t_1. Figure 9.47 illustrates this statement, and it can be seen that the creep recovery is represented by the difference between the actual strain at any time and the strain that would exist at the same time, had the specimen continued to be subjected to the original compressive stress.

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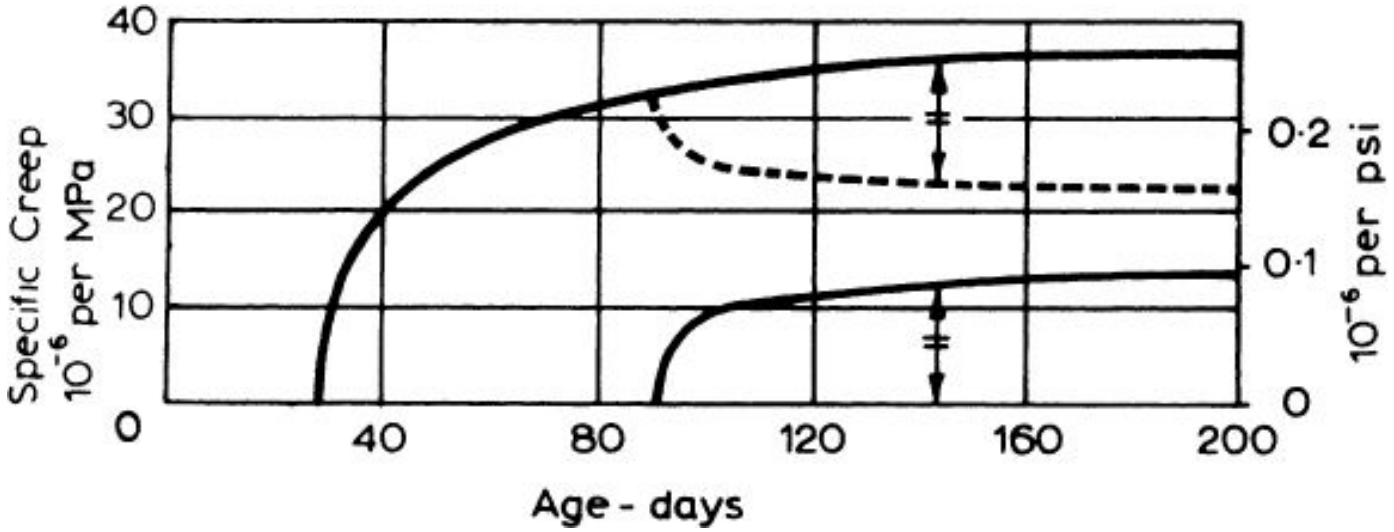


Fig. 9.47. Example of McHenry's principle of superposition of strains[9.126]

Figure 9.48 shows a comparison of actual and computed strains (the computed values being in reality the difference between two experimental curves) for sealed concrete, i.e. subject to basic creep only.[9.127] It appears that, in all cases, the actual strain after the removal of load is higher than the residual strain predicted by the principle of superposition. Thus, actual creep is less than expected. Similar error is found when the principle is applied to specimens under a variable stress.[9.107] It seems then that the principle of su-

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perposition does not fully satisfy the phenomena of creep and creep recovery.

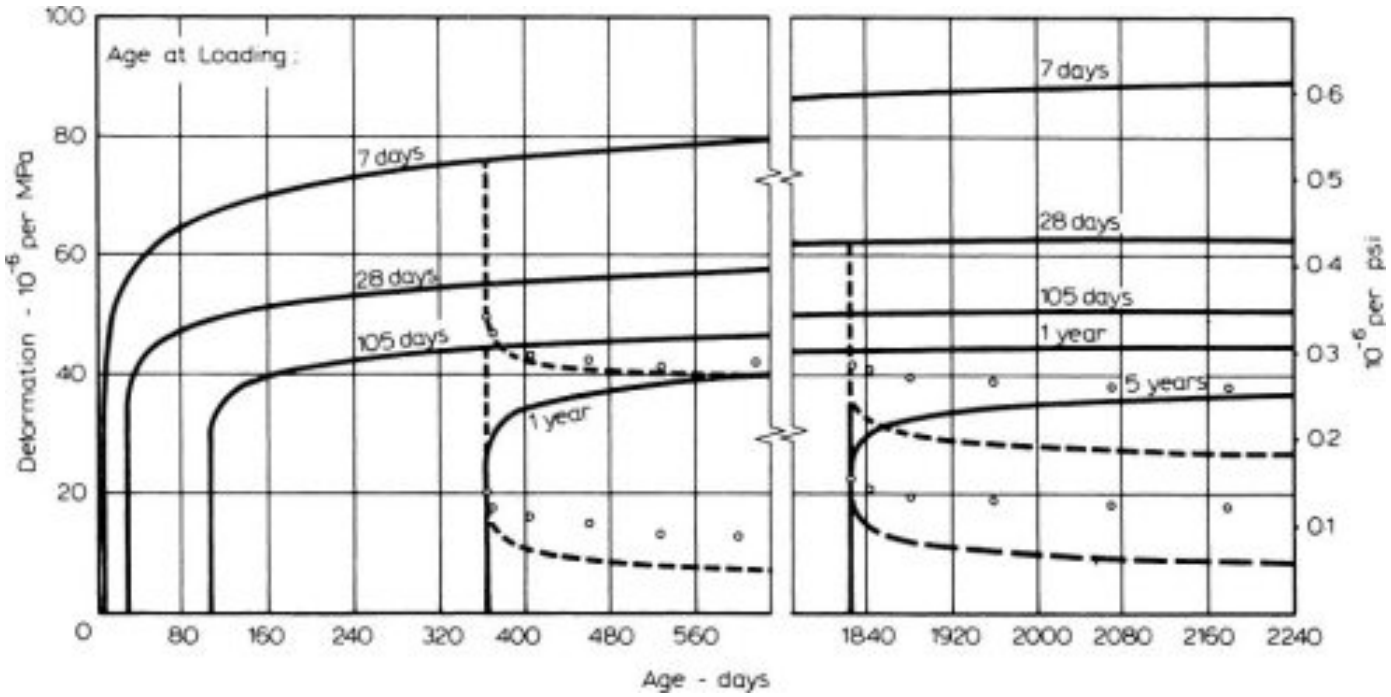


Fig. 9.48. Comparison of measured and computed strains on the basis of McHenry's principle of superposition[9.127]

The principle of superposition of strains is, nevertheless, a convenient working assumption. It implies that creep is a delayed elastic phenomenon in which full recovery is generally impeded by the progressive hydration of the cement. Because the properties of old concrete change only very little with age, creep of concrete subjected to sustained loading at the age of several

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years would be expected to be fully reversible, but this has not been verified experimentally. It should be noted that the principle of superposition leads to a tolerable error under mass-curing conditions, i.e. when only basic creep occurs. When drying creep is present, the error is large in that the creep recovery is grossly overestimated.

The problem of the nature of creep is still controversial[9.128] and cannot be discussed here in full. The locus of creep is the hydrated cement paste, and creep is related to internal movement of adsorbed or intracrystalline water, i.e. to internal seepage. Glucklich's tests[9.132] have shown that concrete from which all evaporable water has been removed exhibits practically no creep. However, the changes in the creep behaviour of concrete at high temperatures suggest that, at that stage, the water ceases to play a role and the gel itself becomes subjected to creep deformation.

Because creep can take place in mass concrete, it follows that seepage of water to the outside of concrete is not essential to the progress

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of basic creep, although such a process may well take place in drying creep. However, internal seepage of water from the adsorbed layers to voids such as capillary voids is possible. An indirect evidence of the role of such voids is given by the relation between creep and the strength of the hydrated cement paste: it would appear that creep is a function of the relative amount of the unfilled space, and it can be speculated that it is the voids in the gel that govern both strength and creep; in the latter case, the voids may be related to seepage. The volume of voids is, of course, a function of the water/cement ratio and is affected by the degree of hydration.

We should remember that capillary voids do not remain full even against full hydrostatic pressure of a water bath. Thus, internal seepage is possible under any storage conditions. The fact that creep of non-shrinking specimens is independent of the ambient relative humidity would indicate that the fundamental cause of creep ‘in air’ and ‘in water’ is the same.

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The creep–time curve shows a definite decrease in its slope, and the question arises whether this signifies a change, possibly a gradual one, in the mechanism of creep. It is conceivable that the slope decreases with the same mechanism continuing throughout, but it is reasonable to imagine that, after many years under load, the thickness of the adsorbed water layers could be reduced so far that no further reduction can take place under the same stress, and yet creep after as many as 30 years has been recorded. It is, therefore, probable that the slow, long-term part of creep is due to causes other than seepage but the deformation can develop only in the presence of some evaporable water. This would suggest viscous flow or sliding between the gel particles. Such mechanisms are compatible with the influence of temperature on creep, and can explain also the largely irreversible character of long-term creep.

Observations on creep under cyclic loading, and especially on the temperature rise within con-

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crete under such loading, have led to a modified hypothesis of creep. As already mentioned, creep under a cyclic stress is increased compared with creep under a static stress equal to the mean cyclic stress.[9.140] This increased creep is largely irrecoverable and consists of accelerated creep due to increased viscous sliding of the gel particles, and of increased creep due to a limited amount of microcracking at very early stages of the creep process. Other experimental data on creep in tension and in compression[9.143] suggest that the behaviour is best explained by a combination of the seepage and viscous shear theories of creep.

Generally, the role of microcracking is small and, excepting cyclic creep, is probably limited to concrete loaded at very early ages, and loaded to high stress/strength ratios in excess of 0.6.

Having said all this, we have to admit that the exact mechanism of creep remains uncertain.

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Effects of creep

Creep affects strains and deflections and often also stress distribution, but the effects vary with the type of structure.[9.130]

Creep of plain concrete does not *per se* affect the strength, although under *very* high stresses creep hastens the approach of the limiting strain at which failure takes place; this applies only when the sustained load is above 85 or 90 per cent of the rapidly applied static ultimate load.[9.115] Under a low sustained stress, the volume of concrete decreases (as the creep Poisson's ratio is less than 0.5) and this would be expected to increase the strength of the concrete. However, this effect is probably small.

The influence of creep on the behaviour and strength of reinforced and prestressed concrete structures is fully discussed in ref. 9.84. Here, it may be useful to mention that, in reinforced concrete columns, creep results in a gradual transfer of load from the concrete to the reinforce-

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ment. Once the steel yields, any increase in load is taken by the concrete, so that the full strength of both the steel and the concrete is developed before failure takes place – a fact recognized by the design formulae. However, in eccentrically loaded columns, creep increases the deflection and can lead to buckling. In statically indeterminate structures, creep may relieve stress concentrations induced by shrinkage, temperature changes, or movement of supports. In all concrete structures, creep reduces internal stresses due to non-uniform shrinkage, so that there is a reduction in cracking. In calculating creep effects in structures, it is important to realize that the actual time-dependent deformation is not the 'free' creep of concrete but a value modified by the quantity and position of reinforcement.

On the other hand, in mass concrete, creep in itself may be a cause of cracking when a restrained concrete mass undergoes a cycle of temperature change due to the development of the heat of hydration and subsequent cooling. A com-

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pressive stress is induced by the rapid rise in temperature in the interior of the concrete mass. This stress is low because the modulus of elasticity of very young concrete is low. The strength of very young concrete is also low so that its creep is high; this relieves the compressive stress, and the remaining compression disappears as soon as some cooling has taken place. On further cooling of concrete, tensile stresses develop and, because the rate of creep is reduced with age, cracking may occur even before the temperature has dropped to the initial (placing) value (see Fig. 9.49). For this reason, the rise in temperature in the interior of a large concrete mass must be controlled (see p. 395).

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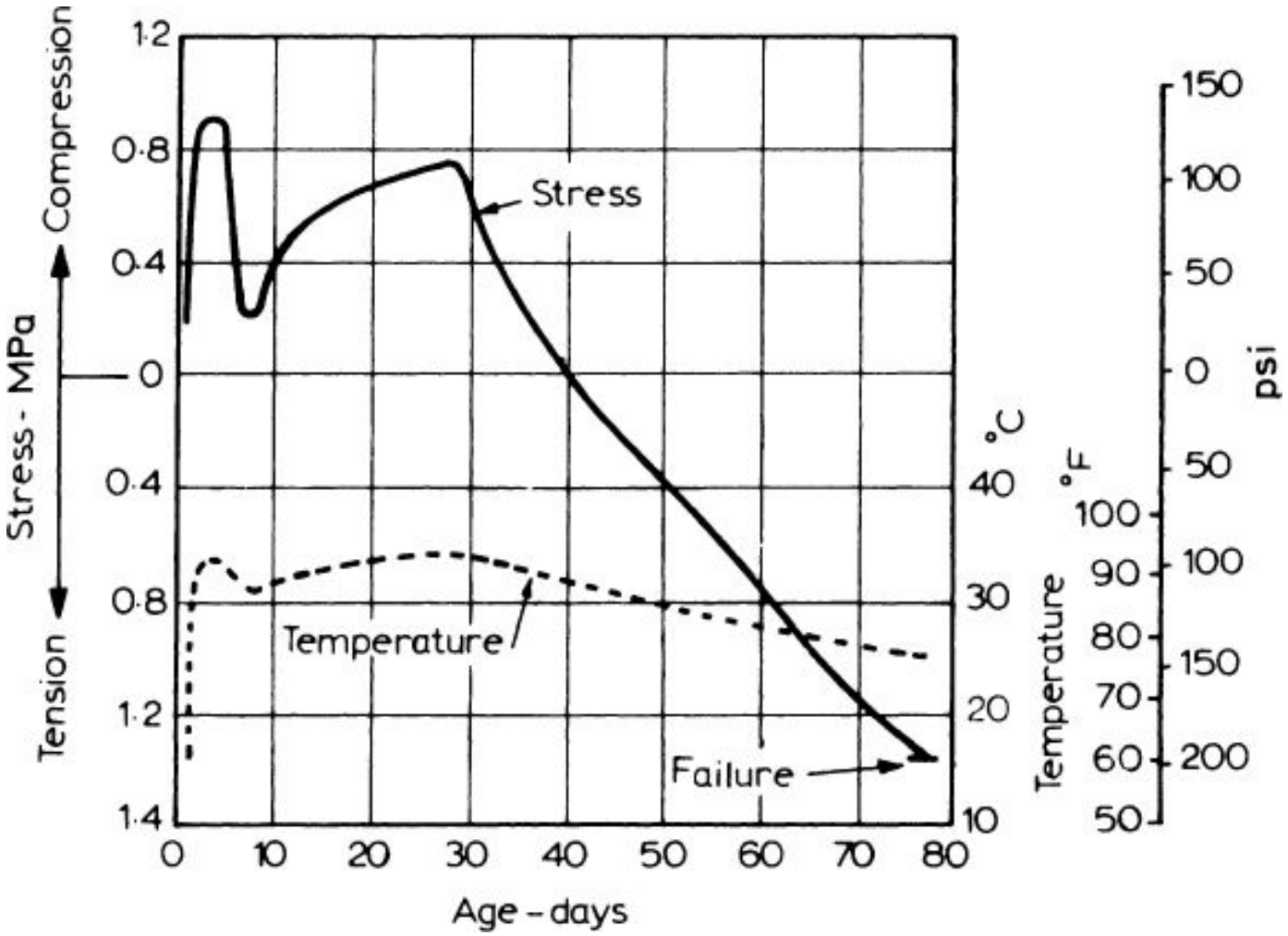


Fig. 9.49. Stress in concrete subjected to a temperature cycle at a constant length[9.131]

Creep can also lead to an excessive deflection of structural members and cause other serviceability problems, especially in high-rise buildings and long bridges.

The loss of prestress due to creep is well known and, indeed, accounts for the failure of the original attempts at prestressing.

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The effects of creep may thus be harmful but, on the whole, creep, unlike shrinkage, is beneficial in relieving stress concentrations and has contributed very considerably to the success of concrete as a structural material. Rational design methods allowing for creep in various types of structures have been developed.[9.112]

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Chapter 10. Durability of concrete

It is essential that every concrete structure should continue to perform its intended functions, that is maintain its required strength and serviceability, during the specified or traditionally expected service life. It follows that concrete must be able to withstand the processes of deterioration to which it can be expected to be exposed. Such concrete is said to be durable.

It is worth adding that durability does not mean an indefinite life, nor does it mean withstanding *any* action on concrete. Moreover, it is nowadays realized, although it was not so in the past, that, in many situations, routine maintenance of concrete is required;[10.68] an example of maintenance procedures is given by Carter.[10.72]

The fact that durability has not been considered in this book up to now could be inter-

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preted to mean that this topic is of lesser importance than other properties of concrete, notably strength. This is not so, and, indeed, in many situations, durability is of paramount importance. Nevertheless, until recently, developments in cement and concrete technology have concentrated on achieving higher and higher strengths (see p. 334). There was an assumption that 'strong concrete is durable concrete', the only special considerations being the effects of alternating freezing and thawing and some forms of chemical attack. It is now known that, for many conditions of exposure of concrete structures, both strength and durability have to be considered explicitly at the design stage. The emphasis is on the word 'both' because it would be a mistake to replace overemphasis on strength by overemphasis on durability. Durability requirements for a service life of 50 and 100 years are given in BS 8500-1 : 2006.

This chapter considers various aspects of durability. Two special topics, the effects of freezing and thawing, including the action of de-icing

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agents, and chloride attack, are the subject matter of Chapter 11.

Causes of inadequate durability

Inadequate durability manifests itself by deterioration which can be due either to external factors or to internal causes within the concrete itself. The various actions can be physical, chemical, or mechanical. Mechanical damage is caused by impact (considered on p. 345), abrasion, erosion or cavitation; the last three are discussed towards the end of the present chapter. The chemical causes of deterioration include the alkali-silica and alkali-carbonate reactions which are also discussed in this chapter. External chemical attack occurs mainly through the action of aggressive ions, such as chlorides, sulfates, or of carbon dioxide, as well as many natural or industrial liquids and gases. The damaging action can be of various kinds and can be direct or indirect.

Physical causes of deterioration include the effects of high temperature or of the differences

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in thermal expansion of aggregate and of the hardened cement paste (discussed in Chapter 8). An important cause of damage is alternating freezing and thawing of concrete and the associated action of de-icing salts; these topics are discussed in Chapter 11.

It should be observed that the physical and chemical processes of deterioration can act in a synergistic manner. The various factors affecting the durability of concrete are the subject matter of the present chapter. At this stage, it is worth noting that deterioration of concrete is rarely due to one isolated cause: concrete can often be satisfactory despite some undesirable features but, with an additional adverse factor, damage will occur. For this reason, it is sometimes difficult to assign deterioration to a particular factor, but the quality of concrete, in the broad sense of the word, though with a special reference to permeability, nearly always enters the picture. Indeed, with the exception of mechanical damage, all the adverse influences on durability in-

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volve the transport of fluids through the concrete. For this reason, consideration of durability requires an understanding of the phenomena involved.

Transport of fluids in concrete

There are three fluids principally relevant to durability which can enter concrete: water, pure or carrying aggressive ions, carbon dioxide and oxygen. They can move through the concrete in different ways, but all transport depends primarily on the structure of the hydrated cement paste. As stated earlier, durability of concrete largely depends on the ease with which fluids, both liquids and gases, can enter into, and move through, the concrete; this is commonly referred to as permeability of concrete. Strictly speaking, permeability refers to flow through a porous medium. Now, the movement of the various fluids through concrete takes place not only by flow through the porous system but also by diffusion and sorption, so that our concern is really with

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penetrability of concrete. Nevertheless, the commonly accepted term 'permeability' will be used for the overall movement of fluids into and through concrete except where, for clarity, distinctions between the various types of flow need to be made.

Towards the end of 2010 a paper[10.142] was published describing and discussing the transport phenomena involved in evaluating the measurement of chlorides in concrete: a total of 11 phenomena are listed. This paper is of great value in helping to understand the transport phenomena involved. Alas, in a practical situation concrete fluids do not follow a single mode of transport, nor is a single type of ion involved. I know that wick action is involved in the old-fashioned oil lamp but, even there, there is a confusion between the UK and US in the name of the fluid involved: paraffin and kerosene.

These remarks do not detract from the value of the paper: they simply illustrate the need to *un-*

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derstand the phenomena involved which have to be applied to a given situation.

Influence of the pore system

The aspect of the structure of hardened cement paste relevant to permeability is the nature of the pore system within the bulk of the hardened cement paste and also in the zone near the interface between the cement paste and the aggregate. The interface zone occupies as much as one-third to one-half of the total volume of hardened cement paste in concrete and is known to have a different microstructure from the bulk of the hardened cement paste. The interface is also the locus of early microcracking. For these reasons, the interface zone can be expected significantly to contribute to the permeability of concrete.[10.44] However, Larbi[10.49] found that, despite the higher porosity of the interface zone, the permeability of concrete is controlled by the bulk of the hardened cement paste, which is the only continuous phase in concrete.

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Support to Larbi's view is lent by the fact that the permeability of hardened cement paste is not lower than that of concrete made with a similar cement paste. However, of relevance in concrete is also the fact that any movement of fluids has to follow a path made longer and more tortuous by the presence of aggregate, which also reduces the effective area for flow. Thus, the significance of the interface zone with respect to permeability remains uncertain. Even more generally, it has to be admitted that the relationship between permeability and the pore structure of hardened cement paste is, at best, qualitative.[10.97]

The pores relevant to permeability are those with a diameter of at least 120 or 160 nm. These pores have to be continuous. Pores which are ineffective with respect to flow, that is to permeability, include, in addition to discontinuous pores, those which contain adsorbed water and those which have a narrow entrance, even if the pores themselves are large (cf. Fig. 6.16).

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Aggregate can also contain pores, but these are usually discontinuous. Moreover, aggregate particles are enveloped by the cement paste so that the pores in aggregate do not contribute to the permeability of concrete. The same applies to discrete air voids, such as entrained-air bubbles (see p. 547). In addition, the concrete as a whole contains voids caused by incomplete compaction or by trapped bleed water. These voids may occupy between a fraction of one per cent and 10 per cent of the volume of the concrete, the latter figure representing a highly honeycombed concrete of very low strength. Such concrete or concrete with leaking joints should not be made, and it will not be further discussed.

Flow, diffusion, and sorption

Because of the existence of pores of different kinds, some of which contribute to permeability and some of which do not do so, it is important to distinguish between porosity and permeability. *Porosity* is a measure of the proportion of the total volume of concrete occupied by pores, and

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is usually expressed in per cent. If the porosity is high and the pores are interconnected, they contribute to the transport of fluids through concrete so that its permeability is also high. On the other hand, if the pores are discontinuous or otherwise ineffective with respect to transport, then the permeability of the concrete is low, even if its porosity is high.

Porosity can be measured by mercury intrusion; this topic was referred to on p. 283, and is comprehensively treated by Cook and Hover.[10.46] Other fluids can also be used. An indication of porosity can be obtained from the measurement of absorption of concrete, which is considered on p. 489.

As far as the ease of movement of fluids through concrete is concerned, thus far loosely referred to as permeability, three mechanisms should be distinguished. *Permeability* refers to flow under a pressure differential. *Diffusion* is the process in which a fluid moves under a differential in concentration; the relevant property of

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concrete is *diffusivity*. Gases can diffuse through water-filled space or through air-filled space but, in the former case, the process is 10^4 to 10^5 times slower than in the latter.

Sorption is the result of capillary movement in the pores in concrete which are open to the ambient medium. It follows that capillary suction can take place only in partially dry concrete; there is no sorption of water in completely dry concrete or in saturated concrete.

Because the penetrability of concrete is described in the literature in varying terms, it is important to present briefly the relevant mathematical expressions and to state clearly the units of measurement. A comprehensive discussion of various aspects of permeability is presented in ref. 10.96.

Coefficient of permeability

Flow in capillary pores in saturated concrete follows Darcy's law for laminar flow through a porous medium:

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$$\frac{\mathrm{d}q}{\mathrm{d}t}\frac{1}{A} = \frac{K'\rho g}{\eta}\frac{\Delta h}{L}$$

where $\mathrm{d}q/\mathrm{d}t$ = rate of flow of water in m^3/s,

A = cross-sectional area of the sample in m^2,

Δh = drop in hydraulic head through the sample, measured in m,

L = thickness of the sample in m,

η = dynamic viscosity of the fluid in N s/m^2,

ρ = density of the fluid in kg/m^3, and

g = acceleration due to gravity.

The coefficient K' is then expressed in metres to the power 2, and represents the *intrinsic permeability* of the material, independently of the fluid involved.

As the fluid involved is generally water, we can put:

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$$K = \frac{K'\rho g}{\eta}.$$

The coefficient K is then expressed in metres per second and is referred to as the *coefficient of permeability* of concrete, it being understood that it refers to water at room temperature. The last qualification arises from the fact that the viscosity of water changes with temperature. The flow equation can thus be written as:

$$\frac{\mathrm{d}q}{\mathrm{d}t}\frac{1}{A} = K\frac{\Delta h}{L}$$

and when a steady state of flow $\mathrm{d}q/\mathrm{d}t$ has been reached, K is determined direct.

Diffusion

As stated earlier, when the transport of a gas or a vapour through concrete is the result of a concentration gradient, and not of a pressure differential, diffusion takes place.

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As far as the diffusion of gases is concerned, carbon dioxide and oxygen are of primary interest: the former leads to carbonation of hydrated cement paste, and the latter makes possible the progress of corrosion of embedded steel. The first of these mechanisms of deterioration is discussed later in this chapter; corrosion is considered in Chapter 11. At this stage, it is useful to note that the diffusivity coefficient of a gas is inversely proportional to the square root of its molar mass,[10.130] so that, for instance, oxygen diffuses, theoretically, 1.17 times faster than carbon dioxide. This relation makes it possible to calculate the diffusion coefficient of one gas from experimental data on another gas.

Diffusion coefficient

The diffusion equation applicable to water vapour and air can be expressed by Fick's first law as:

$$J = -D \frac{\mathrm{d}c}{\mathrm{d}L}$$

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where dc/dL = concentration gradient in kg/m^4 or $moles/m^4$,

D = diffusion coefficient in m^2/s,

J = mass transport rate in kg/m^2 s (or $moles/m^2$ s), and

L = thickness of the sample in metres.

Even though diffusion takes place only through the pores, the values of J and D refer to the cross-section of the concrete sample; thus, D is, in reality, the *effective diffusion coefficient*.

The diffusion coefficient of a gas can be determined experimentally under a steady-state system, with two sides of a concrete specimen being exposed, each to a different pure gas: the mass of the gases on the side opposite to that where they were originally present is then determined. The pressure on each side of the specimen should be the same because the driving force in diffusion is the difference in molar concentration and not a pressure differential.

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Diffusion through air and water

Papadakis *et al.*[10.130] present expressions for the effective diffusion coefficient of carbon dioxide as a function of the relative humidity of the air and of the porosity of hardened cement paste or of the compressive strength of concrete. The diffusion through water is 4 orders of magnitude slower than through air. It should be noted that the diffusion coefficient changes with age because the pore system in concrete changes with time, especially when hydration of cement continues.

Oxygen diffusion through concrete is strongly affected by moist curing,[10.96] prolonged curing reducing the diffusion coefficient by a factor of about 6. The moisture condition of the concrete under test also has a large influence because water in the pores significantly reduces the diffusion. By way of illustration, the oxygen diffusion coefficient of well-cured concrete, conditioned at a relative humidity of 55 per cent, is less than 5

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$\times 10^{-8}$ m^2/s for high-quality concrete, and more than 50×10^{-8} m^2/s for poor-quality concrete.[10.96]

Movement of water vapour through concrete can occur as a result of a humidity differential on its two opposed sides.[10.12] The relative humidity on the two sides of the concrete has to be known because an increase in relative humidity decreases the air-filled pore space available for diffusion. It follows then that, if the moist side is, for instance, saturated, an increase in the relative humidity of the dry side reduces the vapour permeability. Water vapour transmission of concrete is generally affected in a similar manner to air permeability.

In addition to the diffusion of gases, ions of aggressive character, notably chlorides and sulfates, move by diffusion in the pore water. It is in the pore water that reactions with hydrated cement paste take place so that *ionic diffusion* is of importance with respect to sulfate attack of con-

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crete and chloride attack of embedded steel. Ionic diffusion is most effective when the pores in the hardened cement paste are saturated, but it can also take place in partially saturated concrete.

Like permeability, diffusion is lower at lower water/cement ratios, but the influence of the water/cement ratio on diffusion is much smaller than on permeability.

Absorption

The volume of pore space in concrete, as distinct from the ease with which a fluid can penetrate it, is measured by absorption; the two quantities are not necessarily related. Absorption is usually measured by drying a specimen to a constant mass, immersing it in water, and measuring the increase in mass as a percentage of dry mass. Various procedures can be used, and widely different results are obtained as shown in Table 10.1. One reason for this variation in the values of absorption is that, at one extreme, drying at ordinary temperature may be ineffective in removing

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all the water; on the other hand, drying at high temperatures may remove some of the combined water. Absorption cannot, therefore, be used as a measure of quality of concrete, but most good concretes have an absorption well below 10 per cent by mass; if the volume occupied by water is to be calculated, an allowance for the difference in the specific gravity of water and of concrete needs to be made.

Table 10.1. Values of Absorption of Concrete Determined in Various Ways[10.7]

Drying condition	*Immersion condition*	*Absorption (per cent) for concrete mix*					
		A	*B*	*C*	*D*	*E*	*F*
100 °C (212 °F)	Water for 30 minutes	4.7	3.2	8.9	12.3		
100 °C (212 °F)	Water for 24 hours	7.4	6.9	9.1	12.9		
100 °C (212 °F)	Water for 48 hours	7.5	7.0	9.2	13.1		
100 °C (212 °F)	Water for 48 hours plus 5 hours boiling	8.1	7.3	14.1	18.2		
65 °C (149 °F)	5 hours boiling	6.4	6.4	13.2	17.2		
105 °C (221 °F) to constant mass	1 hour					3.0	7.4
	24 hours					3.4	7.7
	7 days					3.5	7.8
20 °C (68 °F) *in vacuo* over lime for 30 days	1 hour					1.9	5.9
	24 hours					2.2	6.3
	7 days					2.3	6.4

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An absorption test on several small portions of concrete is prescribed by ASTM C 642-06; drying at 100 to 110 °C (212 to 230 °F) and immersion in water at 21 °C (70 °F) for at least 48 hours are used. The requirements of BS 1881-122 : 1983 are similar except that the test is performed on whole core specimens.

Absorption tests are not used frequently except for routine quality control of precast products such as paving flags, slabs or kerb (curb) units. The absorption of sawn-off small test specimens, dried for 72 hours at 105 °C, and then immersed in water for 30 minutes and for 24 hours, is determined.

Surface absorption tests

For practical purposes, it is the absorption characteristics of the outer zone of concrete (which offers protection to reinforcement) that are of greatest interest. For that reason, tests measuring the surface absorption have been developed.

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A test to determine the *initial surface absorption* is prescribed in BS 1881-5 : 1984 (withdrawn). In essence, the rate of absorption of water by the surface zone of concrete is determined during a prescribed period (ranging between 10 minutes and 1 hour) under a head of 200 mm (8 in.) of water: this head is only slightly greater than that which would be caused by driving rain. The rate of initial surface absorption is expressed in millilitres per square metre per second.

Initial absorption after 10 minutes greater than 0.50 ml/m^2 per second would be considered high, and smaller than 0.25 ml/m^2 per second, low. Corresponding values after 2 hours are, respectively, greater than 0.15 ml/m^2 per second and smaller than 0.07 ml/m^2 per second.[10.96]

A shortcoming of the initial surface absorption test is that the flow of water through the concrete is not uni-directional. To remedy this, several modified tests have been proposed but none has gained general acceptance.

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The mass of water which is absorbed by concrete during the test depends on the pre-existing moisture content. For this reason, the results of the initial surface absorption test cannot be readily interpreted unless the concrete has been conditioned to a known hygrometric state prior to the tests. This requirement cannot be satisfied in in-situ concrete. In consequence, a low value of the initial surface absorption may be due either to the inherent low absorption characteristics of the concrete tested or else to the fact that the pores in poor-quality concrete are already full of water.

Bearing the above limitation in mind, the initial surface absorption test can be used to compare the effectiveness of curing of the outer zone of concrete.

A test which gives some measure of the ease with which water or air enters concrete in situ was developed by Figg.[10.22] A small hole is drilled and sealed with silicone rubber. This plug is pierced by a hypodermic needle connected to a vacuum pump, and pressure in the system is

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reduced by a given amount. The time required for air to permeate through the concrete and increase the pressure in the cavity to a specified value is an indication of the air 'permeability' of the concrete. Another model of the apparatus makes it possible to assess the water 'permeability' of concrete by measuring the time for a given volume of water to enter the concrete.[10.22] Several modifications of the Figg apparatus have been developed.[10.96]

It should be pointed out that the term 'permeability' is not really valid because the output of the Figg tests is not directly related to the coefficient of permeability as properly defined. Nevertheless, the tests are useful for comparative purposes.

Sorptivity

Because of the difficulties associated with the absorption tests, on the one hand, and, on the other, because permeability tests measure the response of concrete to pressure, which is rarely the driv-

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ing force of fluids entering concrete, there is a need for another type of test. Such a test measures the rate of absorption of water by capillary suction of unsaturated concrete placed in contact with water; no head of water exists.

Essentially, the *sorptivity test* determines the rate of capillary-rise absorption by a concrete prism which rests on small supports in a manner such that only the lowest 2 to 5 mm of the prism is submerged. The increase in the mass of the prism with time is recorded.

It has been shown[10.98] that there exists a relation of the form

$$i = St^{0.5}$$

where i = increase in mass since the beginning of the testing per unit of cross-sectional area in contact with water, divided by the density of water. Working in metric units i can be expressed in mm

t = time, measured in minutes, at which the mass is determined, and

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S = sorptivity in $\text{mm/min}^{0.5}$.

In practice, it is easier to measure the value of i as a rise in the water level in the concrete, which manifests itself by a darker colour. In such a case, i is measured direct in millimetres. If sorptivity is to be expressed in consistent SI units, then the following conversion can be used:

$$1\ \text{mm/min}^{0.5} = 1.29 \times 10^{-4}\ \text{m/s}^{0.5}.$$

In the test, several measurements are taken over a period of up to 4 hours, and a straight line is fitted to the plot of the increase in mass, or the rise of the water front, versus the square root of time. The point of origin (and possibly also the very early readings) is ignored because there is a small increase in mass at the instant when the open surface pores in the lowest 2 to 5 mm of the prism first become submerged (see Fig. 10.1).

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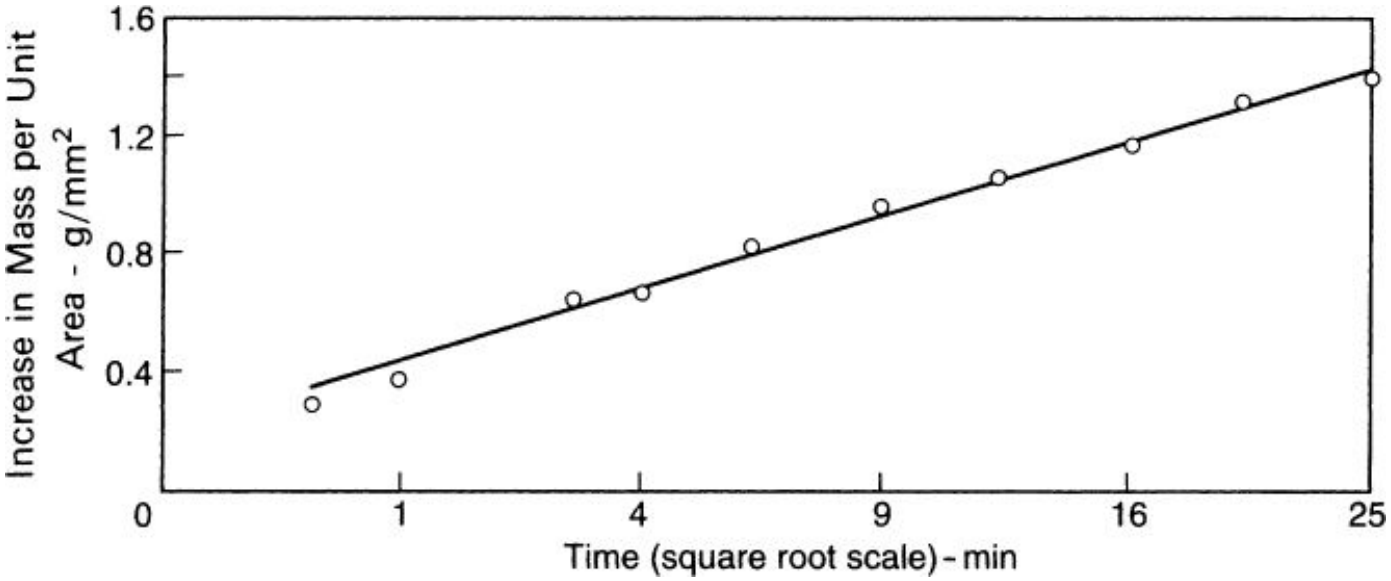


Fig. 10.1. Example of relation between increase in mass of water per unit area and time used to calculate sorptivity

Some typical values of sorptivity are: 0.09 $mm/min^{0.5}$ for concrete with a water/cement ratio of 0.4, and 0.17 $mm/min^{0.5}$ at a water/cement ratio ratio of 0.6; these should not be considered as anything more than examples.

As in the initial surface absorption test, the higher the moisture content of the concrete the lower the measured sorptivity so that, if possible, the specimen should be conditioned at 105 °C prior to testing; alternatively, the hygral state of the specimen should be established.

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Water permeability of concrete

The principles of flow of water through concrete under pressure were discussed on p. 486 in terms of flow through a porous body. Some more specific features of the permeability of concrete will now be considered.

First of all, we can note that the hardened cement paste is composed of particles connected over only a small fraction of their total surface. For this reason, a part of the water is within the field of force of the solid phase, i.e. it is adsorbed. This water has a high viscosity but is, nevertheless, mobile and takes part in the flow.[10.2] As already stated, the permeability of concrete is not a simple function of its porosity, but depends also on the size, distribution, shape, tortuosity, and continuity of the pores. Thus, although the cement gel has a porosity of 28 per cent, its permeability[10.3] is only about 7×10^{-16} m/s. This is due to the extremely fine texture of hardened cement paste: the pores and the solid particles are

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very small and numerous, whereas, in rocks, the pores, though fewer in number, are much larger and lead to a higher permeability. For the same reason, water can flow more easily through the capillary pores than through the much smaller gel pores: the cement paste as a whole is 20 to 100 times more permeable than the gel itself.[10.3] It follows that the permeability of hardened cement paste is controlled by its capillary porosity. The relation between these two quantities is shown in Fig. 10.2. For comparison, Table 10.2 lists the water/cement ratio of pastes having the same permeability as some common rocks.[10.3] It is interesting to see that the permeability of granite is about the same as that of mature cement paste with a water/cement ratio of 0.7, i.e. not of high quality.

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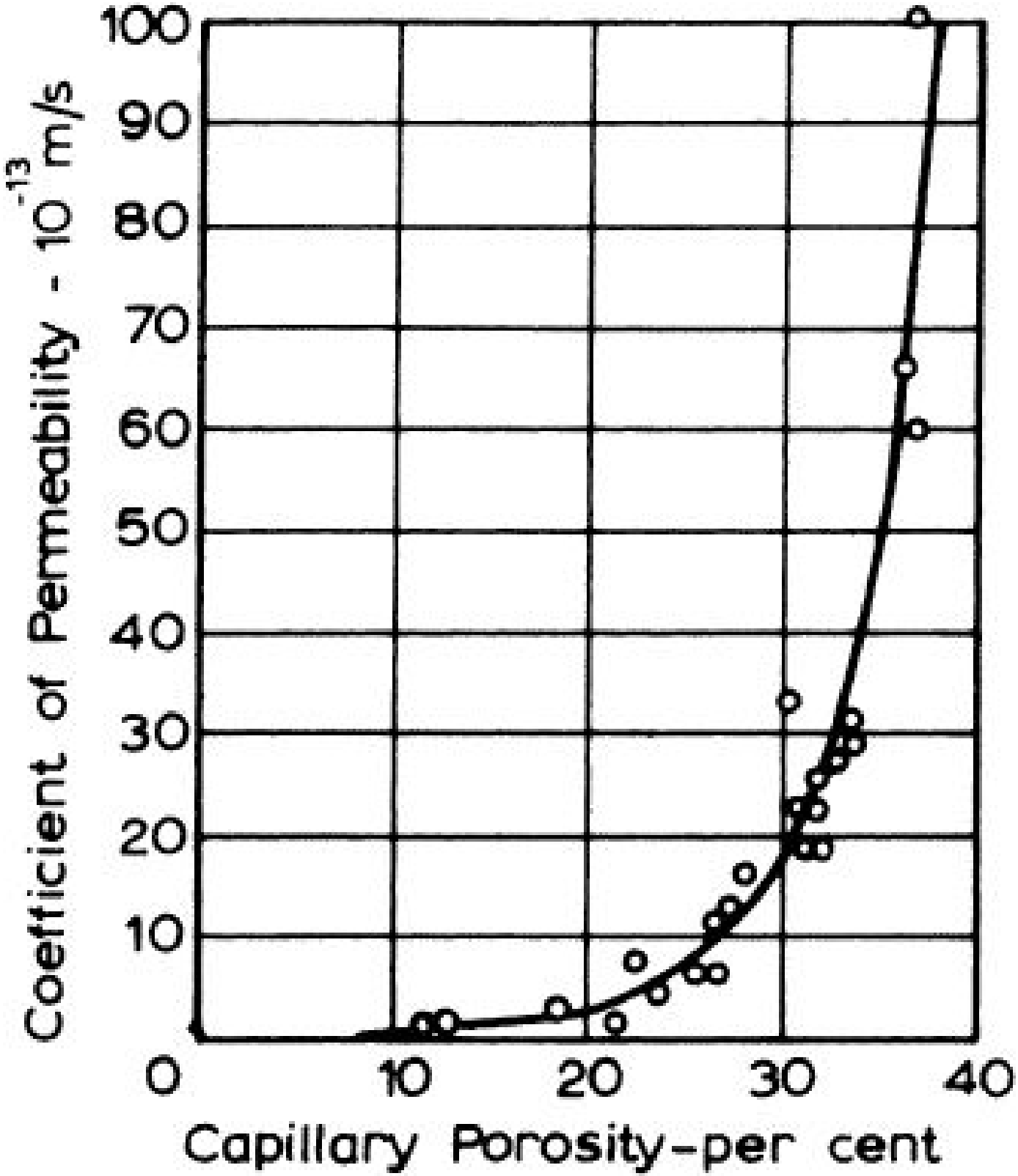


Fig. 10.2. Relation between permeability and capillary porosity of cement paste[10.3]

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Table 10.2. Comparison Between Permeabilities of Rocks and Cement Pastes[10.3]

Type of rock	*Coefficient of permeability m/s*	*Water/cement ratio of mature paste of the same permeability*
Dense trap	2.47×10^{-14}	0.38
Quartz diorite	8.24×10^{-14}	0.42
Marble	2.39×10^{-13}	0.48
Marble	5.77×10^{-12}	0.66
Granite	5.35×10^{-11}	0.70
Sandstone	1.23×10^{-10}	0.71
Granite	1.56×10^{-10}	0.71

The permeability of cement paste varies with the progress of hydration. In a fresh paste, the flow of water is controlled by the size, shape, and concentration of the original cement particles. With the progress of hydration, the permeability decreases rapidly because the gross volume of gel (including the gel pores) is approximately 2.1 times the volume of the unhydrated cement, so that the gel gradually fills some of the original water-filled space. In a mature paste, the permeability depends on size, shape, and concentration of the gel particles and on whether or not the capillaries have become discontinuous.[10.4] Table 10.3 gives values of the coefficient of permeab-

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ility[10.5] at different ages for a cement paste with a water/cement ratio of 0.7. The reduction in the coefficient of permeability is faster the lower the water/cement ratio of the paste, so that there is little reduction after wet curing for a period of:[10.21]

7 days when the water/cement ratio is 0.45
28 days when the water/cement ratio is 0.60
90 days when the water/cement ratio is 0.70.

Table 10.3. Reduction in Permeability of Cement Paste (Water/Cement Ratio = 0.7) with the Progress of Hydration[10.5]

Age days	*Coefficient of permeability, K m/s*
Fresh	2×10^{-6}
5	4×10^{-10}
6	1×10^{-10}
8	4×10^{-11}
13	5×10^{-12}
24	1×10^{-12}
Ultimate	6×10^{-13} (calculated)

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For cement pastes hydrated to the same degree, the permeability is lower the higher the cement content of the paste, i.e. the lower the water/cement ratio. Figure 10.3 shows values obtained for pastes in which 93 per cent of the cement has hydrated.[10.5] The slope of the line is considerably lower for pastes with water/cement ratios below about 0.6, i.e. pastes in which some capillaries have become segmented (see p. 32). From Fig. 10.3 it can be seen that a reduction of water/cement ratio from, say, 0.7 to 0.3 lowers the coefficient of permeability by 3 orders of magnitude. The same reduction occurs in a paste with a water/cement ratio of 0.7 between the ages of 7 days and one year.

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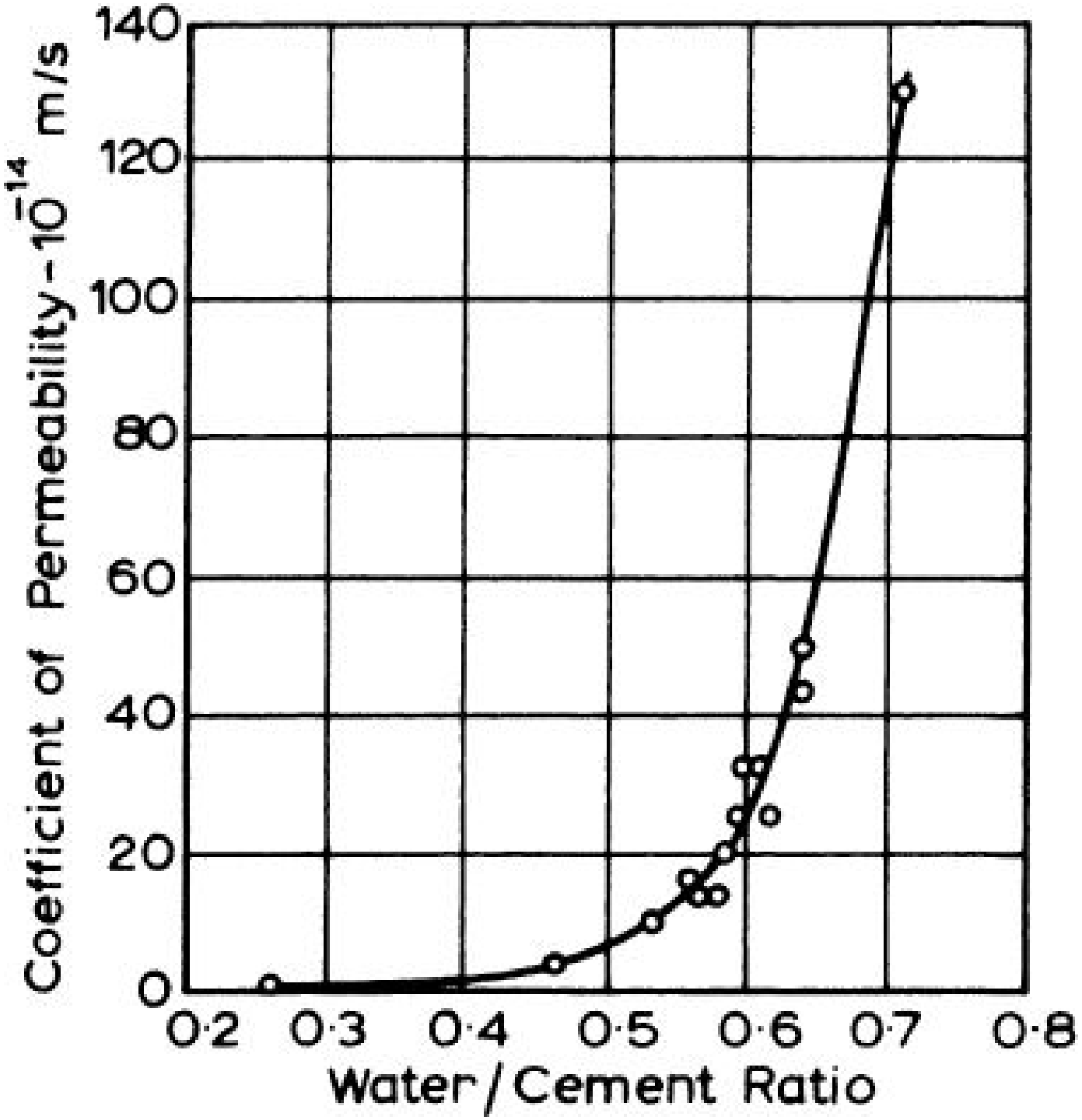


Fig. 10.3. Relation between permeability and water/cement ratio for mature cement pastes[10.5] (93 per cent of cement hydrated)

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In concrete, the value of the coefficient of permeability decreases very substantially with a decrease in the water/cement ratio: over the range of water/cement ratios of 0.75 to 0.26, the coefficient decreases by up to 4 orders of magnitude,[10.51] and over the range of 0.75 to 0.45, by 2 orders of magnitude. Specifically, at a water/cement ratio of 0.75, the coefficient of permeability is typically 10^{-10} m/s, and this would be considered to represent concrete with a high permeability. At a water/cement ratio of 0.45, the coefficient is typically 10^{-11} or 10^{-12} m/s; permeabilities of an order of magnitude lower than the last value are considered to represent concretes with a very low permeability.

In this connection, it is useful to refer again to Fig. 10.3 which applies to mature cement pastes. There is a large increase in the permeability at water/cement ratios in excess of about 0.4. In the vicinity of this water/cement ratio, the capillaries become segmented so that there is a substantial difference in permeability between mature

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cement pastes with a water/cement ratio below 0.4 and those with higher water/cement ratios. This difference has implications for ingress of aggressive ions into concrete. Permeability of concrete is also of interest in relation to water-tightness of liquid-retaining and some other structures, and also with reference to the problem of hydrostatic pressure in the interior of dams. Furthermore, ingress of moisture into concrete affects its thermal insulation properties (see pp. 376 and 708).

Increasing the wet-curing period of concrete with a very high water/cement ratio from 1 day to 7 days was found[10.51] to reduce water permeability by a factor of 5.

The permeability of concrete is affected also by the properties of cement. For the same water/cement ratio, coarse cement tends to produce a hardened cement paste with a higher porosity than a finer cement.[10.5] The compound composition of the cement affects permeability in so far as it influences the rate of hydration, but the ul-

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timate porosity and permeability are unaffected.[10.5] In general terms, it is possible to say that the higher the strength of the hardened cement paste the lower its permeability – a state of affairs to be expected because strength is a function of the relative volume of gel in the space available to it. There is one exception to this statement: drying the cement paste increases its permeability, probably because shrinkage may rupture some of the gel between the capillaries and thus open new passages to water.[10.5]

The difference between the permeability of hardened cement paste and of concrete containing a paste of the same water/cement ratio should be appreciated as the permeability of the aggregate itself affects the behaviour of the concrete (see Table 10.2). If the aggregate has a very low permeability, its presence reduces the effective area over which flow can take place. Furthermore, because the flow path has to circumvent the aggregate particles, the effective path becomes considerably longer so that the effect of

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aggregate in reducing the permeability may be considerable. The interface zone does not seem to contribute to flow. Generally, the influence of the aggregate content in the mix is small and, because the aggregate particles are enveloped by the cement paste, in fully compacted concrete it is the permeability of the hardened cement paste that has the greatest effect on the permeability of the concrete. This was referred to on p. 485.

The permeability of concrete under cryogenic conditions, e.g. to liquid nitrogen at –196 °C, involves different mechanisms because ice reduces the flow and the aggregate appears to have a substantial influence.[10.50] Typical values of the intrinsic permeability coefficient between 10^{-18} and 10^{-17} m^2 have been reported.[10.50]

Permeability testing

Testing concrete for permeability has not been generally standardized[10.123] so that the values of the coefficient of permeability quoted in different publications may not be comparable. In such tests

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as are used, the steady-state flow of water through concrete due to a pressure differential is measured, and Darcy's equation (see p. 486) is used to calculate the coefficient of permeability, *K*.

The United States Bureau of Reclamation prescribes Procedure 4913-92[10.43] in which a water pressure of 2.76 MPa (400 psi) is used; this corresponds to a head of water of 282 m. There exist also Canadian tests[10.45,10.109] and a German test prescribed in DIN 1048-1991.[10.131] In these tests, the pressure under which water is forced to flow through the concrete specimen is high, and this may alter the natural state of the concrete; blocking of some pores by silting is also possible. Moreover, during the progress of the test, hydration of the hitherto unhydrated cement can take place so that the value of the calculated coefficient of permeability decreases with time.

The U.S. Bureau of Reclamation Procedure 4913-92[10.43] provides for a correction for the age

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of the specimen at test, as shown in Fig. 10.4. The Bureau of Reclamation test is relevant to the behaviour of concrete in large dams. On the other hand, for usual concrete structures, the flow of water under a high pressure is not representative of service conditions.

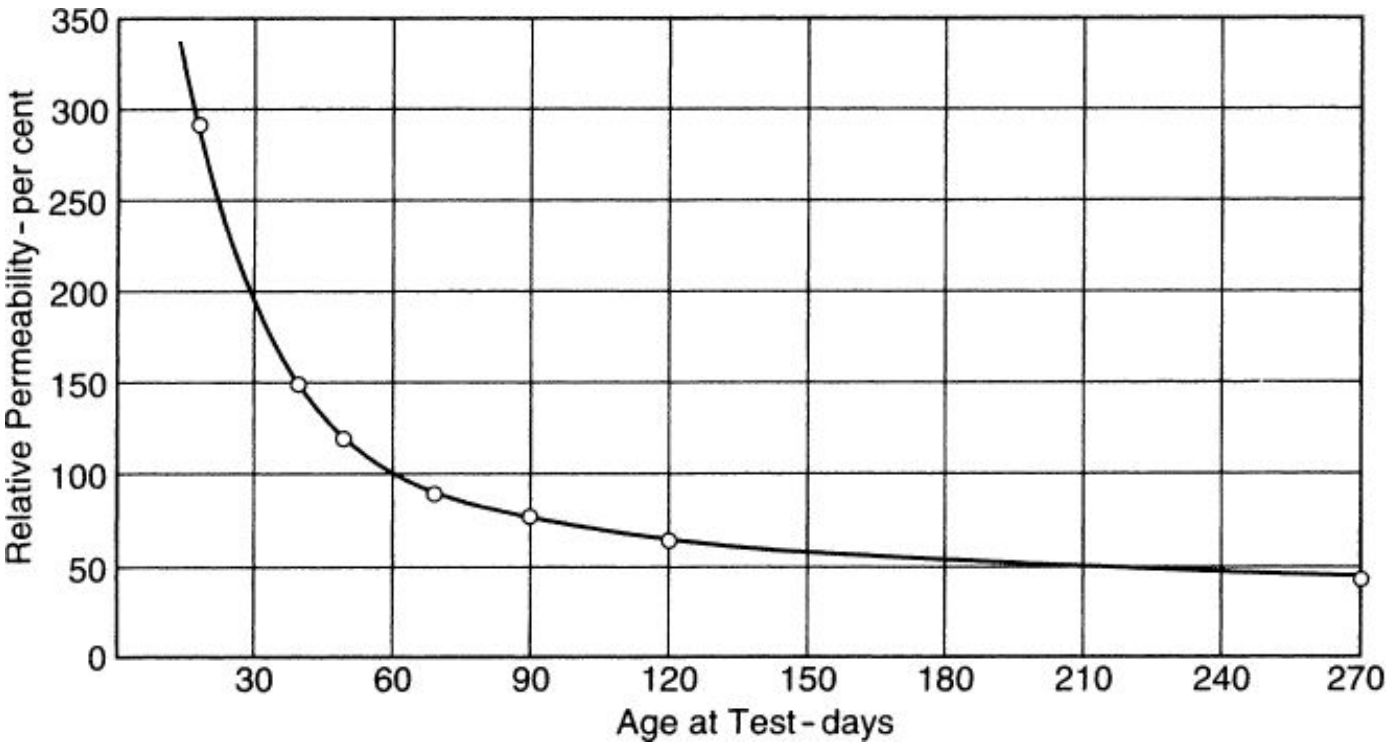


Fig. 10.4. Correction for age in U.S. Bureau of Reclamation test for permeability of concrete: the ordinate gives the permeability at any age as a percentage of permeability at the age of 60 days[10.43]

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It is important to note that the scatter of permeability test results made on similar concrete at the same age, and using the same equipment, is large. Differences such as between, say, 2×10^{-12} m/s and 6×10^{-12} m/s are not significant, so that reporting the order of magnitude, or at the most the nearest 5×10^{-12} m/s, is adequate. Smaller differences in the value of the coefficient of permeability are not significant and can be misleading.

Water penetration test

There is a further problem with permeability testing, namely that, in good quality concrete, there is no flow of water *through* the concrete. Water penetrates *into* the concrete to a certain depth, and an expression has been developed by Valenta[10.48] to convert the depth of penetration into the coefficient of permeability, K (in metres per second) equivalent to that used in Darcy's law:

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$$K = \frac{e^2 v}{2ht}$$

where e = depth of penetration of concrete in metres,

h = hydraulic head in metres,

t = time under pressure in seconds, and

v = the fraction of the volume of concrete occupied by pores.

The value of v represents discrete pores, such as air bubbles, which do not become filled with water except under pressure, and can be calculated from the increase in the mass of concrete during the test, bearing in mind that only the voids in the part of the specimen penetrated by water should be considered. Typically, v lies[10.47] between 0.02 and 0.06.

The hydraulic head is applied by pressure which usually ranges between 0.1 and 0.7 MPa.[10.21] The depth of penetration is found by observation of the split surface of the test speci-

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men (moist concrete being darker) after a given length of time. This is the value of *e* in Valenta's expression given above.

It is also possible to use the depth of penetration of water as a qualitative assessment of concrete: a depth of less than 50 mm classifies the concrete as 'impermeable'; a depth of less than 30 mm, as 'impermeable under aggressive conditions'.[10.21]

Air and vapour permeability

As mentioned earlier, the ease with which air, some gases, and water vapour can penetrate into concrete is relevant to the durability of concrete under various conditions of exposure. Distinction should be made between a situation when the driving force is a pressure differential, on the one hand, and, on the other hand, a situation when the pressure and temperature are the same on two sides of a concrete specimen or member, but two different gases sweep the two sides. In the latter case, the gases move through the concrete by dif-

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fusion whereas in the former we are concerned with permeability.

Lawrence[10.52] has reviewed the derivation and measurement of diffusivity of concrete to gas, measured in square metres per second, and he has shown that, on a log–log scale, diffusivity is linearly related to the intrinsic permeability of concrete, measured in square metres. An example of this relation for oxygen is shown in Fig. 10.5. The relation can be exploited to establish the value of diffusivity from tests on permeability, which are easier to perform.[10.52]

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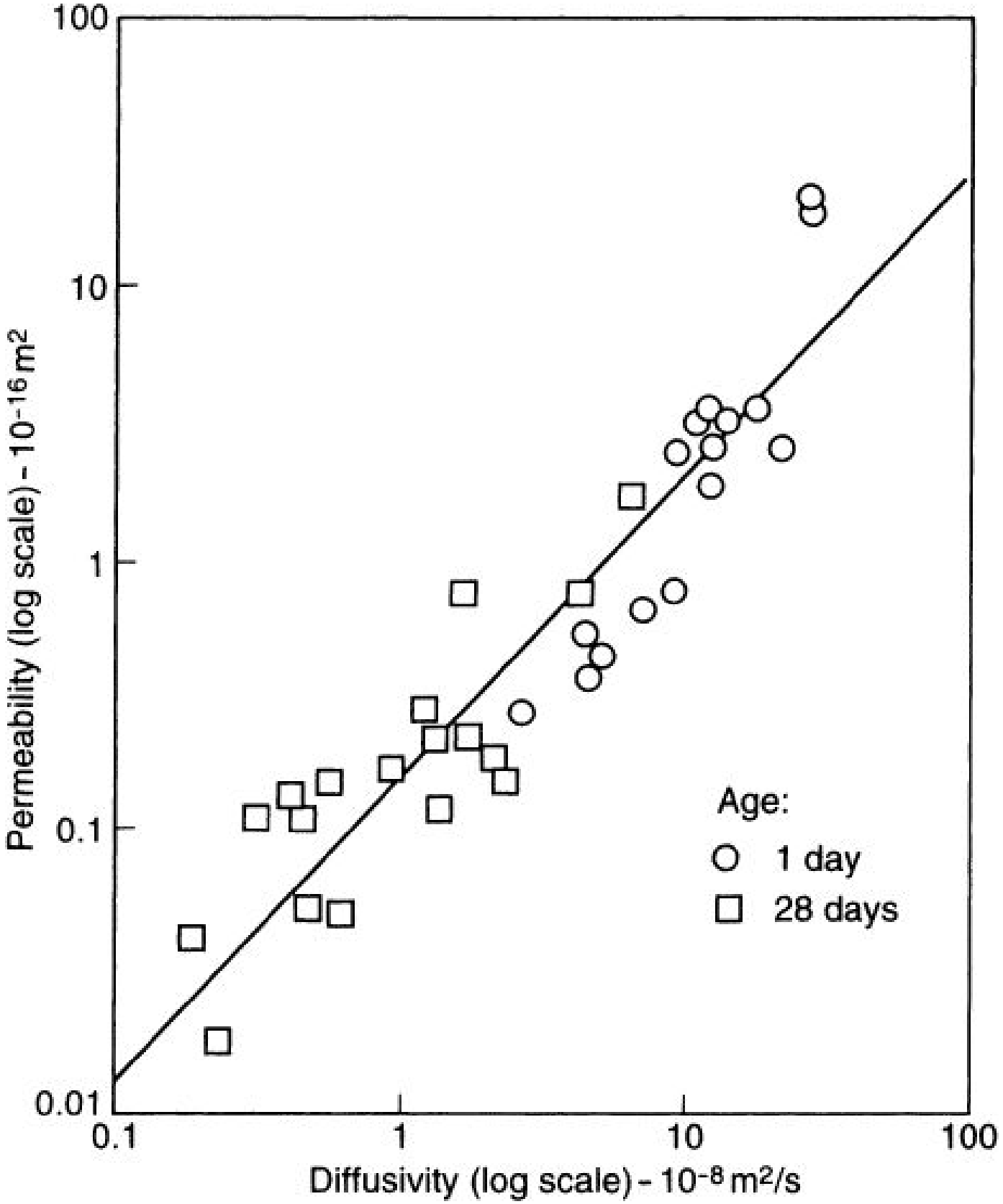


Fig. 10.5. Relation between intrinsic permeability and diffusivity of concrete[10.52]

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Because gases are compressible, the pressure, p_0, at which the volume flow rate, q (in m^3/s), is measured, has to be taken into account, in addition to the inlet pressure, p, and outlet pressure, p_a; all pressures are absolute values in N/m^2.

The intrinsic permeability coefficient, K, expressed in m^2, is:[10.96]

$$K = \frac{2qp_0L\eta}{A(p^2 - p_a^2)}$$

where A = cross-sectional area of the specimen in m^2,

L = its thickness in m, and

η = dynamic viscosity in $N\ s/m^2$.

For oxygen at 20 °C, $\eta = 20.2 \times 10^{-6}\ N\ s/m^2$.

Theoretically, the intrinsic permeability coefficient of a given concrete should be the same regardless of whether a gas or a liquid is used in the tests. However, gases yield a higher value

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of the coefficient because of the phenomenon of gas slippage; this means that, at the flow boundary, the gas has a finite velocity. The difference between gas permeability and liquid permeability is larger at lower values of the intrinsic permeability coefficient, the ratio of the former to the latter ranging from about 6 to nearly 100.[10.132]

Air permeability is greatly affected by curing, especially in concretes of low and moderate strengths.[10.92] Figure 10.6 shows this effect for concrete cured for 28 days: (a) in water, and (b) in air at a relative humidity of 65 per cent, and subsequently stored for one year in air at 20 °C at a relative humidity of 65 per cent.

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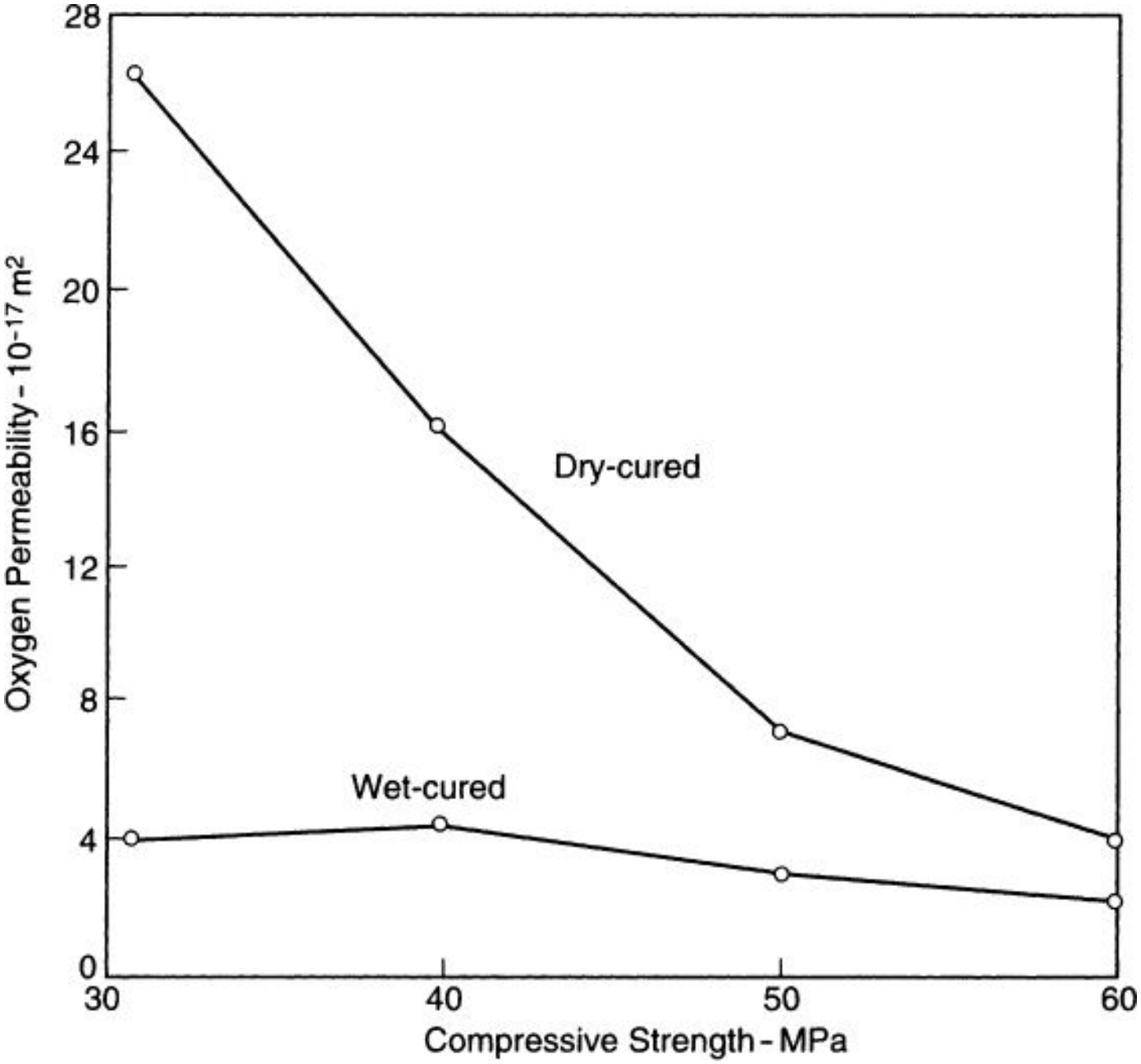


Fig. 10.6. Relation between oxygen permeability and compressive strength for concretes cured for 28 days in water and in air at a relative humidity of 65 per cent (based on ref. 10.92)

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For the purposes of illustration,[10.132] it can be mentioned that the order of magnitude of intrinsic permeability (using gas) of concrete with a water/cement ratio of 0.33 is 10^{-18} m^2.

The air permeability of concrete is strongly affected by its moisture content: a change from near saturation to an oven-dried condition has been reported to increase the gas permeability coefficient by nearly 2 orders of magnitude. For this reason, a clearly defined condition of concrete should be used in all tests. From the standpoint of the ease of testing, the oven-dried condition is preferred. However, this condition is not representative of the concrete in service, and it is the permeability of concrete to oxygen under actual conditions that is of importance in connection with corrosion of reinforcing steel.

Conditioning a specimen in air at a constant relative humidity, even for as long as 28 days, does not necessarily result in a uniform moisture condition within the concrete.[10.59]

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The permeability of concrete to oxygen can be determined by a method developed by Cembureau.[10.53] However, there is no generally accepted test method.

Carbonation

Discussion of the behaviour of concrete is generally based on the assumption that the ambient medium is air which does not react with hydrated cement paste. However, in reality, air contains CO_2 which, in the presence of moisture, reacts with hydrated cement; the actual agent is carbonic acid because gaseous CO_2 is not reactive.

The action of CO_2 takes place even at small concentrations such as are present in rural air, where the CO_2 content is about 0.03 per cent by volume. In an unventilated laboratory, the content may rise above 0.1 per cent; in large cities it is on average 0.3 per cent and, exceptionally, up to 1 per cent. An example of concrete exposed to a very high concentration of CO_2 is offered by the lining of vehicular tunnels. The rate of car-

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bonation of concrete increases with an increase in the concentration of CO_2, especially at high water/cement ratios,[10.107] the transport of CO_2 taking place through the pore system in hardened cement paste.

Of the hydrates in the cement paste, the one which reacts with CO_2 most readily is $Ca(OH)_2$, the product of the reaction being $CaCO_3$, but other hydrates are also decomposed, hydrated silica, alumina, and ferric oxide being produced.[10.7] Theoretically, such a complete decomposition of calcium compounds in hydrated cement is chemically possible even at the low concentration of CO_2 in the normal atmosphere,[10.101] but this is not a problem in practice. In concrete containing Portland cement only, it is solely the carbonation of $Ca(OH)_2$ that is of interest. When, however, $Ca(OH)_2$ becomes depleted, for instance by secondary reaction with pozzolanic silica, the carbonation of calcium silicate hydrate, C-S-H, is also possible. When this occurs, not only is more $CaCO_3$ formed, but also the silica gel which is

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concurrently formed has large pores, larger than 100 nm, which facilitates further carbonation.[10.67] The carbonation of C-S-H is discussed later in connection with carbonation of concretes made with blended cements.

Effects of carbonation

Carbonation *per se* does not cause deterioration of concrete but it has important effects. One of these is carbonation shrinkage, which was discussed on p. 444. With respect to durability, the importance of carbonation lies in the fact that it reduces the pH of the pore water in hardened Portland cement paste from between 12.6 to 13.5 to a value of about 9. When all $Ca(OH)_2$ has become carbonated, the value of pH is reduced to 8.3.[10.35] The significance of the lowering of the pH is as follows.

Steel embedded in hydrating cement paste rapidly forms a thin *passivity layer* of oxide which strongly adheres to the underlying steel and gives it complete protection from reaction

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with oxygen and water, that is from formation of rust or corrosion; corrosion is discussed in Chapter 11. This state of the steel is known as *passivation*. Maintenance of passivation is conditional on an adequately high pH of the pore water in contact with the passivating layer. Thus, when the low pH front reaches the vicinity of the surface of the reinforcing steel, the protective oxide film is removed and corrosion can take place, provided oxygen and moisture necessary for the reactions of corrosion are present. For this reason, it is important to know the depth of carbonation and specifically whether the carbonation front has reached the surface of the embedded steel. In fact, because of the presence of coarse aggregate, the 'front' does not advance as a perfectly straight line. It might also be noted that, if cracks are present, CO_2 can ingress through them so that the 'front' advances locally from the penetrated cracks. In many cases, corrosion can take place even when the full carbonation front is still a few millimetres away from

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the surface of the steel if partial carbonation has taken place.[10.61]

Rate of carbonation

Carbonation occurs progressively from the outside of concrete exposed to CO_2, but does so at a decreasing rate because CO_2 has to diffuse through the pore system, including the already carbonated surface zone of concrete. Such diffusion is a slow process if the pores in hydrated cement paste are filled with water because the diffusion of CO_2 in water is 4 orders of magnitude slower than in air. On the other hand, if there is insufficient water in the pores, CO_2 remains in gaseous form and does not react with the hydrated cement. It follows that the rate of carbonation depends on the moisture content of the concrete, which varies with the distance from its surface. Because of this variable situation, the rate of transport of CO_2 to the advancing carbonation front in the concrete cannot be readily determined from the diffusion equation (see p. 487). The rela-

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tion between diffusivity and intrinsic permeability shown in Fig. 10.5 may possibly be exploited.

The highest rate of carbonation occurs at a relative humidity of between 50 and 70 per cent. This situation can be viewed against the background of a typical relative humidity of 65 per cent in an ordinary laboratory; outdoors in southern England, the average relative humidity is 86 per cent in winter and 73 per cent in summer.

Under steady hygrometric conditions, the depth of carbonation increases in proportion to the square root of time, which is characteristic of sorption rather than diffusion, but carbonation involves an interaction between CO_2 and the pore system. It is thus possible to express the depth of carbonation, D, in millimetres as

$$D = Kt^{0.5}$$

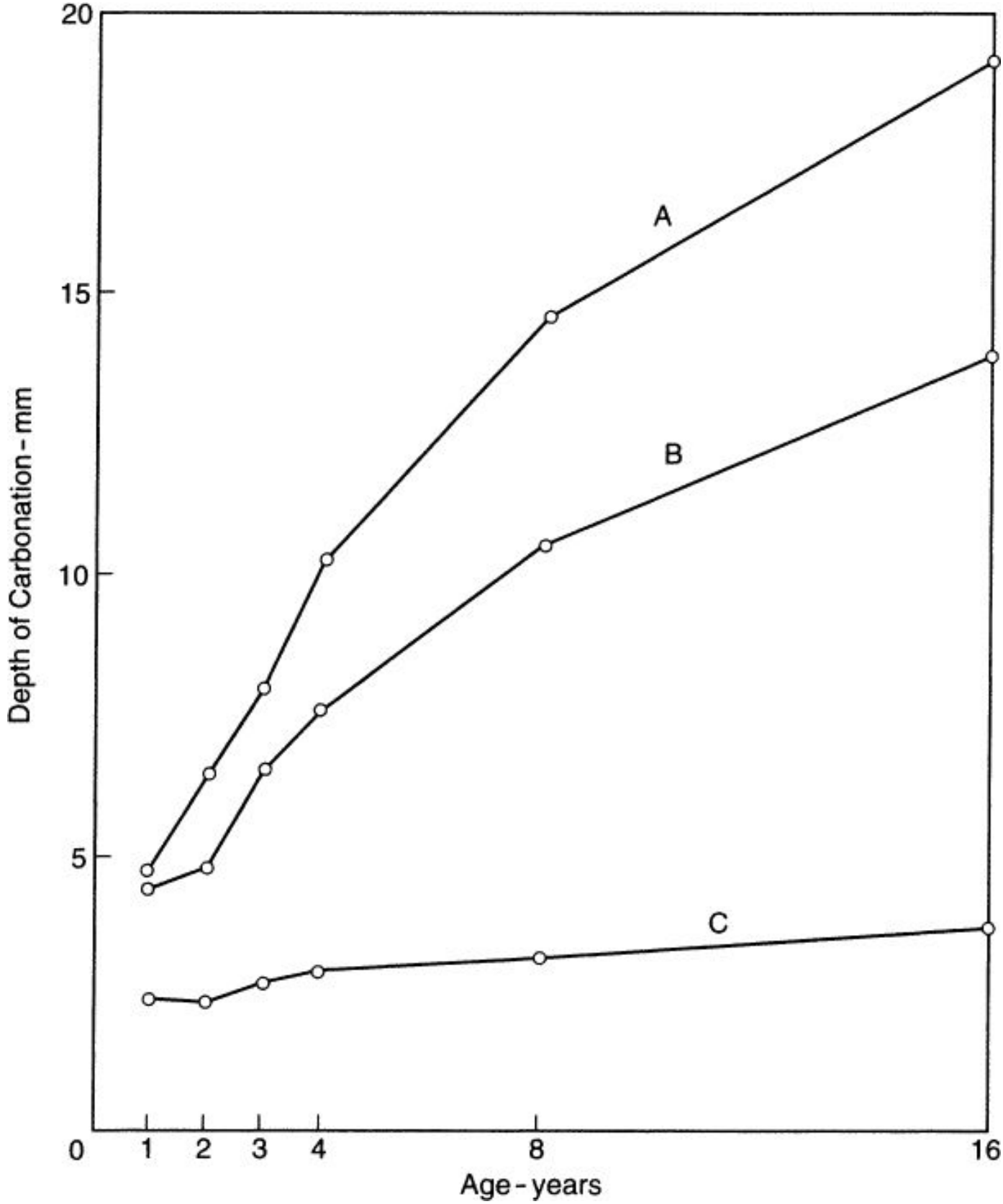
where K = carbonation coefficient in mm/year$^{0.5}$, and

t = time of exposure in years.

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The values of K are often more than 3 or 4 mm/year$^{0.5}$ for low-strength concrete.[10.58] Another way of giving a broad picture is to say that, in concrete with a water/cement ratio of 0.60, a depth of carbonation of 15 mm would be reached after 15 years, but at a water/cement ratio of 0.45 only after 100 years. An example[10.124] of the progress of carbonation over a period of 16 years is shown in Fig. 10.7.

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Fig. 10.7. Progress of carbonation with time of exposure under different conditions: (*A*) 20 °C and 65 per cent relative humidity; (*B*) outdoors, protected by a roof; (*C*) horizontal surface outdoors in Germany. The values are averages for concretes with water/cement ratios of 0.45, 0.60, and 0.80, wet-cured for 7 days (based on ref. 10.124)

The expression involving the square root of time is not applicable when the exposure conditions are not steady. In particular, if the surface of the concrete is exposed to a variable humidity, with periodic wetting, the rate of carbonation is reduced because of a slowing down of the diffusion of CO_2 through saturated pores in the hardened cement paste. Conversely, sheltered parts of a structure undergo carbonation at a faster rate than those exposed to rain, which significantly slows down the progress of carbonation. In the interior of buildings, the rates of carbonation can be high, but there are no ill consequences of this in so far as corrosion of em-

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bedded steel is concerned (see p. 565) unless the carbonated concrete is subsequently wetted. This can arise when water ingresses through the cladding of a building far enough to reach the zone carbonated from inside.

The very considerable influence of the moisture content of the concrete upon carbonation means that, even in a single building, made all of the same concrete, there may be a considerable variation in the depth of carbonation at a given age: the walls more exposed to rain will have a lower depth of carbonation; so will sloping surfaces which can be washed down by rain; the same applies to walls which can be thoroughly dried by strong insolation. Overall, the greatest depth of carbonation can be 50 per cent more than the smallest depth.[10.57]

Small variations in temperature have little effect on carbonation but a high temperature increases the rate of carbonation unless drying overshadows the temperature effect.

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The physico-chemical phenomena influencing the rate of carbonation are discussed by Papadakis *et al.*[10.56]

Factors influencing carbonation

The fundamental factor controlling carbonation is the diffusivity of the hardened cement paste. The diffusivity is a function of the pore system of the hardened cement paste during the period when the diffusion of CO_2 takes place. It follows that the type of cement, the water/cement ratio, and the degree of hydration are relevant. All these influence also the strength of the concrete containing any given hardened cement paste. For this reason, it is often said that the rate of carbonation is simply a function of the strength of concrete. This, while broadly true, is an inadequate simplification. What makes the use of this approach worse is that the value of strength quoted is not that applicable to the concrete in situ when *it* becomes exposed to CO_2 but, usually, the value of strength of test specimens cured in a standard

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manner, which is invariably superior to curing in situ.

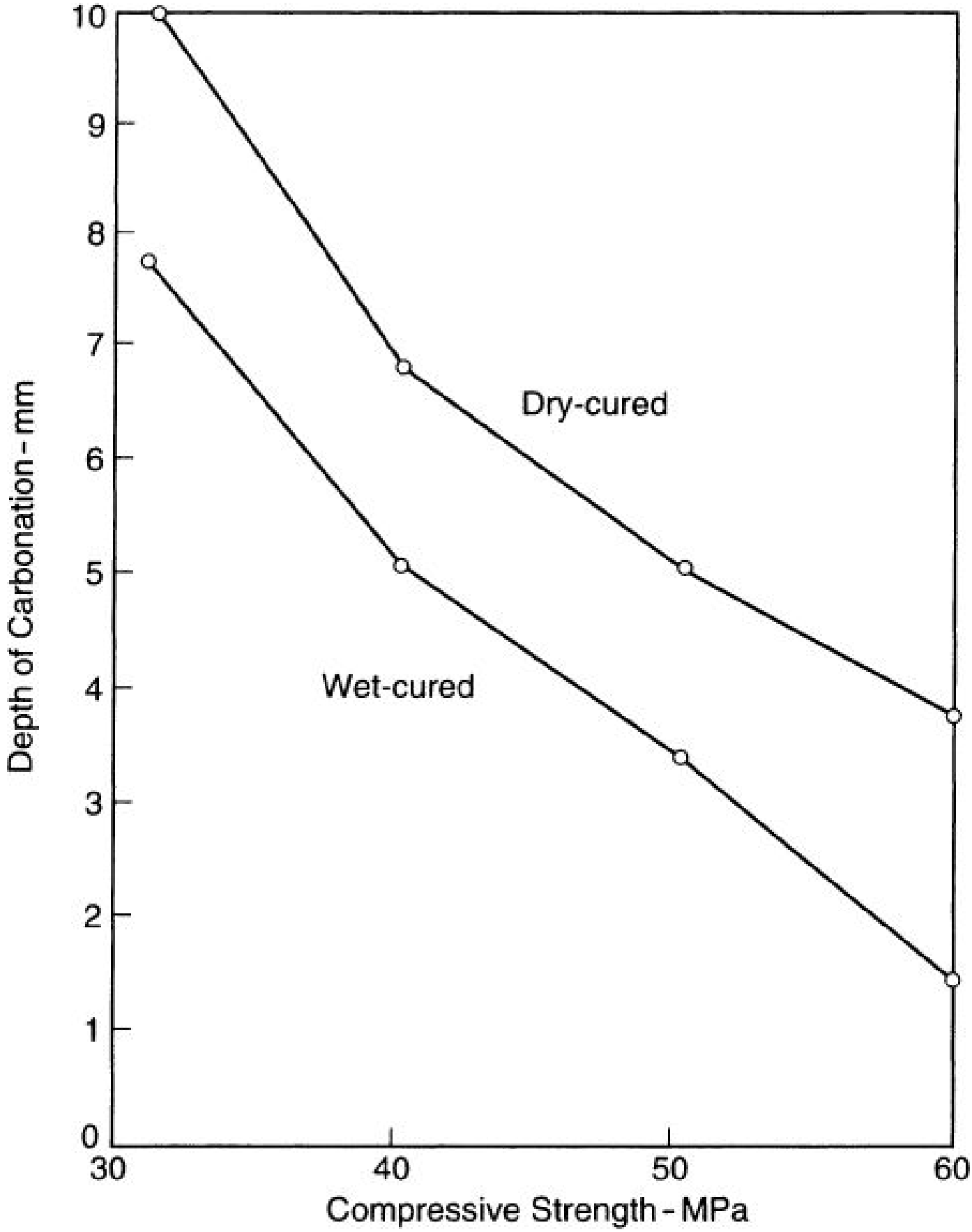
Alternatives to the use of strength as a parameter include expressing carbonation as a function of the water/cement ratio or of the cement content, or of both of these. There is no physical basis for considering the cement content; as far as the water/cement ratio is concerned, such an approach is not superior to the use of strength as a parameter. Indeed, neither strength nor water/cement ratio is informative about the microstructure of the hardened cement paste in the surface zone of concrete *while* the diffusion of CO_2 is taking place. A factor which has a great influence on the outer zone is the curing history of the concrete.

The effect of curing on carbonation of concrete is substantial. Figure 10.8 shows the depth of carbonation of concretes with 28-day compressive strength (measured on standard test cubes) between 30 and 60 MPa: (a) cured in water for 28 days, and (b) cured in air at a relative humidity of 65 per cent; thereafter, all specimens

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were stored for two years at 20 °C and a relative humidity of 65 per cent.[10.92] The detrimental effect of the absence of wet curing, which results in high porosity, is marked. Other research workers[10.133] have reported that increasing the period of wet curing from 1 day to 3 days reduces the depth of carbonation by about 40 per cent.

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Fig. 10.8. Relation between the depth of carbonation and compressive strength of concrete after 2 years' exposure in air at a relative humidity of 65 per cent (based on ref. 10.92)

It should be noted, however, that outdoor exposure in many parts of the world includes frequent or prolonged periods of high humidity so that hydration of cement continues and, in effect, delayed natural curing of the surface zone takes place. Nevertheless, generally, the effects of the absence of initial curing on carbonation persist for many years in so far as it results in a microstructure of the hardened cement paste in the outer zone of the concrete which facilitates the diffusion of CO_2.

A general statement can be made to the effect that, in a situation conducive to continuing carbonation, concrete with a strength lower than 30 MPa (4000 psi) is highly likely to have undergone carbonation to a depth of at least 15 mm in a period of several years.[10.62]

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Despite the considerable variability in the rate of carbonation in different locations, typical values reported by Parrott[10.55] and shown in Table 10.4 are of interest. Clearly, the values of Table 10.4 must not be treated as the norm. From Parrott's data[10.55] it is possible to say that, for sheltered concrete outdoors in the United Kingdom or a similar climate, in 90 per cent of cases, the depth of carbonation will not exceed the values shown in Table 10.4. For the reasons given earlier, in some cases, the depth of carbonation will exceed the 90 per cent upper bound; in others, it will be much lower. Nevertheless, the typical values given in Tables 10.4 and 10.5, as well as the other data presented in this chapter, should make it possible to ensure that the depth of carbonation expected within the intended service life of the structure is smaller than the cover to the reinforcement. Thus, the necessary depth of cover and the actual quality of concrete are interdependent, in so far as protection of reinforcement is concerned, so that at the design stage they

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should be chosen together. The topic of cover is discussed on p. 575.

Table 10.4. Depth of Carbonation as a Function of Strength[10.55]

Exposure	*Depth of carbonation after 50 years, mm*	
	25 MPa concrete	*50 MPa concrete*
Sheltered outdoors	60 to 70	20 to 30
Exposed to rain	10 to 20	1 to 2

Table 10.5. Maximum Depth of Carbonation in Sheltered Concrete Outdoors in the United Kingdom[10.55]

28-day strength MPa	*Depth of carbonation after 30 years mm*
20	45
40	17
60	5
80	2

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Carbonation of concrete containing blended cements

Because blended cements are widely used nowadays, it is important to know the carbonation behaviour of concretes containing fly ash and ground granulated blastfurnace slag. Numerous papers have been published reporting comparative carbonation tests on concretes with and without these cementitious materials, but the comparisons were made on varying bases. Such data do not lend themselves to useful generalizations, and yet what is important when selecting a concrete mix is to assess the carbonation characteristics of the specific proposed mix.

The starting point in this assessment is the knowledge of the microstructure and other properties of the hardened cement paste resulting from the use of the various cementitious materials in so far as these properties physically or chemically influence carbonation. The relevant properties are discussed in Chapter 13, but in the present context two observations should be made

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with respect to Class F fly ash. First of all, the silica in the fly ash reacts with $Ca(OH)_2$ resulting from the hydration of Portland cement. In consequence, the blended cement leads to a lower $Ca(OH)_2$ content in the hardened cement paste so that a smaller amount of CO_2 is required to remove all the $Ca(OH)_2$ by producing $CaCO_3$. Bier[10.67] has shown that the depth of carbonation is greater when the amount of $Ca(OH)_2$ present is lower. It follows that the presence of fly ash results in a more rapid carbonation. There is, however, another effect of the reaction between the pozzolanic silica and $Ca(OH)_2$, namely it results in a denser structure of the hardened cement paste so that its diffusivity is reduced and carbonation is slowed down.

The question to ask is: which effect is dominant? One important factor is the quality of curing. Good curing is necessary for the pozzolanic reactions to take place (see p. 658), and yet tests involving only 1-day curing when concrete contains fly ash have been made;[10.55,10.66] such tests

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are destined to demonstrate high carbonation of concrete with fly ash but they are predicated on bad concrete practice. The effects of inadequate curing upon carbonation of concrete containing fly ash persist even in the long term.[10.63] On the other hand, concretes made with cement containing fly ash up to 30 per cent and with actual strengths above 35 MPa (5000 psi) have shown no increase, or only a marginal increase, in carbonation when fly ash is included in the mix.[10.63,10.67]

The use of ground granulated blastfurnace slag in the concrete mix entails an even greater necessity for good curing. In consequence, poorly cured concrete containing blastfurnace slag exhibits very high carbonation: depths of 10 to 20 mm after one year's exposure have been reported.[10.64] High slag contents lead to a greater depth of carbonation.[10.64,10.65] However, when the blastfurnace slag content in the blended cement is below 50 per cent and the concrete is exposed to

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CO_2 at a concentration of 0.03 per cent, there is only a marginal increase in carbonation.[10.67]

In view of the use of fillers in modern cements (see p. 88) it is useful to mention that they have no effect on the microstructure of the hardened cement paste and, therefore, they do not influence carbonation.[10.60]

Sulfate-resisting cement leads to a 50 per cent greater depth of carbonation than Portland cement.[10.108] For this reason, increased cover to reinforcement may be required when sulfate-resisting cement is used. Carbonation of concrete containing regulated set cement is also increased.[10.137]

Carbonation takes place also in high-alumina cement concrete but, as hydration of that cement does not produce $Ca(OH)_2$, it is calcium aluminate hydrates CAH_{10} and C_3AH_6 that react with CO_2. The end products are $CaCO_3$ and alumina gel, which have a lower strength than the hydrates. At the same strength as Portland cement

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concrete, high-alumina cement concrete exhibits twice as high carbonation.[10.124]

The carbonation of hardened high-alumina cement paste may lead to the depassivation of reinforcing steel, which in any case is in contact with pore water at a pH lower than in the case of Portland cement, namely between 11.4 and 11.8. The rate of carbonation of high-alumina-cement paste which has undergone conversion (see p. 95) is much higher than prior to conversion.

Measurement of carbonation

Laboratory techniques which can be used to determine the depth of carbonation include chemical analysis, X-ray diffraction, infra-red spectroscopy and thermogravimetric analysis. A common and simple method to establish the extent of carbonation is by treating a freshly broken surface of concrete with a solution of phenolphthalein in diluted alcohol. The free $Ca(OH)_2$ is coloured pink while the carbonated portion is uncoloured; with progress of carbonation of the

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newly exposed surface, the pink colouring gradually disappears. The procedure is prescribed by RILEM.[10.54] The test is easy to perform and is rapid but it should be remembered that the pink colour indicates the presence of $Ca(OH)_2$ but not necessarily a total absence of carbonation. Indeed, the phenolphthalein test gives a measure of the pH (the colour being pink above about 9.5) but does not distinguish between a low pH caused by carbonation and by other acidic gases. As far as the risk of corrosion of reinforcement is concerned, the cause of a low value of pH may not be important but care is required in interpreting the observed colour pattern.

The phenolphthalein test cannot be used with high-alumina cement because, unlike Portland cement, it does not contain free lime.

If breaking off a concrete sample is not practicable, dust samples of concrete can be obtained by drilling to successively greater depths; the samples are then subjected to the phenolphthalein test. Care is required because, if free lime from

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uncarbonated concrete contaminates a sample, the entire sample will turn pink, giving the impression of an absence of carbonation.

Under some circumstance, measurement of the carbonation front from crack surfaces could be used to show that the crack is old; when cracks of known age exist, comparisons could indicate the age of the given crack.[10.140]

To determine how rapidly a given concrete is likely to undergo carbonation, accelerated testing can be used. This consists of exposing a concrete specimen to concentrated CO_2 of c_s per cent. The depth of carbonation after a certain period of exposure, t_t, can then be transformed into an estimate of the length of time, t_s, for the same depth to be reached at the service concentration of CO_2 of c_s per cent, on the basis that the time is inversely proportional to the concentration of CO_2:

$$t_t : t_s = c_s : 100.$$

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In the Swiss method,[10.125] now discontinued, test concentration of CO_2 of 100 per cent is used; more often the concentration is 4 or 5 per cent.[10.36] For carbonation to proceed, the relative humidity should be 60 to 70 per cent.

Considerable care is required in interpreting accelerated tests, not only because in situ carbonation is greatly influenced by the exact exposure conditions, especially with respect to wetting by rain and drying by sun and wind, but also because a high concentration of CO_2 distorts the phenomena involved. For example, Bier,[10.67] using a concentration of CO_2 of 2 per cent, found that the depth of carbonation of well-cured concrete containing fly ash or blastfurnace slag is greater by a factor of at least 2 than when Portland cement only is present. There was no such increase in the depth of carbonation where the CO_2 concentration was 0.03 per cent and the content of fly ash was below 30 per cent or of blastfurnace slag below 50 per cent. A likely explanation of this difference in behaviour is that, at a high concentra-

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tion of CO_2, the carbonation of $Ca(OH)_2$ was followed by the carbonation of C-S-H.

Extensive carbonation of C-S-H in concrete in service has been reported by Kobayashi *et al.*,[10.110] but information on the type of cement used is not available.

Further aspects of carbonation

Carbonation can have some positive consequences. Because $CaCO_3$ occupies a greater volume than $Ca(OH)_2$ which it replaces, the porosity of carbonated concrete is reduced. Also, water released by $Ca(OH)_2$ on carbonation may aid the hydration of hitherto unhydrated cement. These changes are beneficial and they result in increased surface hardness, increased strength at the surface,[10.104] reduced surface permeability,[10.102] reduced moisture movement,[10.103] and increased resistance to those forms of attack which are controlled by permeability. On the other hand, carbonation accelerates chloride-induced corrosion of reinforcement (see p. 572).

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Unlike Portland cement concrete, with supersulfated cement there is a loss of strength on carbonation but, because this applies to the surface zone of the concrete only, the loss is not structurally significant.

Because carbonation affects the porosity and also the pore size distribution (causing a decrease in the volume of pores, especially of the smaller ones) of the outer zone of concrete, the penetration of paint into concrete will vary. In consequence, the bond of paint and the colouring are affected by carbonation.[10.100] Because the latter depends on the relative humidity of air and on age, it is easy to see how differences in colouring and quality of painting can readily arise.

Sakuta *et al*.[10.138] have proposed the use of additives that absorb carbon dioxide which has entered the concrete, thereby preventing carbonation.

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Acid attack on concrete

Concrete is generally well resistant to chemical attack, provided an appropriate mix is used and the concrete is properly compacted. There are, however, some exceptions.

First of all, concrete containing Portland cement, being highly alkaline, is not resistant to attack by strong acids or compounds which may convert to acids. Consequently, unless protected, concrete should not be used when this form of attack may occur.

Generally speaking, chemical attack of concrete occurs by way of decomposition of the products of hydration and formation of new compounds which, if soluble, may be leached out and, if not soluble, may be disruptive in situ. The attacking compounds must be in solution. The most vulnerable cement hydrate is $Ca(OH)_2$, but C-S-H can also be attacked. Calcareous aggregates are also vulnerable.

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Of the common forms of attack, that by CO_2 was considered in the preceding section, whereas the attack by sulfates and the action of sea water will be discussed later in the present chapter. Comprehensive lists of substances which attack concrete to varying degrees can be found in ACI 515.1R (Revised 1985),[10.93] ACI 201.2R-92,[10.42] and in a book by Biczok.[10.71] A limited composite extract is given in Table 10.6. Additionally, specific mention of some aggressive substances is made below.

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Table 10.6. A List of Some Substances which Cause Severe Chemical Attack of Concrete

Acids	
Inorganic	*Organic*
Carbonic	Acetic
Hydrochloric	Citric
Hydrofluoric	Formic
Nitric	Humic
Phosphoric	Lactic
Sulfuric	Tannic
Other substances	
Aluminium chloride	Vegetable and animal fats
Ammonium salts	Vegetable oils
Hydrogen sulfide	Sulfates

Concrete can be attacked by liquids with a pH value below 6.5[10.26] but the attack is severe only at a pH below 5.5; below 4.5, the attack is very severe. A concentration of CO_2 of 30 to 60 ppm results in a severe attack, and above 60 ppm results in a very severe attack.

The attack progresses at a rate approximately proportional to the square root of time because the attacking substance has to travel though the

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residual layer of the low-solubility products of reaction which remain after $Ca(OH)_2$ has been dissolved. Thus it is not only pH but also the ability of aggressive ions to be transported that influence the progress of the attack.[10.26] Also, the rate of attack decreases when aggregate has become exposed because the vulnerable surface is smaller and the attacking medium has to travel around the particles of aggregate.[10.26]

Concrete is also attacked by water containing free CO_2, such as moorland water or mineral waters, which may also contain hydrogen sulfide. Not all CO_2 is aggressive because some of it is required to form and stabilize calcium bicarbonate in the solution. Flowing pure water, formed by melting ice or by condensation (for instance, in a desalination plant) and containing little CO_2, also dissolves $Ca(OH)_2$, thus causing surface erosion. Peaty water with CO_2 is particularly aggressive; it can have a pH value as low as 4.4.[10.31] This type of attack may be of importance in conduits in mountain regions, not only from the standpoint

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of durability but also because the leaching out of hydrated cement leaves behind protruding aggregate and increases the roughness of the pipe. To avoid this, the use of calcareous, rather than siliceous, aggregate is advantageous[10.10] because both the aggregate and the cement paste are eroded.

Acid rain, which consists mainly of sulfuric acid and nitric acid and has a pH value between 4.0 and 4.5, may cause surface weathering of exposed concrete.[10.70]

Although domestic sewage by itself is alkaline and does not attack concrete, severe damage of sewers has been observed in many cases, especially at fairly high temperatures,[10.10] when sulfur compounds become reduced by anaerobic bacteria to H_2S. This is not a destructive agent in itself, but it is dissolved in moisture films on the exposed surface of the concrete and undergoes oxidation by aerobic bacteria, finally producing sulfuric acid. The attack occurs, therefore, above the level of flow of the sewage. The hardened ce-

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ment paste is gradually dissolved, and progressive deterioration of concrete takes place.[10.27] A somewhat similar form of attack can occur in offshore oil storage tanks.[10.134]

Sulfuric acid is particularly aggressive because, in addition to the sulfate attack of the aluminate phase, acid attack on $Ca(OH)_2$ and C-S-H takes place. Reduction in the cement content of the concrete is therefore beneficial,[10.78] provided, of course, that the density of the concrete is unimpaired.

Concrete is generally resistant to microbiological attack because its high pH does not encourage such action; nevertheless, under certain, fortunately rare, tropical conditions, some algae, fungi and bacteria can use atmospheric nitrogen to form nitric acid which attacks concrete.[10.73]

Lubricating oils and hydraulic fluid, sometimes spilt on concrete aprons at airports, break down when heated by exhaust gases and react with $Ca(OH)_2$, thus causing leaching.[10.69]

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Various physical and chemical tests on the resistance of concrete to acids have been developed,[10.7] but there are no standard procedures. It is essential that tests are performed under realistic conditions because, when a concentrated acid is used, all cements dissolve and no assessment of their relative quality is possible. For this reason, care is required in interpreting the results of accelerated tests.

The need for testing under specific conditions arises from the fact that pH alone is not an adequate indicator of the potential attack: the presence of CO_2 in relation to the hardness of water also influences the situation. An increase in the rate of flow of the attacking medium, and in its temperature and pressure, all increase the attack.

Use of blended cements which include ground granulated blastfurnace slag, pozzolanas, and especially silica fume, is beneficial in reducing the ingress of aggressive substances. Pozzolanic action also fixes $Ca(OH)_2$, which is usually the most vulnerable product of hydration of cement

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in so far as acid attack is concerned. However, the performance of concrete depends more on its quality than on the type of cement used. The resistance of concrete to chemical attack is increased by allowing it to dry out before exposure, but following proper curing. A thin layer of calcium carbonate (produced by the action of CO_2 on lime) then forms, blocking the pores and reducing the permeability of the surface zone. It follows that precast concrete is generally less vulnerable to attack than concrete cast in situ. Good protection of concrete from acid attack is obtained by subjecting precast concrete in a vacuum to the action of silicon tetrafluoride gas.[10.11] This gas reacts with lime:

$$2Ca(OH)_2 + SiF_4 \rightarrow 2CaF_2 + Si(OH)_4.$$

$Ca(OH)_2$ can also be fixed by treatment with diluted water-glass (sodium silicate). Calcium silicates are then formed, filling the pores. Treatment with magnesium fluorosilicate is also possible. The pores become filled and the resistance of the

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concrete to acid is also slightly increased, probably due to the formation of colloidal silicofluoric gel. There exist numerous surface treatment methods[10.93] but this subject is outside the scope of the present book.

Sulfate attack on concrete

Solid salts do not attack concrete but, when present in solution, they can react with hydrated cement paste. Particularly common are sulfates of sodium, potassium, magnesium, and calcium which occur in soil or in groundwater. Because the solubility of calcium sulfate is low, groundwaters with a high sulfate content contain the other sulfates as well as calcium sulfate. The significance of this lies in the fact that those other sulfates react with the various products of hydration of cement and not only with $Ca(OH)_2$.

Sulfates in groundwater are usually of natural origin but can also come from fertilizers or from industrial effluents. These sometimes contain ammonium sulfate, which attacks hydrated cement

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paste[10.83] by producing gypsum.[10.95] Soil in some disused industrial sites, particularly gas works, may contain sulfates and often also other aggressive substances. Sulfides can oxidize to sulfates under some conditions, e.g. under compressed air used in excavation.

The reactions of the various sulfates with hardened cement paste are as follows.

Sodium sulfate attacks $Ca(OH)_2$:

$$Ca(OH)_2 + Na_2SO_4.10H_2O \rightarrow CaSO_4.2H_2O + 2NaOH + 8H_2O.$$

This is an acid-type attack. In flowing water, $Ca(OH)_2$ can be completely leached out but if NaOH accumulates, equilibrium is reached, only a part of the SO_3 being deposited as gypsum.

The reaction with calcium aluminate hydrate can be formulated as follows:[10.7]

$$2(3CaO.Al_2O_3.12H_2O) + 3(Na_2SO_4.10H_2O) \rightarrow$$

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$$3CaO.Al_2O_3.3CaSO_4.32H_2O + 2Al(OH)_3 + 6NaOH + 17H_2O.$$

Calcium sulfate attacks only calcium aluminate hydrate, forming calcium sulfoaluminate ($3CaO.Al_2O_3.3CaSO_4.32H_2O$), known as *ettringite*. The number of molecules of water may be 32 or 31, depending upon the ambient vapour pressure.[10.74]

On the other hand, *magnesium sulfate* attacks calcium silicate hydrates as well as $Ca(OH)_2$ and calcium aluminate hydrate. The pattern of reaction is:

$$3CaO.2SiO_2.aq + 3MgSO_4.7H_2O \rightarrow 3CaSO_4.2H_2O + 3Mg(OH)_2 + 2SiO_2.aq. + xH_2O.$$

Because of the very low solubility of $Mg(OH)_2$, this reaction proceeds to completion so that, under certain conditions, the attack by magnesium sulfate is more severe than by other sulfates. A

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further reaction between $Mg(OH)_2$ and silica gel is possible and may also cause deterioration.[10.23] The critical consequence of the attack by magnesium sulfate is the destruction of C-S-H.

Thaumasite form of sulfate attack

This type of attack occurs in concrete buried in the ground. Thaumasite problems have been encountered in a number of bridge supports in the UK but they are not very common. At temperatures below about 15 °C (59 °F) in the presence of sulfate, carbonate and water, C-S-H can convert to thaumasite, which is a non-binder, with a composition of $CaSiO_3.CaCO_3.CaSO_4.15H_2O$.[10.139] The carbonate may be present in the aggregate (limestone or dolomite) or as bicarbonate in the groundwater. Mixes containing ggbs offer resistance to thaumasite attack.

Delayed ettringite formation

This phenomenon (known as DEF) came into prominence in the 1990s and it attracted much

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academic research; the interest has abated somewhat since then.

The formation of ettringite in the expansive cement Type K was discussed on p. 448. This is an early-age controlled expansion. However, the formation of ettringite in mature concrete tends to be disruptive and harmful, and is a form of sulfate attack, resulting in the compound $3CaO.Al_2O_3.3CaSO_4.32H_2O$.

High temperature during hydration can be the result of applied heat or be due to heat generation in a large pour when natural loss of heat is inadequate. If the temperature in the interior of concrete reaches 70° to 80 °C, a slow formation of ettringite can lead to expansion and cracking. For the harmful effects to take place after return to room temperature, the concrete has to be wet or moist either permanently or intermittently.[10.143] These harmful effects are a loss of strength, a decrease in the modulus of elasticity, and sometimes cracking.

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Occasionally, there is a problem of distinguishing DEF from alkali-silica reaction. This was the issue in a court case concerning prestressed steam-cured railway sleepers (ties). One reason for confusion is that ettringite may be so fine as to look like alkali-silica gel.[10.144]

DEF is often avoidable by a selection of appropriate blended cement that does not lead to an excessive temperature rise.

Mechanisms of attack

The mechanism of the delayed ettringite expansion is still debated, there being two principal schools of thought.

Mather[10.81] and many others are of the opinion that the reaction between calcium sulfate and C_3A is topochemical, that is, it is a solid-state reaction, not involving solution and re-precipitation which would allow movement of the newly formed product away from the original location. Such movement would not result in the development of pressure. If the product of the topo-

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chemical reaction occupies a larger volume than the volume of the two original compounds, then expansive and disruptive forces are created. In the case of the reaction between calcium sulfate and $Ca(OH)_2$, there is no overall increase in volume[10.74] but, because of the differences in the solubility of C_3A and gypsum, oriented, acicular ettringite is formed at the surface of the C_3A. Thus, there is a local increase in volume and, at the same time, an increase in porosity elsewhere.[10.75]

The second school of thought whose chief protagonist is Mehta[10.83] attributes the development of expansive forces to the swelling pressure induced by the adsorption of water by the originally colloidal ettringite which precipitated in the solution in the presence of lime. Thus, the formation of ettringite *per se* is thought to be the cause of expansion. However, Odler and Glasser[10.75] point out that an uptake of water from the environment is not a necessary condition for expansion to take place. Nevertheless, expansion in-

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creases significantly under wet conditions[10.75] so that it is likely that both the mechanisms of expansion discussed above are involved at different stages.[10.82] It should be added that the concept of expansive forces induced by crystallization *per se*, advanced by some researchers, seems erroneous.

Ettringite can also form from the reaction between sulfate and C_4AF, but this ettringite is nearly amorphous and no damaging expansion has been reported.[10.75] Nevertheless, ASTM C 150-09 prescribes a limit on the combined content of C_3A and C_4AF (see p. 76), when sulfate resistance is required.

The consequences of sulfate attack include not only disruptive expansion and cracking, but also loss of strength of concrete due to the loss of cohesion in the hydrated cement paste and of adhesion between it and the aggregate particles. Concrete attacked by sulfates has a characteristic whitish appearance. The damage usually starts at

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edges and corners and is followed by progressive cracking and spalling which reduce the concrete to a friable or even soft state.

The attack occurs only when the concentration of the sulfates exceeds a certain threshold. Above that, the rate of sulfate attack increases with an increase in the strength of the solution, but beyond a concentration of about 0.5 per cent of $MgSO_4$ or 1 per cent of Na_2SO_4 the rate of increase in the intensity of the attack becomes smaller.[10.7] A saturated solution of $MgSO_4$ leads to serious deterioration of concrete, although with a low water/cement ratio this takes place only after 2 to 3 years.[10.13] BS EN 206-1 : 2000 expresses sulfate as SO_3 while ACI uses SO_4; multiplying the former by 1.2 converts it into the latter. Water-soluble sulfates, and not acid-soluble, are considered The classification of the severity of exposure recommended by ACI 318-08[10.42] and BS EN 206-1 : 2000 is given in Table 10.7. Because extraction of sulfates from soil depends on the compaction of the soil, and on the water-

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soil extraction ratio, the measurement of sulfates in groundwater is more reliable. The class boundaries are, in a sense, arbitrary because they have not been calibrated by measurement of recorded incidence of damage to concrete caused by sulfate attack. Moreover, the actual conditions of exposure may vary during the lifetime of a structure owing to a variation in groundwater flow or the drainage pattern.

Table 10.7. Classification of Severity of Sulfate Environment

Exposure	*According to ACI 318-08*		*According to BS EN 206-1 : 2000*	
	Concentration of water-soluble sulfates expressed as SO_4		*Concentration of water-soluble sulfates expressed as SO_3*	
	In soil per cent	*In water ppm*	*In water ppm*	
Mild	<0.1	<150	moderately	
Moderate	0.1 to 0.2	150 to 1500	aggressive	600 to 3000
Severe	0.2 to 2.0	1500 to 10 000	highly	
Very severe	>2.0	>10 000	aggressive	3000 to 6000

The classification of the severity of exposure recommended by ACI 201.2R-92[10.42] is given in Table 10.7. The approach of BS 8110-1 : 1985

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(superseded by Eurocode 2 : 2008) is somewhat more elaborate in that there are more subdivisions corresponding to the "severe" exposure condition of ACI 201.2R-92.

It should be noted that, under certain conditions, the sulfate concentration in water can be greatly increased by evaporation. This is the case with sea water splash on horizontal surfaces and on the surface of cooling towers.[10.79]

In addition to the concentration of the sulfate, the speed with which concrete is attacked depends also on the rate at which the sulfate removed by the reaction with cement can be replenished. Thus, in estimating the danger of sulfate attack, the movement of groundwater has to be known. When concrete is exposed to the pressure of sulfate-bearing water on one side, the rate of attack will be highest. Likewise, alternating saturation and drying leads to rapid deterioration. On the other hand, when the concrete is completely buried, without a channel for the groundwater, conditions will be much less severe.

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Factors mitigating the attack

The purpose of the classification of the severity of sulfate exposure shown in Table 10.7 is to suggest preventive measures. Two approaches can be used. The first one is to minimize the C_3A content in the cement, that is, to use sulfate-resisting cement; this is discussed on p. 76. The second approach is to reduce the quantity of $Ca(OH)_2$ in hydrated cement paste by the use of blended cements containing blastfurnace slag or pozzolana. The effect of pozzolana is two-fold. First, it reacts with $Ca(OH)_2$ so that $Ca(OH)_2$ is no longer available for reaction with sulfates. Second, compared with Portland cement only, the same content of blended cement per cubic metre of concrete results in less $Ca(OH)_2$. These measures are helpful, but even more important is the prevention of ingress of sulfates into the concrete: this is achieved by making the concrete as dense as possible and with as low a permeability as possible. This must never be forgotten: for instance, the use of lean concrete in the haunching or bedding of sewers

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produces vulnerable parts of a possibly otherwise durable construction.

As far as the choice of cement is concerned, ACI 201.2R-92[10.42] recommends, for moderate exposure, the use of Type II cement, or of blended cement with blastfurnace slag or pozzolana. For severe exposure, sulfate-resisting cement is the preferred choice; for very severe exposure, a blend of sulfate-resisting cement and pozzolana (between 25 and 40 per cent by mass of total cementitious material) or blastfurnace slag (not less than 70 per cent by mass) *proven* to improve sulfate resistance, is required.[10.135] The relevant property of the blast-furnace slag is its alumina content;[10.80] advice on this is given in ASTM C 989-09a and in ref. 10.135. It should also be noted that not all pozzolanas are beneficial: a low calcium oxide content is desirable;[10.77] specifically, Class C fly ash decreases the sulfate resistance of concrete.[10.76]

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The reason why sulfate-resisting cement alone is inadequate under severe conditions is that not only calcium sulfate but also other sulfates are present. Therefore, although sulfate-resisting cement does not contain enough C_3A for the formation of expansive ettringite, the $Ca(OH)_2$ present and possibly also C-S-H are vulnerable to the acid-type attack of the sulfates.

The recommendations of ACI 201.2R-92[10.42] reflect the beneficial effect on sulfate resistance of pozzolanas and ground granulated blastfurnace slag used with Portland cement. Pozzolanas have also to be used with regulated-set cement, which alone shows a poor resistance to sulfates. However, partial replacement (20 per cent) of this cement by pozzolanas reduces the early strength of concrete[10.24] so that the practicality of use of regulated-set cement under conditions of sulfate attack is questionable.

Silica fume incorporated in concrete is beneficial with respect to permeability, but tests on hardened cement *paste* indicate that the effect of

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silica fume in various sulfate environments is not clear.[10.126]

Supersulfated cement offers very high resistance to sulfates, especially if its Portland cement component is of the sulfate-resisting variety.

High-pressure steam curing improves the resistance of concrete to sulfate attack. This applies to concretes made both with sulfate-resisting and ordinary Portland cements because the improvement is due to the change of C_3AH_6 into a less reactive phase, and also to the removal of $Ca(OH)_2$ by the reaction with silica.

It is worth noting that, because of changes in solubility with temperature, expansion due to the formation of ettringite is very low at temperatures above 30 °C (86 °F).[10.127]

Low permeability of concrete, as mentioned earlier in this chapter, is the consequence of an appropriate microstructure of the hardened cement paste. In order to achieve this, the mix proportions need to be specified. There are three pos-

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sible approaches, one or more of these being used by various codes: specifying a maximum water/cement ratio, specifying a minimum strength, and specifying a minimum cement content. The same choice applies when a low permeability of concrete is sought in cases of protection from other forms of attack.

The concept of ensuring protection from sulfate attack by specifying a minimum cement content has no scientific basis. As Mather[10.25] points out, for instance, with 356 kg of ordinary Portland cement per cubic metre of concrete (600 lb/yd^3) it is possible to obtain concretes ranging in cylinder strength from 14 MPa (2000 psi) to 41 MPa (6000 psi) depending on the water/cement ratio and on slump. The durability of these concretes will clearly vary enormously.

The use of strength for specifying purposes is convenient but strength only reflects the water/cement ratio; it is this that is relevant to density and permeability, as discussed on p. 272. However, specifying the water/cement ratio re-

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gardless of the nature of the cement used is inadequate: reference to the influence of the various blended cements on sulfate resistance was made earlier in this section.

Tests on sulfate resistance

The resistance of concrete to sulfate attack can be tested in the laboratory by storing specimens in a solution of sodium or magnesium sulfate, or in a mixture of the two. Alternate wetting and drying accelerates the damage due to the crystallization of salts in the pores of the concrete. The effects of exposure can be estimated by the loss in strength of the specimen, by changes in its dynamic modulus of elasticity, by its expansion, by its loss of mass, or can even be assessed visually.

Figure 10.9 shows the change in the dynamic modulus of 1:3 mortar immersed (after 78 days' moist curing) in a 5 per cent solution of different sulfates.[10.9] The test method of ASTM C 1012-09 uses immersion of well-hydrated mortar in a sulfate solution, and considers excessive expan-

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sion as a criterion of failure under sulfate attack. This test can be used to assess the effects of using various cementitious materials in the mix. As, however, it is mortar and not concrete that is tested, some physical effects of materials such as silica fume or fillers are not reflected in the test. A further drawback of the test is that it is slow, several months sometimes being required before failure is recorded or its absence can be inferred.

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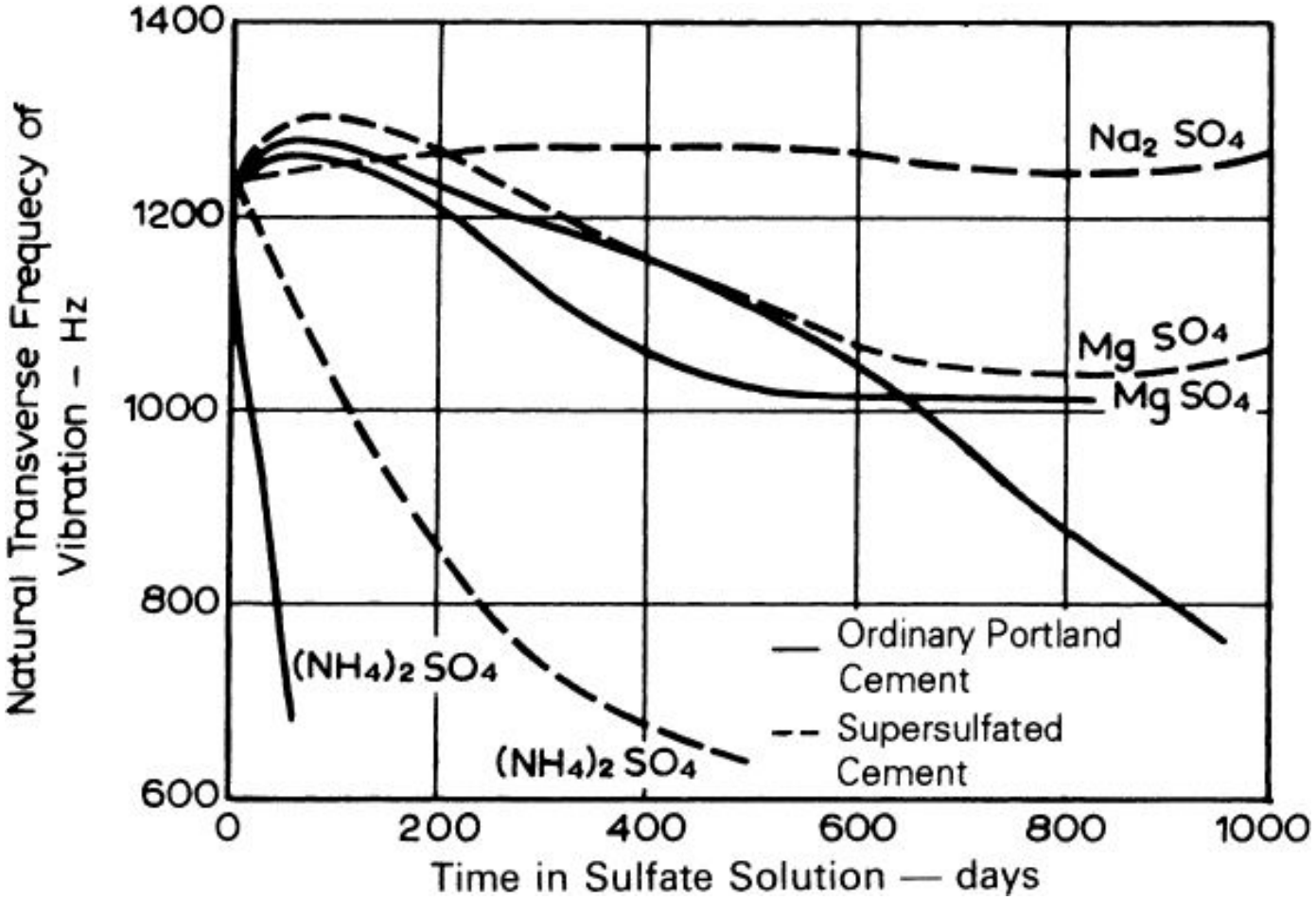


Fig. 10.9. Effect of immersion in a 5 per cent sulfate solution on the dynamic modulus of elasticity of 1:3 mortars made with ordinary Portland and supersulfated cements[10.9]

As an alternative to immersion in a sulfate solution, ASTM C 452-06 prescribes a method in which a certain amount of gypsum is included in the original mortar mix. This speeds up the reaction with C_3A but the method is not appropriate for use with blended cements, in which some ce-

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mentitious material is still unhydrated at the stage of coming into contact with the sulfates. This is so because, in the test of ASTM C 452-06, the criterion of sulfate resistance is the expansion at the age of 14 days.

It may be relevant to mention one more test, namely, ASTM C 1038-04, which determines the expansion of mortar made from Portland cement of which sulfate is an integral part. Thus, the test identifies excessive sulfate content of a Portland cement rather than attack by external sulfates.

All the ASTM tests are performed on mortars of prescribed proportions and, in consequence, are more sensitive to the chemical resistance of cement than to the physical structure of the hardened cement paste in the actual concrete.

Efflorescence

Leaching of lime compounds, mentioned earlier, may under some circumstances lead to the formation of salt deposits on the surface of the concrete, known as efflorescence. This is found, for

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instance, when water percolates through poorly compacted concrete or through cracks or along badly made joints, and when evaporation can take place at the surface of the concrete. Calcium carbonate formed by the reaction of $Ca(OH)_2$ with CO_2 is left behind in the form of a white deposit. Calcium sulfate deposits are encountered as well.

Efflorescence is more likely to occur in concrete which is porous near the surface. Thus, the type of formwork may play a role in addition to the degree of compaction and to the water/cement ratio.[10.28] The occurrence of efflorescence is greater when cool, wet weather is followed by a dry and hot spell; in this sequence, there is little initial carbonation, lime is dissolved by the surface moisture, and $Ca(OH)_2$ is finally drawn to the surface.[10.28]

Efflorescence can also be caused by the use of unwashed seashore aggregate. The salt coating on the surface of the aggregate particles may, in due course, lead to a white deposit on the surface of the concrete. Gypsum and alkalis in the aggregate

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have a similar effect. Transport of salts from the ground through porous concrete to a drying surface can also result in efflorescence.

Apart from the leaching aspect, efflorescence is of importance only in so far as it mars the appearance of concrete.

Early efflorescence can be removed with a brush and water. Heavy deposits may require acid treatment of the surface of the concrete. Such treatment can also be used to remove laitance on architectural concrete and to restore the roughness of floor surfaces.[10.29] The acid used is HCl diluted from its concentrated form in a ratio of 1:20 or 1:10. Typically, the thickness of the layer of acid (applied by a sponge) would be 0.5 mm (0.02 in.), and the quantity of a 1:10 solution of the acid used would be 200 g/m^2, and the depth of concrete removed about 0.01 mm (0.0004 in.). The action of the acid stops when it has been used up by the reaction with lime, but the concrete should be washed in order to remove the salts which have been formed.[10.29]

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Because lime is removed by the acid, the surface of the concrete becomes darker. For this reason, as well as to avoid local 'excesses', the acid must be applied uniformly in terms of concentration, quantity, and duration of action. Acid treatment is a very delicate operation and trying it out on concrete samples is essential.

Another surface blemish is the appearance of dark stains of irregular shape which are visible depending on the direction of light. Their origin is totally different from efflorescence: they are compacts of cement paste, almost without pores. These can be caused by the aggregation of coarse particles of cement which have hydrated only little in locations where the water/cement ratio is very low. It is the lack of hydration and of production of lime that leads to the dark colour. Such a segregation of coarse particles of cement can be caused by a filtering action of leaky formwork or of aggregate particles. With time, hydration may take place and the dark colour may disappear.[10.30]

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Effects of sea water on concrete

Concrete exposed to sea water can be subjected to various chemical and physical actions. These include chemical attack, chloride-induced corrosion of steel reinforcement, freeze–thaw attack, salt weathering, and abrasion by sand in suspension and by ice. The presence and intensity of these various forms of attack depend on the location of the concrete with respect to the sea level. These forms of attack will be considered later in this chapter and in Chapter 11, starting with chemical attack, which is the subject matter of this section.

Chemical action of sea water on concrete arises from the fact that sea water contains a number of dissolved salts. The total salinity is typically 3.5 per cent. Specific values are: 0.7 per cent in the Baltic Sea, 3.3 per cent in the North Sea, 3.6 per cent in the Atlantic and Indian Oceans, 3.9 per cent in the Mediterranean Sea, 4.0 per cent in the Red Sea, and 4.3 per cent in the Persian–Arabian Gulf. In all the seas, the ratio of

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the individual salts is very nearly constant; for example, in the Atlantic Ocean, the ion concentration (in per cent) is as follows: chloride 2.00, sulfate 0.28, sodium 1.11, magnesium 0.14, calcium 0.05, and potassium 0.04. Sea water contains also some dissolved CO_2. Shallow coastal areas in hot climates, where evaporation is high, can be very salty. The Dead Sea is the extreme case: its salinity is 31.5 per cent, that is, nearly 9 times that of the oceans, but the sulfate concentration is lower than in the oceans.[10.91]

The pH of sea water varies between 7.5 and 8.4, the average value in equilibrium with atmospheric CO_2 being 8.2.[10.79] Ingress of sea water into concrete *per se* does not significantly lower the pH of pore water in the hardened cement paste: the lowest value reported is 12.0.[10.86]

The presence of a large quantity of sulfates in sea water could lead to the expectation of sulfate attack. Indeed, the reaction between sulfate ions and both C_3A and C-S-H takes place, resulting

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in the formation of ettringite, but this is not associated with deleterious expansion because ettringite, as well as gypsum, are soluble in the presence of chlorides and can be leached out by the sea water.[10.7] It follows that the use of sulfate-resisting cement in concrete exposed to the sea is not essential, but a limit on C_3A of 8 per cent when the SO_3 content is less than 3 per cent, is recommended; cements with a C_3A content up to 10 per cent can be used, provided the SO_3 content does not exceed 2.5 per cent.[10.90] It seems that it is the excess of SO_3 that leads to a delayed expansion of concrete. The same tests[10.90] confirmed that C_4AF also leads to the formation of ettringite so that the requirement of the earlier version of ASTM C 150-09 to the effect that the content of $2C_3A + C_4AF$ be less than 25 per cent of the clinker for sulfate-resisting cement should be observed.

The preceding comments and requirements apply to concrete permanently immersed in water, which represents relatively protected expos-

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ure conditions[10.88] because a steady state of saturation and salt concentration is reached so that the diffusion of ions is greatly reduced. Alternating wetting and drying represents much more severe conditions because a build-up of salts within the concrete can occur in consequence of the ingress of sea water, followed by evaporation of pure water, with the salts left behind. As the most damaging effect of sea water on concrete structures arises from the action of chlorides on the steel reinforcement, the build-up of salts will be discussed in Chapter 11 in the section on chloride attack.

The chemical action of sea water on concrete is as follows. The magnesium ion present in the sea water substitutes for the calcium ion:

$$MgSO_4 + Ca(OH)_2 \rightarrow CaSO_4 + Mg(OH)_2.$$

The resulting $Mg(OH)_2$, known as brucite, precipitates in the pores at the surface of the con-

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crete, thus forming a protective surface layer which impedes further reaction. Some precipitated $CaCO_3$ in the form of aragonite, arising from the reaction of $Ca(OH)_2$ with CO_2, may also be present. The precipitated deposits, typically 20 to 50 μm thick, form rapidly;[10.84] they have been observed in a number of fully submerged sea structures. The blocking nature of brucite makes its formation self-limiting. However, if abrasion can remove the surface deposit, then the reaction by the magnesium ion freely available in the sea water continues.

This situation is an example of the synergistic action of the different modes of attack by the sea: wave action enhances chemical attack, and chemical attack by way of formation and crystallization of salts makes the concrete more vulnerable to erosion by wave action and to abrasion by sand suspended in sea water.

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Salt weathering

When concrete is repeatedly wetted by sea water, with alternating periods of drying during which pure water evaporates, some of the salts dissolved in sea water are left behind in the form of crystals, mainly sulfates. These crystals re-hydrate and grow upon subsequent wetting, and thereby exert an expansive force on the surrounding hardened cement paste. Such progressive surface weathering, known as salt weathering, occurs in particular when the temperature is high and insolation is strong so that drying occurs rapidly in the pores over some depth from the surface. Thus, intermittently wetted surfaces are vulnerable; these are surfaces of the concrete in the tidal zone and in the splash zone. Horizontal or inclined surfaces are particularly prone to salt weathering, and so are surfaces wetted repeatedly but not at short intervals so that thorough drying can take place. Salt water can also rise by sorption, that is, by capillary action; evaporation of pure water from the

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surface leaves behind salt crystals which, when re-wetted, can cause disruption.

Salt weathering can occur not only in consequence of direct spray by sea water, but also when air-borne salt which has been deposited on the surface of the concrete becomes dissolved by dew, and this is followed by evaporation. Such behaviour has been observed in desert areas where the large temperature drop in the small hours of the night reduces the relative humidity of the air to the point where condensation in the form of dew occurs. Salt weathering can extend to a depth of several millimetres: hardened cement paste and the embedded fine aggregate particles are removed, leaving behind protruding coarse aggregate particles. With time, these particles can become loosened, thereby exposing more hardened cement paste which, in turn, becomes liable to salt weathering. The process is, in essence, similar to the salt weathering of porous rocks. Even when sodium sulfate is involved, the

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damage mechanism is physical so that it does not represent sulfate attack.

It should be added that, unless the aggregate is dense and has very low absorption, the aggregate itself is liable to damage. Clearly, such aggregate should not be used in concrete exposed to conditions conducive to salt weathering so that the choice of suitable aggregate is of considerable importance.[10.85] Because the attack of concrete by salt weathering is physical in nature, the type of cement used is of little importance *per se* but, to ensure low permeability of the surface zone of concrete, the choice of the concrete mix is critical.

Salt weathering can also result from the use of de-icing salts on concrete surfaces in cold climates. This is known as *salt scaling*; this topic is discussed in Chapter 11.

A peculiar form of marine attack of concrete in very warm sea water has been cited by Bijen.[10.129] When limestone aggregate is present,

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a genus of oysters and also a genus of sponges devour lime and produce holes up to 10 mm in diameter and 150 mm deep. The rate of attack is up to 10 mm per annum.

Selection of concrete for exposure to sea water

The preceding discussion of the various modes of attack by sea water has emphasized the importance of the low permeability of the exposed concrete. This can be achieved by the use of a low water/cement ratio, an appropriate choice of cementitious materials, good compaction, and absence of cracking due to shrinkage, thermal effects, or stresses in service. It is important for the concrete to be well cured prior to exposure to the sea water. The assumption that sea water also provides curing is erroneus (see p. 574) unless the concrete, once immersed in sea water, remains permanently submerged. Tests on mortar have led to a recommendation of a minimum period of seven days of curing in fresh water, regardless of the type of cement used.[10.89]

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Reference to the choice of cement was made on p. 517 in so far as fully submerged concrete is concerned. For other conditions of exposure, the danger of ingress of chlorides influences the choice of cement, and therefore this topic is discussed in the section dealing with chloride attack, in Chapter 11.

Disruption by alkali–silica reaction

In Chapter 3, the reactions between the alkalis and reactive silica and some carbonates in aggregate were discussed. The consequences of the alkali–silica reaction and the means of avoiding these consequences will now be considered.

The reaction can be disruptive and manifest itself as cracking. The crack width can range from 0.1 mm to as much as 10 mm in extreme cases. The cracks are rarely more than 25 mm, or at most 50 mm, deep.[10.136] Hence, in most cases, the alkali–silica reaction adversely affects the appearance and serviceability of a structure, rather than its integrity; in particular, the compressive

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strength of concrete in the direction of the applied stress is not greatly affected.[10.115] Nevertheless, cracking can facilitate the ingress of harmful agents.

The pattern of surface cracking induced by the alkali–silica reaction is irregular, somewhat reminiscent of a huge spider's web. However, the pattern is not necessarily distinguishable from that caused by sulfate attack or by freezing and thawing, or even by severe plastic shrinkage. In order to ascertain whether any observed cracking is due to the alkali–silica reaction, a procedure recommended by a working party of the British Cement Association[10.112] can be followed. Within the concrete, many of the cracks caused by the reaction can be seen to pass through individual aggregate particles but also through the surrounding hydrated cement paste.

If the sole source of alkalis in concrete is Portland cement, then limiting the alkali content in the cement would prevent the occurrence of deleterious reactions. The minimum alkali content

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of cement at which expansive reaction can take place is 0.6 per cent of the soda equivalent. This is calculated from stoichiometry as the actual Na_2O content plus 0.658 times the K_2O content of the clinker. This method of calculation of the alkali content, which does not distinguish between sodium and potassium, is convenient but simplistic. Chatterji[10.119] found that potassium ions are transported towards the silica faster than the sodium ions, and are therefore, on a mass for mass basis, potentially more harmful.

The equivalent soda limit of 0.6 per cent by mass of cement lies at the origin of *low-alkali cement* (see p. 48) and, indeed, defines it. It can be noted, nevertheless, that, in exceptional cases, cements with an even lower alkali content have been known to cause expansion.[10.1]

The background to the low-alkali cement, offered by Hobbs,[10.128] may be of interest. The alkali–silica reaction takes place only at high concentrations of OH^-, that is at high values of

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pH in the pore water. Now, the pH of the pore water depends on the alkali content of the cement. Specifically, a high-alkali cement leads to a pH of between 13.5 and 13.9 while a low-alkali cement results in a pH of 12.7 to 13.1.[10.128] Given that an increase in pH of 1.0 represents a ten-fold increase in hydroxyl ion concentration, the hydroxyl ion concentration with a low-alkali cement is about 10 times lower than when a high-alkali cement is used. This is the rationale of using low-alkali cement with potentially reactive aggregate.

The assumption of prevention of a deleterious alkali–silica reaction by limiting the alkali content in cement is valid only when two conditions are satisfied: there is no other source of alkalis in the concrete; and the alkalis do not become concentrated in some locations, at the expense of others. Such concentration may be caused by moisture gradients or by alternating wetting and drying.[10.118] This may be an appropriate place to mention that the alkalis may also become con-

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centrated by an electric current passed through the concrete; this may occur when cathodic protection is used to prevent corrosion of embedded steel.[10.114]

The additional sources of alkalis in concrete include sodium chloride present in unwashed sand dredged from the sea or obtained from the desert. The use of such sand in reinforced concrete should not be allowed because chlorides are conducive to corrosion of steel (see Chapter 11). Other internal sources of alkalis are some admixtures, especially superplasticizers, or even the mix water. The alkalis from these sources, and also from fly ash and ground granulated blastfurnace slag, should be included in the calculation of the amount of alkalis present, but only taking a proportion of the actual amount of alkalis in these cementitious materials. There is no agreement on how much this proportion should be, but BS 5328 : 4 : 1990 (replaced by BS EN 206-1-2000) uses 17 per cent for fly ash and 50 per cent for ground granulated blastfurnace slag.

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Because of the varied provenance of alkalis, it is logical to limit the total content of alkalis in concrete. British Standard BS 5328-1 : 1991 (withdrawn) specifies a maximum of 3.0 kg of alkalis (expressed as soda equivalent) which can be present in 1 m^3 of concrete containing alkali-reactive aggregate. This amount of reactive alkalis is determined by a British method which is different from that prescribed by British Standard BS EN 196-21 : 1992 (withdrawn); the latter method gives a value for the alkali content about 0.025 percentage point higher than the British method. Thus, when compliance with BS 5328-1 : 1997 (withdrawn) is required, it is important to exercise care in the choice of the test method for the determination of the alkali content of cement.

It is useful to reiterate the fact that there are three necessary conditions for alkali-silica reaction to proceed: alkalis, reactive silica and adequate moisture. The latter is taken usually as not less than 85 per cent relative humidity (which is common outdoors in the UK at night or in

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winter and which may be present in the interior of concrete members due to residual mixing water). Alkalis are always present in Portland cement, but there exists low-alkali Portland cement (see p. 48). In addition, BRE Digest 330.3 recognizes also moderate alkali and high alkali cements. Alkalis are sometimes also present in admixtures. The British Standards of relevance are BS EN 206-1 : 2000 and BS 8500-2 : 2002.

From the above, it follows that aggregates with a low alkali reactivity should be used, and in the UK they are usually available. However, NaCl is a source of alkalis, and residual NaCl may be present in poorly washed sea-dredged sand and also in de-icing salts.

In practice, it is not easy to eliminate totally the alkali-silica reaction, but we should aim to minimize that reaction. That topic is discussed in ref. 10.141. However, assistance in minimizing the alkali-silica reaction is obtained by incorporating ggbs or fly ash with a minimum of 25 percent of the mass of cement. There is a paradox

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here because both ggbs and fly ash contain alkalis but these are in glass form.

Preventive measures

The discussion of the alkali–silica reaction, presented in Chapter 3, makes it clear that the progress and consequences of the reaction are influenced by the proportions of various ions in the pore water and by the availability of the alkalis and of silica. In particular, the expansion caused by the alkali–silica reaction is greater the greater the content of reactive silica, but only up to a certain content of silica; at higher contents, the expansion is smaller. This is illustrated in Fig. 10.10.[10.6] There is thus a pessimum content of silica. This pessimum content is higher at lower water/cement ratios and at higher cement contents.[10.128] The ratio of reactive silica to the alkalis corresponding to the maximum expansion usually lies in the range of 3.5 to 5.5.[10.128]

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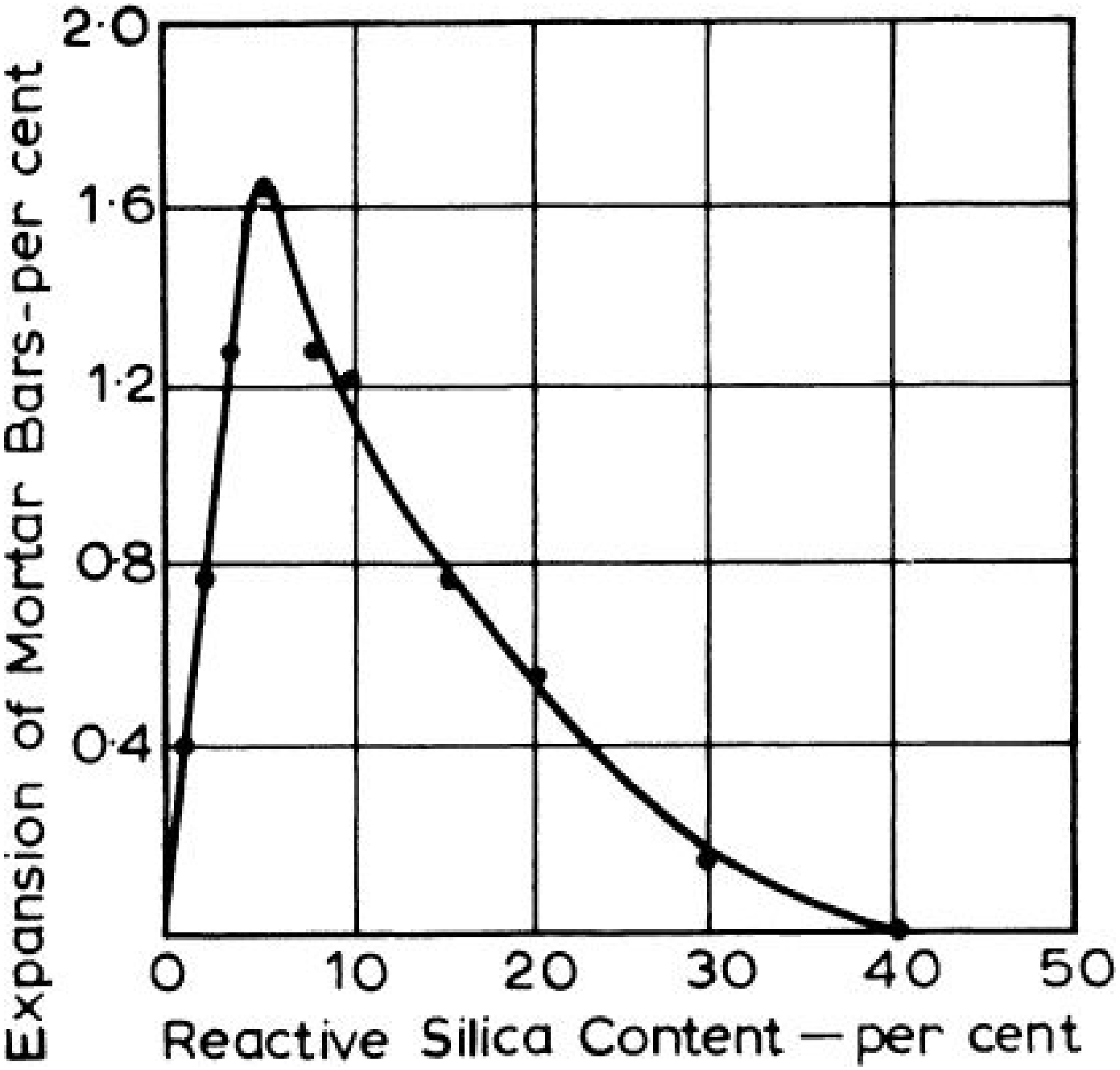


Fig. 10.10. Relation between expansion after 224 days and reactive silica content in the aggregate[10.6]

It follows from the above that varying the silica content in concrete can move the silica/al-

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kali ratio away from the pessimum. Specifically, it has been found that expansion due to the alkali–silica reaction can be reduced or eliminated by the addition to the mix of reactive silica in a finely powdered form. This apparent paradox can be explained by reference to Fig. 10.10, showing the relation between the expansion of a mortar bar and the content of reactive silica of size between 850 and 300 μm (No. 20 and No. 50 ASTM) sieves, i.e. not in a powdered form.[10.6] In the range of low silica contents, the greater quantity of silica for a given amount of alkalis increases expansion but, with higher values of silica content, the situation is reversed: the greater the surface area of the reactive aggregate the lower the quantity of alkalis available per unit of this area, and the less alkali–silica gel can be formed.[10.6] On the other hand, due to the extremely low mobility of calcium hydroxide, only that adjacent to the surface of the aggregate is available for reaction, so that the quantity of calcium hydroxide per unit area of aggregate is in-

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dependent of the magnitude of the total surface area of the aggregate. Thus, increasing the surface area increases the calcium hydroxide/alkali ratio of the solution at the boundary of the aggregate. Under such circumstances, an innocuous (non-expanding) calcium alkali silicate product is formed.[10.8]

By a similar argument, finely divided siliceous material added to the coarse reactive particles already present would reduce expansion, although the reaction with the alkalis still takes place. These pozzolanic additions, such as crushed pyrex glass or fly ash, have indeed been found effective in reducing the penetration of the coarser aggregate particles. The fly ash should contain no more than 2 or 3 per cent by mass of alkalis.[10.136] However, Class F fly ash, used in a quantity representing 58 per cent (by mass) of the total cementitious material, was found to be highly effective in preventing expansion, even when the total alkali content was 5 kg per cubic metre of concrete.[10.117] It is important that the fly

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ash be fine; if necessary, grinding can be used to improve its effectiveness in reducing the expansion.

Pozzolanas in the mix are beneficial also because they reduce the permeability of concrete (see Chapter 13) and therefore reduce the mobility of aggressive agents, both those present within the concrete and those which may ingress. Furthermore, C-S-H formed by pozzolanic activity incorporates a certain amount of alkalis and thus lowers the value of pH;[10.136] the influence of pH upon the alkali–silica reaction was discussed earlier in this section.

Silica fume is particularly effective because the silica reacts preferentially with the alkalis. Although the product of reaction is the same as that between the alkalis and the reactive silica in the aggregate, the reaction takes place at the very large surface of the fine particles of silica fume (see p. 87). In consequence, the reaction does not result in expansion.[10.116]

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Ground granulated blastfurnace slag is also effective in mitigating or preventing the deleterious effects of the alkali–silica reaction. It should be noted that the presence of ground granulated blastfurnace slag results in a reduced permeability of concrete (see Chapter 13). There is evidence that, when Portland blastfurnace slag cement is used, a maximum alkali content of 0.9 per cent is harmless when the slag content of the cement is not less than 50 per cent.[10.99] An even higher alkali content, 1.1 per cent, is considered to be tolerable by BS 5328-1 : 1991, replaced by BS EN 206-1 : 2000. There is anecdotal evidence of the beneficial effects of ground granulated blastfurnace slag in so far as the deleterious alkali–silica expansion is concerned. In the Netherlands, deleterious expansion in a number of structures was observed but it was absent where Portland blastfurnace cement had been used.[10.122]

To be effective, the various cementitious materials must be present in adequate proportions of the total cementitious material. Expressed by

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mass, these proportions are as follows: Class F fly ash–at least 30 or 40 per cent; silica fume–at least 20 per cent; ground granulated blastfurnace slag–50 to 65 per cent.[10.120,10.136] Inadequate amounts can actually aggravate the situation and increase expansion if a particularly bad silica–alkali ratio is reached (cf. Fig. 10.10). The performance of any pozzolana or ground granulated blastfurnace slag in preventing excessive expansion due to alkali–silica reaction should be tested according to ASTM C 441-05. Advice contained in an appendix to Canadian Standard A23.1-94 is very useful.[10.111]

Inclusion in the concrete of silica fume or fly ash will not be effective in preventing expansion if alkalis can continue to ingress into the concrete.[10.113,10.119] More generally, in considering the alkali content in concrete, it should be noted that water-borne alkalis can ingress from outside in some structures, for example, from other, adjacent building materials or from sodium chloride used as a de-icing agent.

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Some tests indicate that lithium salts may inhibit expansive reactions but the relevant mechanism has not been established.[10.121]

It should be noted that, although silica gel resulting from the alkali-silica reaction can be formed inside air bubbles, it does not follow that air entrainment represents a means of avoiding the deleterious effects of the reaction.

Abrasion of concrete

Under many circumstances, concrete surfaces are subjected to wear. This may be due to attrition by sliding, scraping or percussion.[10.14] In the case of hydraulic structures, the action of the abrasive materials carried by water leads to erosion. Another cause of damage to concrete in flowing water is cavitation.

Tests for abrasion resistance

Resistance of concrete to abrasion is difficult to assess because the damaging action varies depending on the exact cause of wear, and no one

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test procedure is satisfactory in evaluating all the conditions: rubbing test, including rolling balls, dressing wheel, or sandblasting may each be appropriate in different cases.

ASTM C 418-05 prescribes the procedure for determining wear by sandblasting; the loss of volume of concrete serves as a basis for judgement, but not as a criterion of wear resistance under different conditions. ASTM C 779-05 prescribes three test procedures for laboratory or field use. In the *revolving disc test*, there is applied a revolving motion of three flat surfaces driven along a circular path at 0.2 Hz and individually turning on their axes at 4.6 Hz. Silicon carbide is fed as an abrasive material. In the *steel ball abrasion test*, a load is applied to a rotating head which is separated from the specimen by steel balls. The test is performed in circulating water in order to remove the eroded material. The *dressing wheel test* uses a drill press modified to apply a load to three sets of seven rotating dressing wheels which are in contact with the speci-

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men. The driving head is rotated for 30 minutes at 0.92 Hz. In all cases, the depth of wear of the specimen is used as a measure of abrasion.

When it is desired to perform abrasion tests on cores (which are too small for the tests of ASTM C 418-05 and C 779-05) ASTM C 944-99 (2005) can be used. Here, two dressing wheels in a drill press under a fixed load are applied to the surface of the core, and the loss of mass is determined; the depth of wear can also be measured.

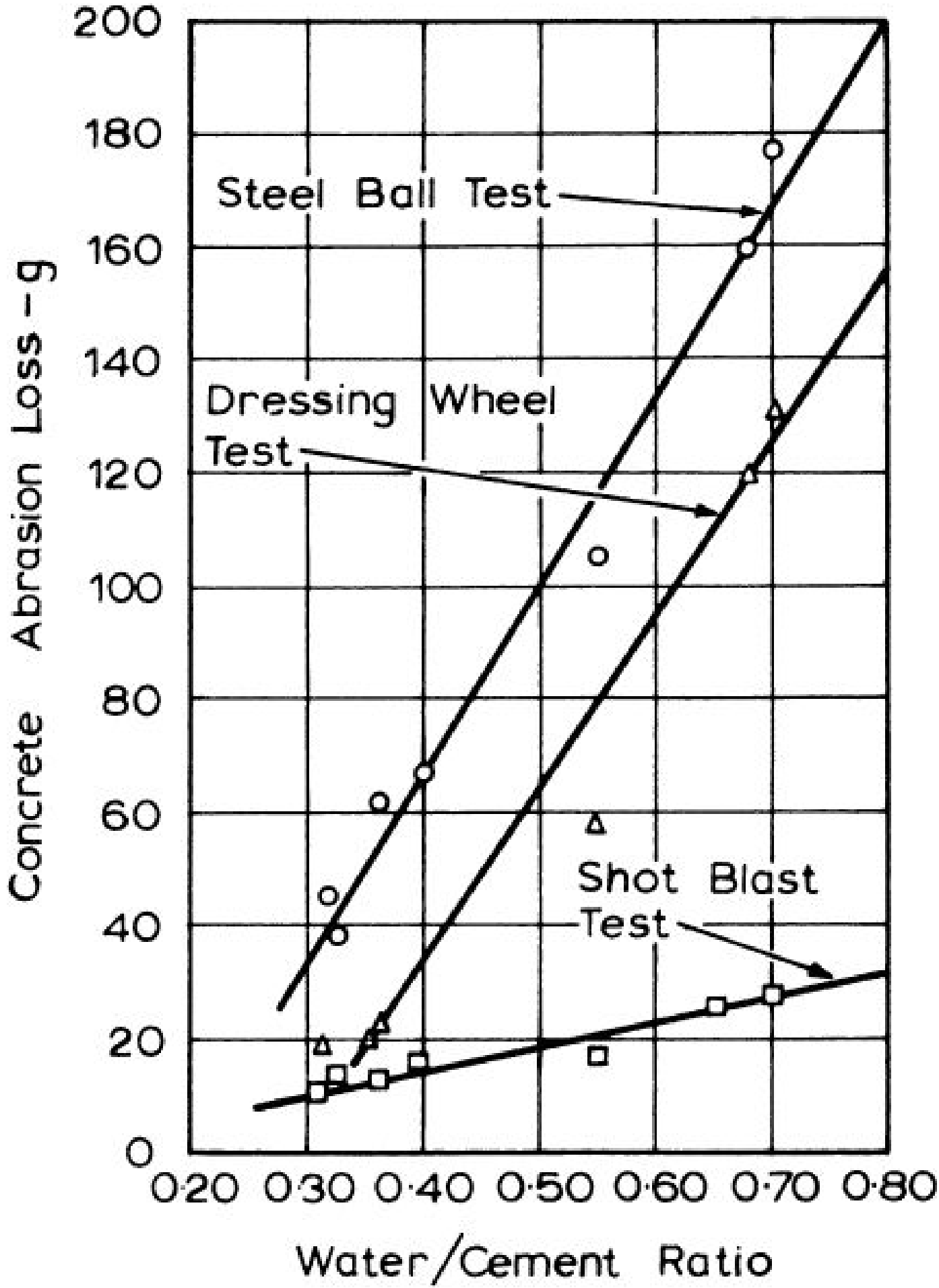
The various tests try to simulate the modes of abrasion found in practice, but this is not easy and, indeed, the main difficulty in abrasion testing is to make sure that the results of a test represent the comparative resistance of concrete to a given type of wear. The tests prescribed by ASTM 779-05 are useful in estimating the resistance of concrete to heavy foot and wheeled traffic and to tyre chains and tracked vehicles. Broadly speaking, the heavier the abrasion in service the more helpful the test in increasing order of use-

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fulness: revolving disc, dressing wheel, and steel ball.[10.32]

Figure 10.11 shows the results of the three tests of ASTM 779-05 on different concretes. Because of the arbitrary conditions of test, the values obtained are not comparable quantitatively, but in all cases the resistance to abrasion was found to be proportional to the compressive strength of concrete.[10.20] The steel ball test appears more consistent and more sensitive than the other tests.

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Fig. 10.11. Influence of the water/cement ratio of the mix on the abrasion loss of concrete for different tests[10.20]

The resistance of concrete to abrasion by water-borne solids can be determined using ASTM C 1138-05. In this test, the behaviour of swirling water containing suspended particles is simulated by high-speed movement of steel grinding balls of various sizes in a water tank over a period of 72 hours. The depth of wear of the concrete surface gives a comparative measurement.

In a totally different category of assessing the abrasion resistance of concrete is the rebound hammer test (see p. 626): the value obtained is sensitive to some of the factors which influence the abrasion resistance of concrete.[10.37]

Factors influencing abrasion resistance

Abrasion seems to involve high-intensity stress applied locally so that the strength and hardness

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of the surface zone of concrete strongly influence the resistance to abrasion. In consequence, the compressive strength of concrete is the principal factor controlling the resistance to abrasion. The minimum strength required depends on the severity of abrasion expected. Very high strength concretes exhibit a high resistance to abrasion: for example, increasing the compressive strength from 50 MPa (7000 psi) to 100 MPa (14 000 psi) increases the abrasion resistance by 50 per cent, and 150 MPa (21 000 psi) concrete is as resistant as high-quality granite.[10.40]

The properties of the concrete in the surface zone are strongly affected by the finishing operations, which may reduce the water/cement ratio and improve compaction. Vacuum dewatering is beneficial (see p. 234). The presence of laitance must be avoided. Particularly good curing is of importance; it is desirable to have a period of curing twice as long as normal in order to achieve good resistance to abrasion.

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Rich mixes are undesirable, a cement content of 350 kg/m^3 (600 lb/yd^3) being probably a maximum because coarse aggregate should be present just below the surface of the concrete.

As far as the aggregate is concerned, inclusion of some crushed sand is desirable,[10.40] and so is the use of strong[10.38] and hard aggregate; however, the abrasion resistance of aggregate, as determined by the Los Angeles test (see p. 124) does not seem to be a good indicator of the abrasion resistance of concrete made with a given aggregate.[10.39] Lightweight aggregate of high quality has good abrasion resistance because it is inherently a ceramic material but, due to its porous structure, it is not resistant to impact which may be associated with abrasion.[10.87]

Shrinkage-compensating concrete has a significantly increased abrasion resistance[10.94] probably because of the absence of fine cracks which would encourage the progress of abrasion.

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Consideration of the use of hardeners incorporated in the surface zone of concrete is outside the scope of the present book.

Erosion resistance

Erosion of concrete is an important type of wear which may occur in concrete in contact with flowing water. It is convenient to distinguish between erosion due to solid particles carried by the water and damage due to pitting resulting from cavities forming and collapsing in water flowing at high velocities. The latter is considered in the next section.

The rate of erosion depends on the quantity, shape, size, and hardness of the particles being transported, on the velocity of their movement, on the presence of eddies, and also on the quality of concrete.[10.41] As in the case of abrasion in general, this quality appears to be best measured by the compressive strength of concrete but the mix composition is also relevant. In particular, concrete with large aggregate erodes less than mortar

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of equal strength, and hard aggregate improves the erosion resistance. However, under some conditions of wear, smaller-size aggregate leads to a more uniform erosion of the surface. In general, at a constant slump, the erosion resistance increases with a decrease in the cement content;[10.15] this has the advantage of reducing laitance. At a constant cement content, the resistance improves with a decrease in slump:[10.15] this is probably in agreement with the general influence of compressive strength.

In all cases, of course, it is only the quality of the concrete in the surface zone that is relevant, but even the best concrete will rarely withstand severe erosion over prolonged periods. Vacuum dewatering and use of permeable formwork are beneficial.

Proneness to erosion by solids in flowing water can be measured by means of a *shot-blast test*. Here, 2000 pieces of broken steel shot (of 850 μm (No. 20 ASTM) sieve size) are ejected under air pressure of 0.62 MPa (90 psi) from a 6.3 mm ($\frac{1}{4}$

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in.) nozzle against a concrete specimen 102 mm (4 in.) away.

Cavitation resistance

While good quality concrete can withstand steady, tangential, high-velocity flow of water, severe damage rapidly occurs in the presence of cavitation. By this is meant the formation of vapour bubbles when the local absolute pressure drops to the value of the ambient vapour pressure of water at the ambient temperature. The bubbles or cavities can be large, single voids, which later break up, or clouds of small bubbles.[10.16] They flow downstream and, on entering an area of higher pressure, collapse with great impact. Because the collapse of the cavities means entry of high-velocity water into the previously vapour-occupied space, extremely high pressure on a small area is generated during very short time intervals, and it is the repeated collapse over a given part of the concrete surface that causes pitting. Greatest damage is caused by clouds of minute

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cavities found in eddies. They usually coalesce momentarily into a large amorphous cavity which collapses extremely rapidly.[10.17] Many of the cavities pulsate at a high frequency, and this seems to aggravate damage over an extended area.[10.18]

Cavitation damage occurs in open channels generally only at velocities in excess of 12 m/s (40 ft/s),[10.41] but in closed conduits even at much lower speeds when there is a possibility of pressure dropping well below atmospheric. Such a drop may be caused by syphonic action, or by inertia on the inside of a bend or over boundary irregularities; often, there is a combination of these. Divergence of flow from the concrete surface of an open channel is a frequent cause of cavitation. Although the advent of cavitation depends primarily on pressure changes (and consequently also on velocity changes), it is especially likely to occur in the presence of small quantities of undissolved air in the water. These bubbles of air behave as nuclei at which the change of

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phase from liquid to vapour can more readily occur. Dust particles have a similar effect, possibly because they 'house' the undissolved air. On the other hand, small bubbles of free air in large quantities (up to 8 per cent by volume near the surface of concrete), while promoting cavitation, cushion the collapse of the cavities and hence reduce the cavitation damage.[10.19] Deliberate aeration of water may therefore be advantageous.[10.41]

The surface of concrete affected by cavitation is irregular, jagged and pitted, in contrast to the smoothly worn surface of concrete eroded by water-borne solids. The cavitation damage does not progress steadily: usually, after an initial period of small damage, rapid deterioration occurs, followed by damage at a slower rate.[10.19]

Best resistance to cavitation damage is obtained by the use of high strength concrete, possibly formed by an absorptive lining (which reduces the local water/cement ratio). The maximum size of aggregate near the surface should

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not exceed 20 mm ($\frac{3}{4}$ in.)[10.19] because cavitation tends to remove large particles. Hardness of aggregate is not important (unlike the case of erosion resistance) but good bond between aggregate and mortar is vital.

Use of polymers, steel fibres or resilient coatings may improve the cavitation resistance, but these topics are outside the scope of the present book. However, while the use of suitable concrete may reduce cavitation damage, not even the best concrete can withstand cavitation forces for an indefinite time. The solution of the cavitation damage problem lies, therefore, primarily in reducing cavitation. This can be achieved by the provision of smooth and well-aligned surfaces free from irregularities such as depressions, projections, joints and misalignments, and by the absence of abrupt changes in slope or curvature that tend to pull the flow away from the surface. If possible, local increase in velocity of water should be avoided as damage is proportional to the sixth or seventh power of velocity.[10.19]

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Types of cracking

Because cracking may impair the durability of concrete by allowing ingress of aggressive agents, it is relevant to review briefly the types and causes of cracking. In addition, cracking may adversely affect the watertightness or sound transmission of structures or mar their appearance. With respect to appearance, the acceptable crack width depends on the distance from which it is viewed and on the function of the structure, e.g. a public hall, at one extreme, and a warehouse, at the other. It may be useful to add that ingress of dirt makes the cracks more perceptible; so does the use of white cement in concrete.

As far as water-tightness is concerned, very narrow, non-moving cracks, 0.12 to 0.20 mm (0.005 to 0.008 in.) wide, may leak initially.[10.33,10.34] However, dissolved calcium hydroxide carried by slowly percolating water may react with atmospheric carbon dioxide to deposit

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calcium carbonate, which would seal the crack[10.33] (see p. 330).

Cracking occurring in fresh concrete, that is, plastic shrinkage cracking and plastic settlement cracking, was discussed in Chapter 9. Another type of early cracking is known as *crazing*, which can occur on slabs or walls when the surface zone of the concrete has a higher water content than deeper in the interior. The pattern of crazing looks like an irregular network with a spacing of up to about 100 mm (4 in.). The cracks are very shallow and develop early, but may not be noticed until etched by dirt; apart from appearance, they are of little importance.

In addition, a somewhat different kind of surface damage, known as *blisters*, can occur if some bleed water or large air bubbles are trapped just below the surface of the concrete by a thin layer of laitance induced by finishing. Blisters are 10 to 100 mm ($\frac{1}{2}$ to 4 in.) in diameter and 2 to 10 mm (or $\frac{1}{16}$ to $\frac{1}{2}$ in.) thick. In service, the laitance

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layer becomes detached, leaving behind a shallow depression.

In hardened concrete, cracking may be caused by drying shrinkage or by restrained early-age thermal movement; these were discussed in Chapters 9 and 8, respectively. The various types of non-structural cracks are listed in Table 10.8 and shown schematically in Fig. 10.12.[10.33] It is useful to note that, whereas one particular cause may initiate a crack, its development can be due to another cause.[10.33] Thus a diagnosis of causes of cracking is not always straightforward.

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Table 10.8. Classification of Intrinsic Cracks (based on ref. 10.33)

Type of cracking	*Symbol in Fig. 10.12*	*Subdivision*	*Most common location*	*Primary cause (excluding restraint)*	*Secondary causes/ factors*	*Remedy (assuming basic redesign is impossible); in all cases reduce restraint*	*Time of appearance*	*Reference in this book*
Plastic settlement	*A*	Over reinforcement	Deep sections	Excess bleeding	Rapid early drying conditions	Reduce bleeding or revibrate	10 min to 3 h	pp. 400 and 425
	B	Arching	Top of columns					
	C	Change of depth	Trough and waffle slabs					
Plastic shrinkage	*D*	Diagonal	Pavements and slabs	Rapid early drying	Low rate of bleeding	Improve early curing	30 min to 6 h	pp. 400 and 424
	E	Random	Reinforced concrete slabs					
	F	Over reinforcement	Reinforced concrete slabs	Rapid early drying or steel near surface				
Early thermal contraction	*G*	External restraint	Thick walls	Excess heat generation	Rapid cooling	Reduce heat and/or insulate	1 day to 2 or 3 weeks	pp. 395 and 400
	H	Internal restraint	Thick slabs	Excess temperature gradients				
Long-term drying shrinkage	*I*		Thin slabs and walls	Inefficient joints	Excess shrinkage Inefficient curing	Reduce water content Improve curing	Several weeks or months	p. 442
Crazing	*J*	Against formwork	Walls	Impermeable formwork	Rich mixes Poor curing	Improve curing and finishing	1 to 7 days, sometimes much later	p. 528
	K	Floated concrete	Slabs	Over-trowelling				
Corrosion of reinforcement	*L*	Carbonation / Chloride	Columns and beams	Inadequate cover	Poor quality concrete	Eliminate causes listed	More than 2 years	p. 566
Alkali–aggregate reaction	*M*		Damp locations	Reactive aggregate plus high-alkali cement		Eliminate causes listed	More than 5 years	p. 517
Blister	*N*		Slabs	Trapped bleed water	Use of metal float	Eliminate causes listed	Upon touching	p. 530
D-cracking	*P*		Free edges of slabs	Frost-damaged aggregate		Reduce aggregate size	More than 10 years	p. 546

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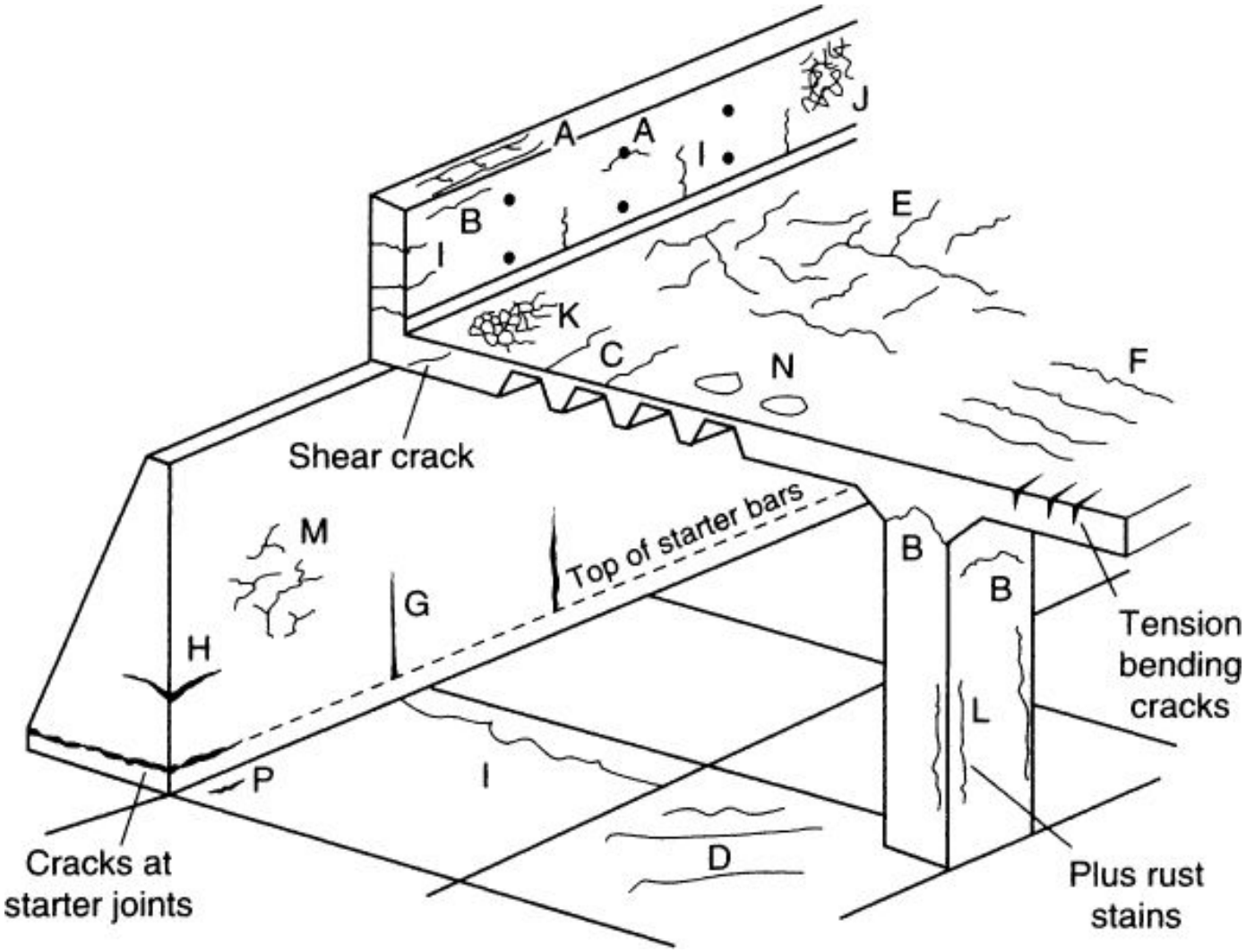


Fig. 10.12. Schematic representation of the various types of cracking which can occur in concrete (see Table 10.8) (based on ref. 10.33)

Cracking can also be caused by overloading in relation to the actual strength of the concrete member, but this is the consequence of inadequate design, or of construction not conforming to the specification. It is important to remember that, in reinforced concrete in service, tension is

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induced in the reinforcing steel and in the surrounding concrete. Surface cracking is therefore inevitable but, with proper structural design and detailing, the cracks are very narrow and barely perceptible. Stress-induced cracks have a maximum width at the surface of the concrete and taper towards the steel, but the difference in width may decrease with time.[10.34] The crack width at the surface is greater the larger the cover to reinforcement.

We should note that, from energy considerations, it is easier to extend an existing crack than to form a new one. This explains why, under an applied load, each subsequent crack occurs under a higher load than the preceding one. The total number of cracks developed is determined by the size of the concrete member, and the distance between cracks depends on the maximum size of aggregate present.[10.106]

Because, under given physical conditions, the total crack width per unit length of concrete is fixed and we want the cracks to be as fine as

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possible, it is desirable to have more cracks. For this reason, the restraint to cracking should be uniform along the length of the member. Provision of reinforcement controls shrinkage cracking by reducing the width of individual cracks, but not the total width of all the cracks taken together. This topic is outside the scope of the present book.

The importance of cracking, and the minimum width at which a crack is considered significant, depend on the function of the structural members and on the conditions of exposure of the concrete. Reis *et al.*[10.105] suggested the following permissible crack widths, which still offer good guidance:

Interior members
mm (0.014 in.)

Exterior members under normal exposure conditions 0.25 mm (0.010 in.)

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Exterior members exposed to particularly aggressive environment 0.15 mm (0.006 in.).

It may be relevant to mention that, although there is a variation between observers, the minimum crack width that can be seen with a naked eye is about 0.13 mm (0.005 in.). Simple magnifying devices make it possible to determine the crack width. Various specialized techniques, such as electro-conductive paint and light-dependent resistors, make it possible to determine the development of cracking. However, very fine cracks are very common but not harmful, so that intensive searching for cracks serves no purpose.

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Chapter 11. Effects of freezing and thawing and of chlorides

This chapter is concerned with two, sometimes separate and sometimes linked, mechanisms of damage to concrete. The first of these, although relevant only in cold climates, is a major cause of a lack of durability of concrete unless proper precautions are taken. The second mechanism, that is, the action of chlorides, is relevant only to reinforced concrete but it, too, can result in extensive damage of structures. The action of chlorides is encountered both in cold climates and in hot climates, but the details under the two conditions differ from one another.

Action of frost

In Chapter 8, the effects of frost on fresh concrete were considered and methods of avoiding freezing of fresh concrete were discussed. What cannot be avoided, however, is the exposure of ma-

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ture concrete to alternating freezing and thawing – a temperature cycle frequently met with in nature.

As the temperature of saturated concrete in service is lowered, the water held in the capillary pores in the hardened cement paste freezes in a manner similar to the freezing in the pores in rock, and expansion of the concrete takes place. If subsequent thawing is followed by re-freezing, further expansion takes place, so that repeated cycles of freezing and thawing have a cumulative effect. The action takes place mainly in the hardened cement paste; the larger voids in concrete, arising from incomplete compaction, are usually air-filled and, therefore, not appreciably subject to the action of frost.[11.4]

Freezing is a gradual process, partly because of the rate of heat transfer through concrete, partly because of a progressive increase in the concentration of dissolved salts in the still unfrozen pore water (which depresses the freezing point), and partly because the freezing point var-

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ies with the size of the pore. Because the surface tension of the bodies of ice in the capillary pores puts them under pressure that is higher the smaller the body, freezing starts in the largest pores and gradually extends to smaller ones. Gel pores are too small to permit the formation of nuclei of ice at temperatures higher than –78 °C, so that in practice no ice is formed in them.[11.4] However, with a fall in temperature, because of the difference in entropy of gel water and ice, the gel water acquires an energy potential enabling it to move into the capillary pores containing ice. The diffusion of gel water which takes place leads to a growth of the ice body and to expansion.[11.4]

There are thus two possible sources of dilating pressure. First, freezing of water results in an increase in volume of approximately 9 per cent, so that the excess water in the cavity is expelled. The rate of freezing will determine the velocity with which water displaced by the advancing ice front must flow out, and the hydraulic pressure developed will depend on the resistance to flow,

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i.e. on the length of path and the permeability of the hardened cement paste in the area between the freezing cavity and a void that can accommodate the excess water.[11.5]

The second dilating force in concrete is caused by diffusion of water leading to a growth of a relatively small number of bodies of ice. Although the action of freezing and thawing upon concrete is still debated, the latter mechanism is believed to be particularly important in causing damage of concrete.[11.6] This diffusion is caused by osmotic pressure brought about by local increases in solute concentration due to the separation of frozen (pure) water from the pore water. A slab freezing from the top will be seriously damaged if water has access from the bottom and can travel through the thickness of the slab due to osmotic pressure. The total moisture content of the concrete will then become greater than before freezing, and in a few cases damage by segregation of ice crystals into layers has actually been observed.[11.7,11.47]

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Osmotic pressure arises also in another connection. When salts are used for de-icing road or bridge surfaces, some of these salts become absorbed by the upper part of the concrete. This produces a high osmotic pressure, with a consequent movement of water toward the coldest zone where freezing takes place. The action of de-icing salts is considered in a later section of this chapter.

When the dilating pressure in the concrete exceeds its tensile strength, damage occurs. The extent of the damage varies from surface scaling to complete disintegration as ice is formed, starting at the exposed surface of the concrete and progressing through its depth. Under the conditions prevailing in a temperate climate, road kerbs (curbs) (which remain wet for long periods) are more vulnerable to frost than any other concrete. The second most severe conditions are those in a road slab, particularly when salt is used for de-icing. In countries with a colder climate, the dam-

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age due to frost is more general and, unless suitable precautions are taken, more serious.

At this stage, it may be useful to consider why it is *alternating* freezing and thawing that causes progressive damage. Each cycle of freezing causes a migration of water to locations where it can freeze. These locations include fine cracks which become enlarged by the pressure of the ice and remain enlarged during thawing when they become filled with water. Subsequent freezing repeats the development of pressure and its consequences.

While the resistance of concrete to freezing and thawing depends on its various properties (e.g. strength of the hardened cement paste, extensibility, and creep), the main factors are the degree of saturation and the pore system of the hardened cement paste. The general influence of saturation of concrete is shown in Fig. 11.1: below some critical value of saturation, concrete is highly resistant to frost,[11.2] and dry concrete is totally unaffected. In other words, if concrete is

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never going to be saturated, there is no danger of damage from freezing and thawing. It may be noted that, even in a water-cured specimen, not all residual space is water-filled, and indeed this is why such a specimen does not fail on first freezing.[11.8] A large proportion of concrete in service dries partially, at least at some time in its life and, on rewetting, such concrete will not re-absorb as much water as it has lost.[11.9] It is desirable, therefore, to allow concrete to dry out before exposure to winter conditions, and failure to do so will increase the severity of frost damage. An example of the influence of age at which first freezing takes place upon damage to concrete is shown in Fig. 11.2.[11.3]

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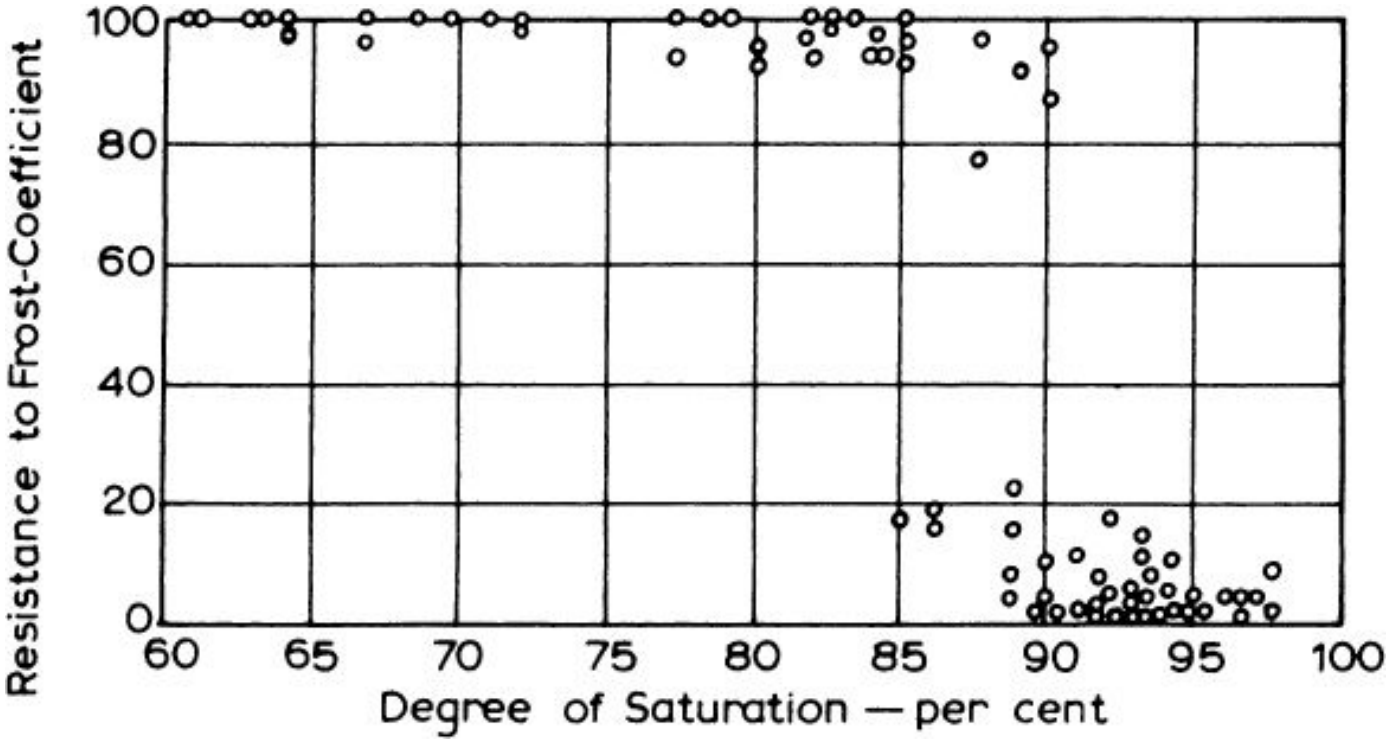


Fig. 11.1. Influence of saturation of concrete on its resistance to frost expressed by an arbitrary coefficient[11.2]

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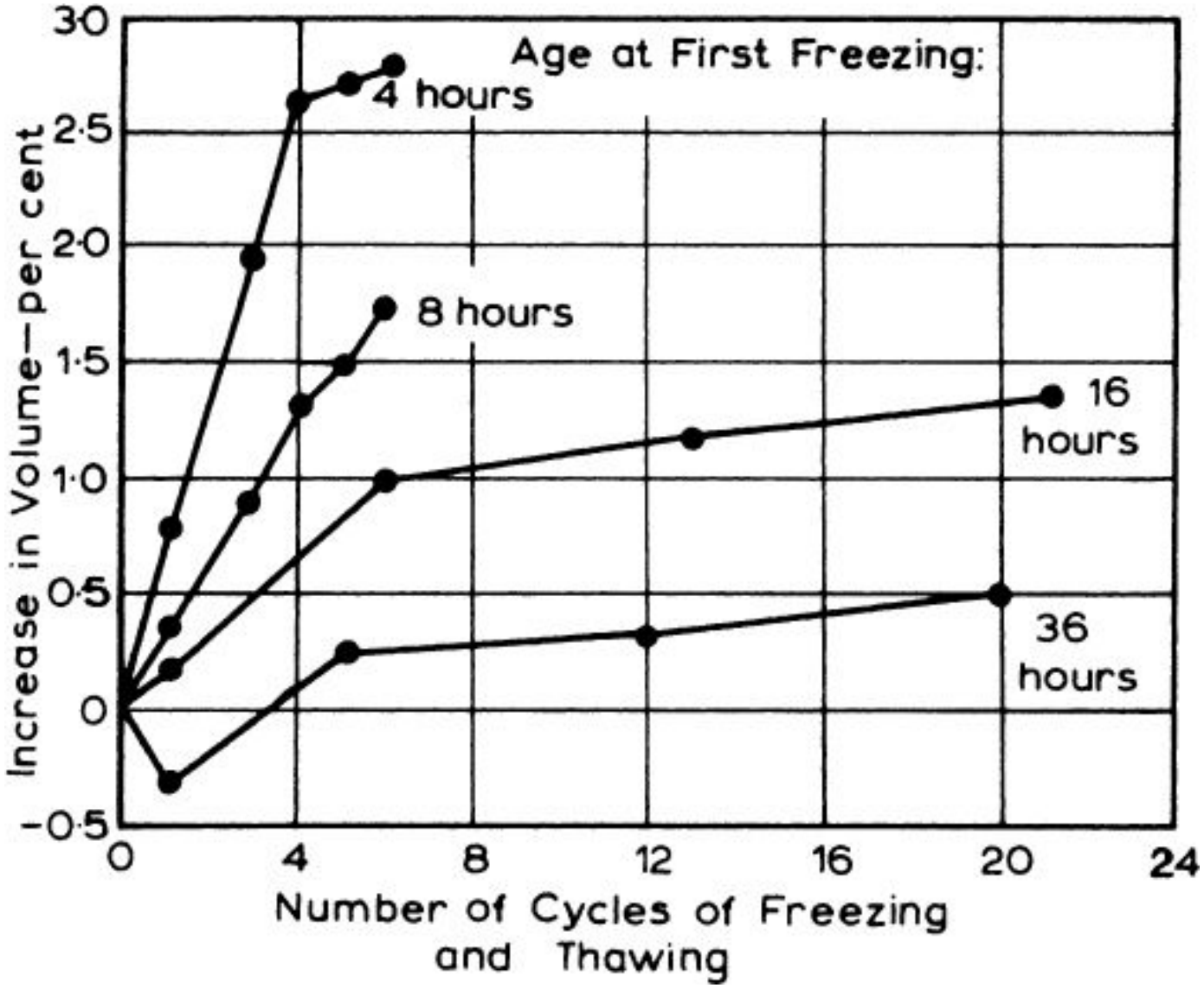


Fig. 11.2. Increase in volume of concrete subjected to freezing and thawing as a function of age at which first freezing starts[11.3]

What is the critical value of saturation? A closed container with more than 91.7 per cent of its volume occupied by water will, on freezing, become filled with ice, and will become subjected to bursting pressure. Thus, 91.7 per cent can

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be considered to be the critical saturation in a closed vessel. This is not, however, the case in a porous body, where the critical saturation depends on the size of the body, on its homogeneity, and on the rate of freezing. Space available for expelled water must be close enough to the cavity in which ice is being formed, and this is the basis of air entrainment: if the hardened cement paste is subdivided into sufficiently thin layers by air bubbles, it has no critical saturation.

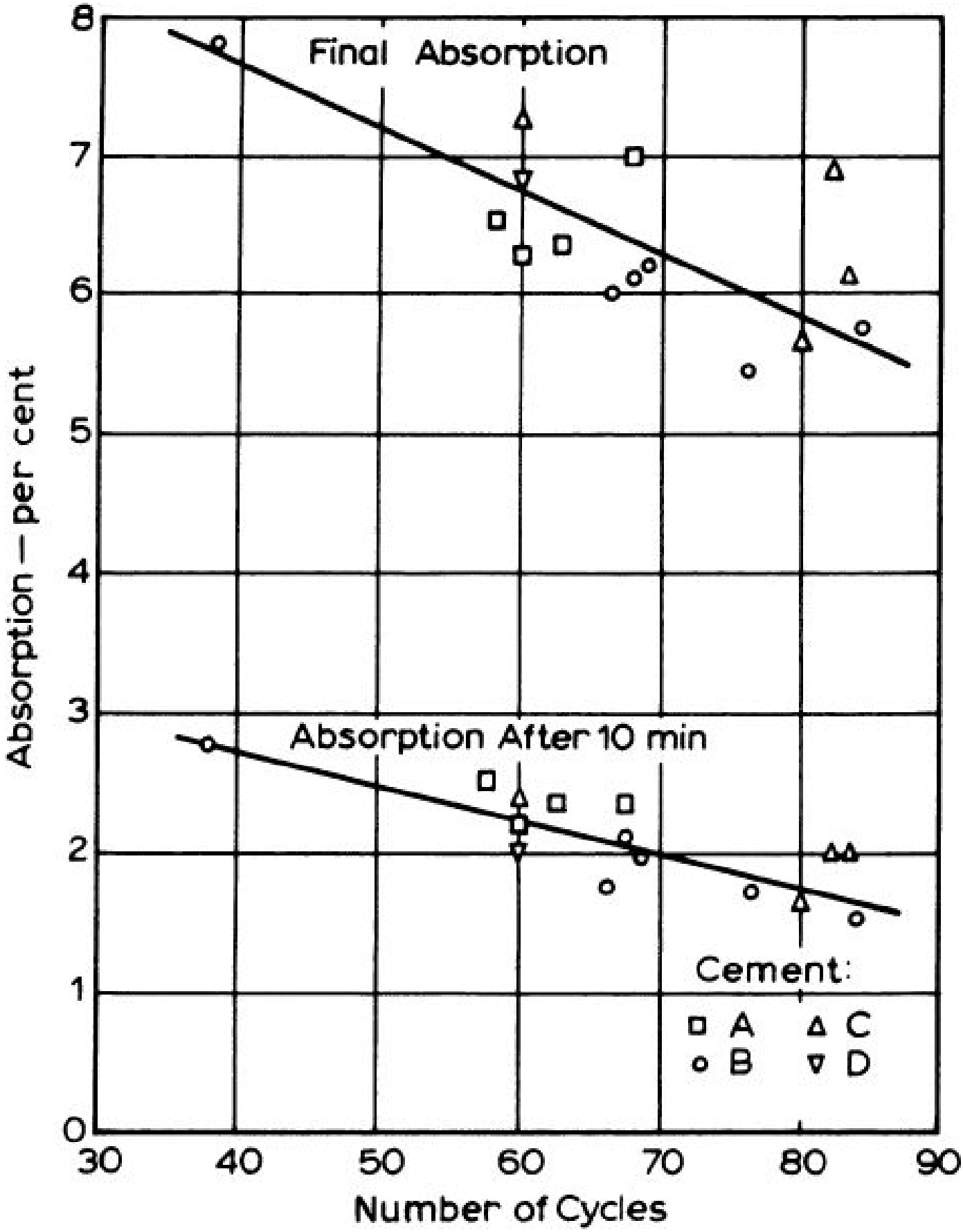
Air bubbles can be introduced by air entrainment, which is discussed later in this chapter. Although air entrainment greatly enhances the resistance of concrete to cycles of freezing and thawing, it is vital that the concrete has a low water/cement ratio so that the volume of capillary pores is small. It is also essential that substantial hydration takes place before exposure to freezing. Such concrete has a low permeability and imbibes less water in wet weather.

Figure 11.3 shows the general effect of the absorption of concrete on its resistance to freezing

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and thawing,[11.99] and Fig. 11.4 illustrates the influence of the water/cement ratio on the resistance to freezing and thawing of concrete moist-cured for 14 days and then stored in air of 50 per cent relative humidity for 76 days prior to exposure to freezing and thawing.[11.11].

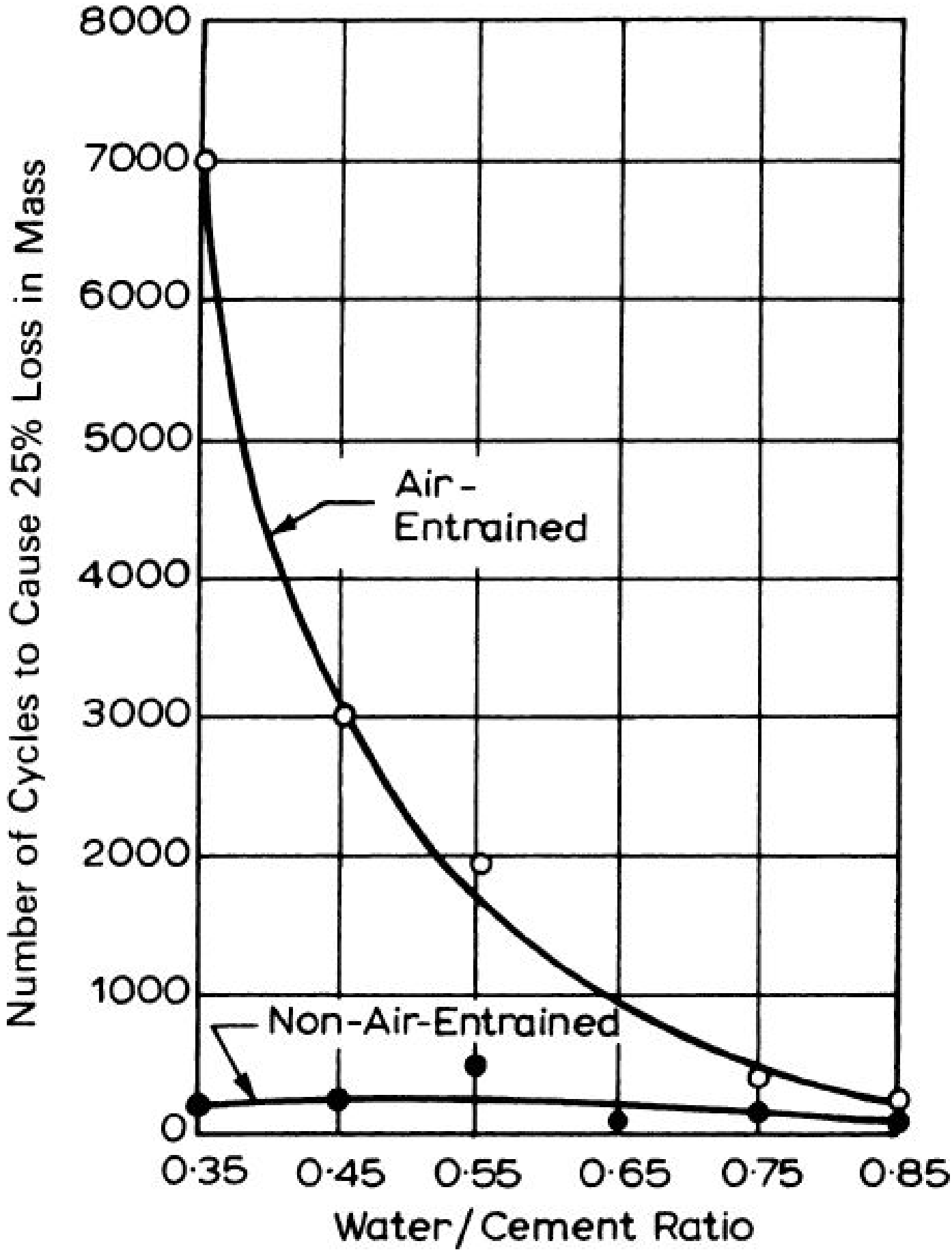
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Fig. 11.3. Relation between absorption of concrete and the number of cycles of freezing and thawing required to cause a 2 per cent reduction in the mass of the specimen[11.99]

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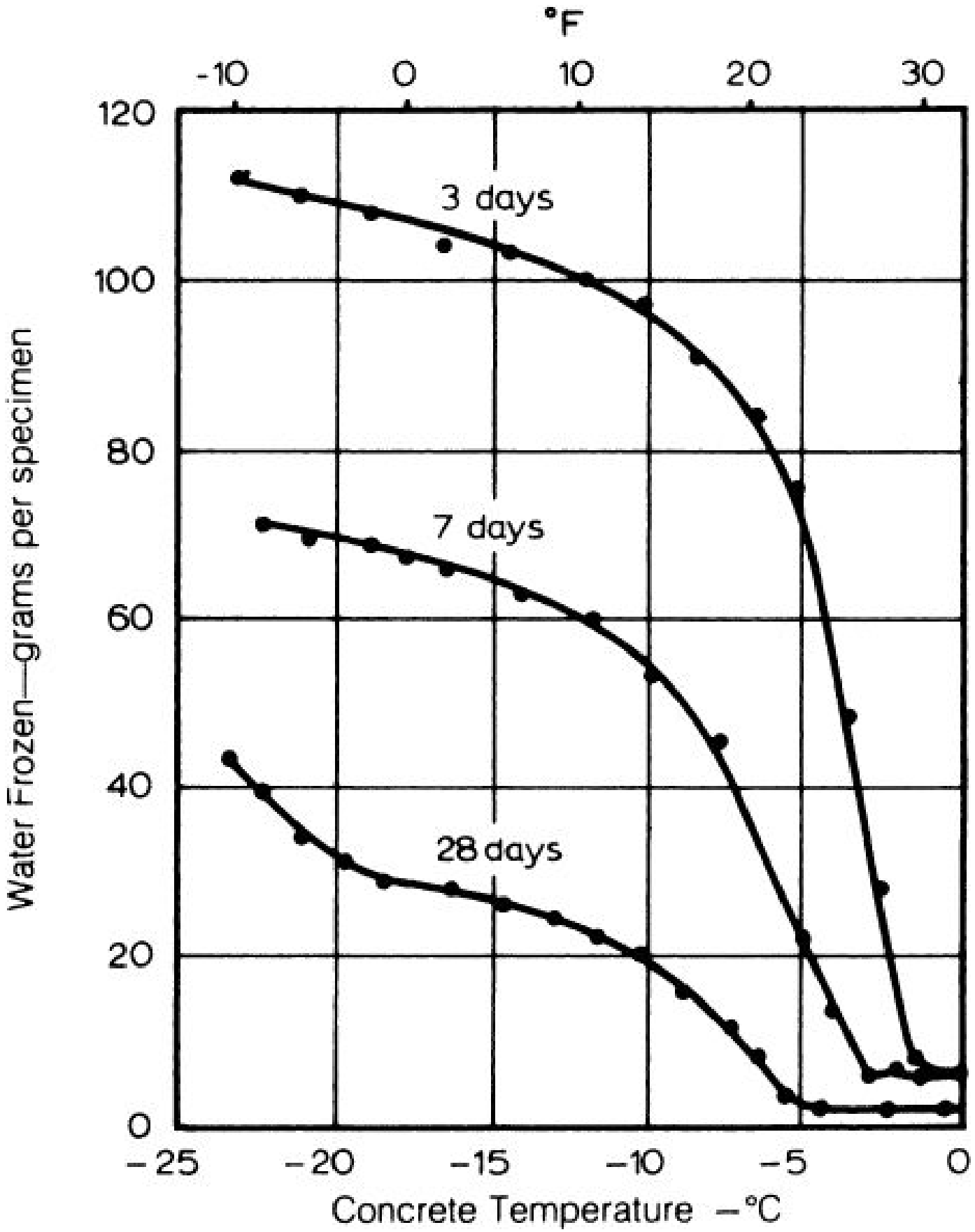


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Fig. 11.4. Influence of the water/cement ratio on the resistance to freezing and thawing of concrete moist-cured for 14 days and then stored for 76 days at a relative humidity of 50 per cent[11.11]

Adequate curing is vital to reduce the amount of freezable water in the paste; this is illustrated in Fig. 11.5 for concrete with a water/cement ratio of 0.41. This figure shows also that the freezing temperature decreases with age because of an increase in the concentration of salts in the still remaining freezable water. In all cases, a small amount of water freezes at 0 °C (32 °F), but this is probably free surface water on the specimen. The temperatures at which freezing of capillary water starts were found to be, approximately, –1 °C (30 °F) at 3 days, –3 °C (27 °F) at 7 days, and –5 °C (23 °F) at 28 days.[11.12]

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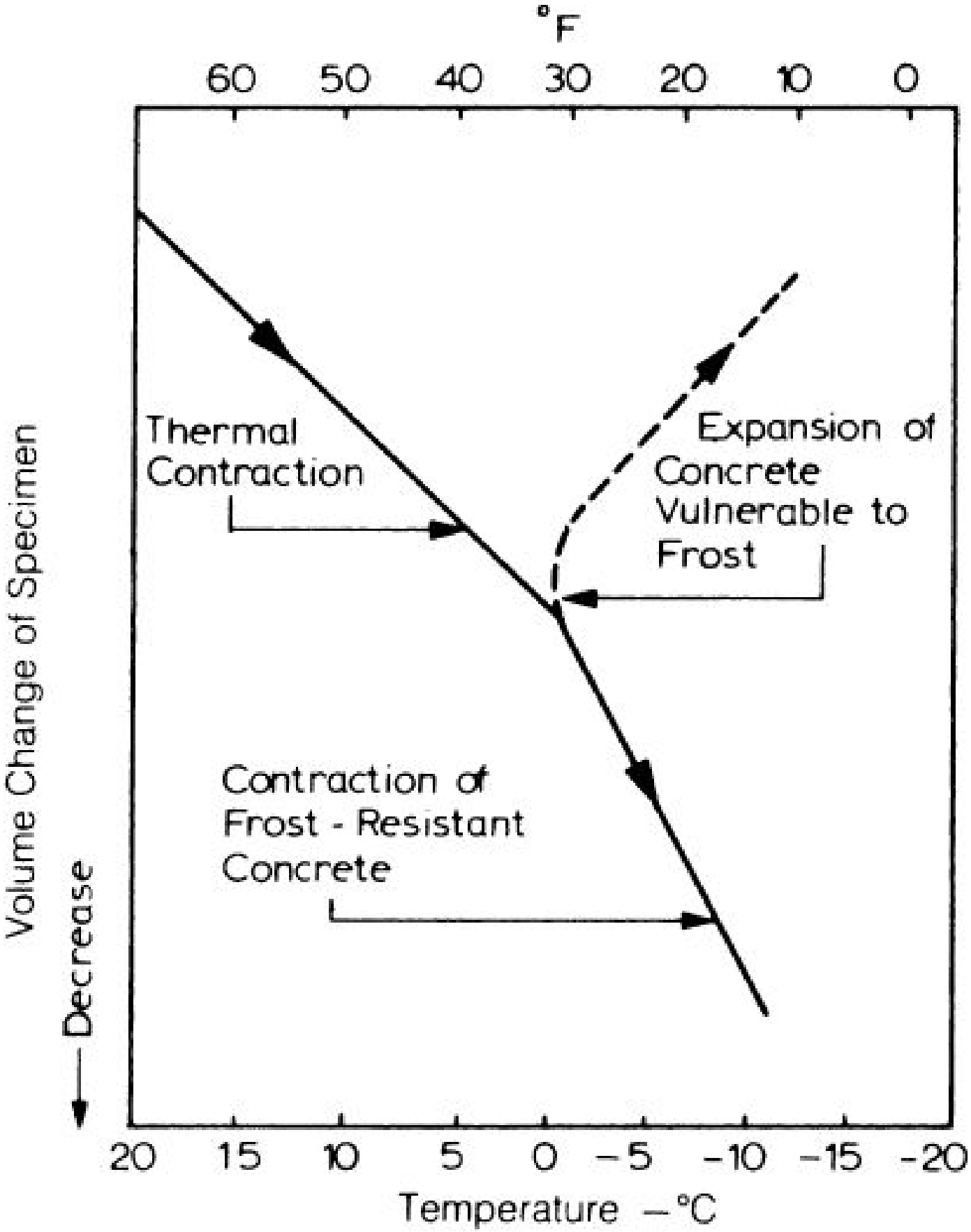


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Fig. 11.5. Effect of age of concrete on amount of water frozen, as a function of temperature[11.12]

Whether or not a given concrete is vulnerable to frost, be it due to the expansion of the hardened cement paste or of the aggregate, can be determined by cooling the specimen through the freezing range and measuring the change in volume: frost-resistant concrete will contract when water is transferred by osmosis from the hardened cement paste to the air bubbles, but vulnerable concrete will dilate, as shown in Fig. 11.6. This one-cycle test is very useful.[11.23] It has been found that the maximum dilation on first freezing correlates linearly with the residual expansion on subsequent thawing; the latter can, therefore, also be used as an indicator of the vulnerability of concrete.[11.26]

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Fig. 11.6. Change in volume of frost-resistant and vulnerable concretes on cooling[11.4]

ASTM C 671-94 prescribed a test method for the critical dilation of concrete subjected to repeated two-week cycles of short freezing and prolonged storage in water. The length of time until critical dilation occurs can be used to rank concretes in terms of their resistance to freezing and thawing under the given conditions. This standard has been withdrawn.

Behaviour of coarse aggregate particles

Consideration of critical saturation applies also to individual particles of coarse aggregate. An aggregate particle *by itself* will not be vulnerable if it has a very low porosity, or if its capillary system is interrupted by a sufficient number of macropores. However, an aggregate particle in concrete can be considered as a closed container, because the low permeability of the surrounding hardened cement paste will not allow water to move sufficiently rapidly into air voids. Thus an

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aggregate particle saturated above 91.7 per cent will, on freezing, destroy the surrounding mortar.[11.4] It may be recalled that common aggregates have a porosity of 0 to 5 per cent, and it is preferable to avoid aggregates of high porosity. However, the use of such aggregates need not necessarily result in frost damage. Indeed, large pores present in aerated concrete and in no-fines concrete probably contribute to the frost resistance of those materials. Furthermore, even with ordinary aggregate, no simple relation between the porosity of the aggregate and the resistance to freezing and thawing of the concrete has been established.

If a vulnerable particle is near the surface of the concrete, instead of disrupting the surrounding hardened cement paste, it can cause a *popout*.

The effect of drying of aggregate, prior to mixing, on the durability of concrete is shown in Fig. 11.7. It can be seen that the presence of saturated aggregate, particularly of large size, can result in the destruction of concrete, whether or not

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the concrete is air entrained. On the other hand, if the aggregate is not saturated at the time of mixing, or if it is allowed to dry partially after placing and the capillaries in the paste are discontinuous, re-saturation is not easily achieved except during a prolonged period of cold weather.[11.1] On rewetting of concrete, it is the hardened cement paste that tends to be more nearly saturated than the aggregate, as water can reach the aggregate only through the paste, and also because the finer-textured paste has a greater capillary attraction. As a result, the hardened cement paste is more vulnerable but it can be protected by air entrainment.

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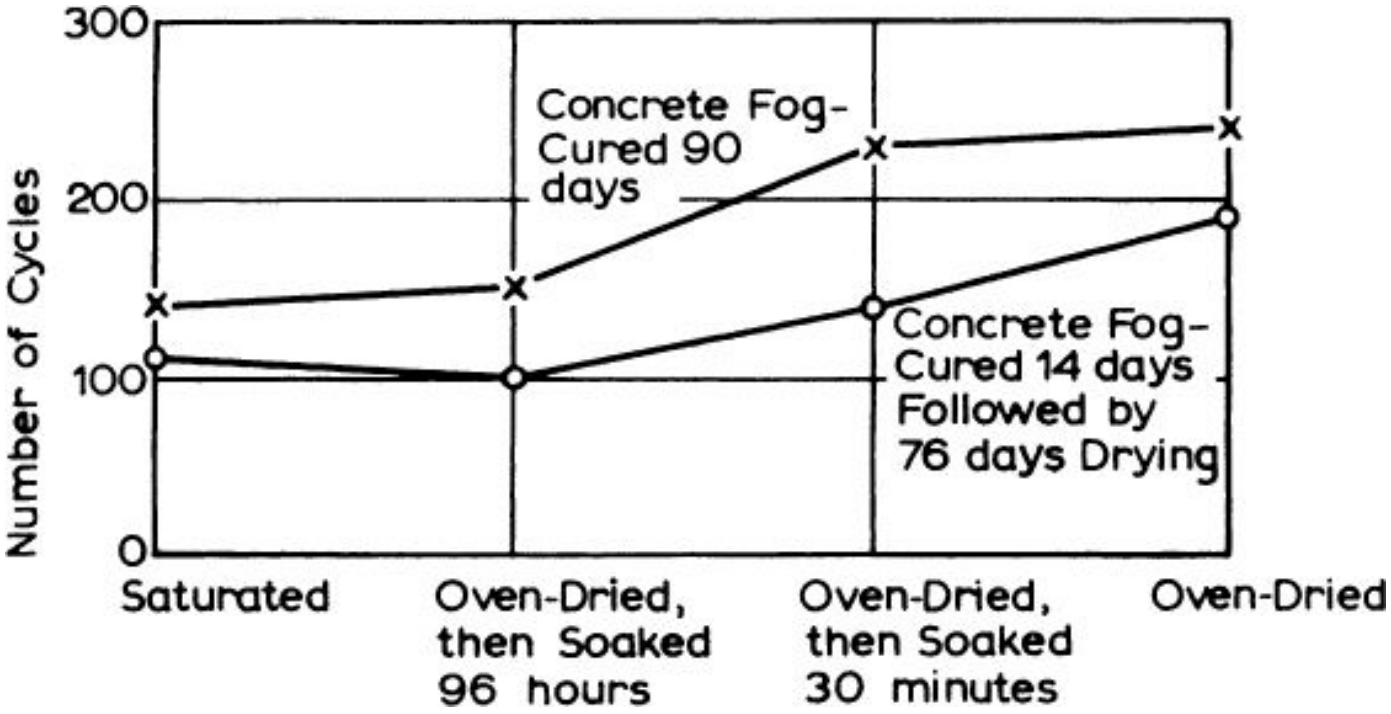


Fig. 11.7. Relation between the condition of aggregate before mixing and the number of cycles of freezing and thawing to produce a 25 per cent loss in the mass of the specimen[11.10]

Air entrainment of the cement paste does not alleviate the effects of freezing of coarse aggregate particles.[11.92] Nevertheless, aggregate should be tested in air-entrained concrete in order to exclude the effect of the durability of the surrounding hardened cement paste. For this reason, ASTM C 682-94 (withdrawn) provides for the evaluation of the frost resistance of coarse aggregate when used in air-entrained concrete using

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the test for the critical dilation of concrete subjected to freezing given in ASTM C 671-94 (withdrawn).

A test for frost heave of *unbound* aggregate is prescribed in BS 812 : 124 : 2009; although not directly applicable to aggregate in concrete, the test may be of interest in a preliminary investigation of previously unused aggregates.

There is one type of cracking of concrete road, bridge, and airfield surfaces which is particularly linked to aggregate. This is called *D-cracking*. It consists of the development of fine cracks near free edges of slabs, but the initial cracking starts lower in the slab where moisture accumulates and the coarse aggregate becomes saturated to the critical level. Thus, we have essentially a failure of aggregate, which, with cyclic freezing and thawing, becomes slowly saturated and causes failure of the surrounding mortar.[11.25] D-cracking can manifest itself very slowly, sometimes reaching the top of the slab only after 10 or 15 years,

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so that assignment of responsibility for failure is difficult.

Aggregates associated with D-cracking are nearly always of sedimentary origin and can be calcareous or siliceous. They can be gravel or crushed rock. While the absorption characteristics of the aggregate are clearly relevant to the proneness of concrete to D-cracking, the absorption value alone does not distinguish durable aggregate from non-durable. Freezing and thawing laboratory tests on concrete containing the given aggregate give a good indication of likely behaviour in service. If after 350 cycles the expansion is less than 0.035 per cent, D-cracking will not develop.[11.25] It should be noted that the same parent rock leads to less D-cracking when the aggregate particles are smaller (see Fig. 11.8); thus, comminution of a given aggregate may reduce the risk of D-cracking.[11.25]

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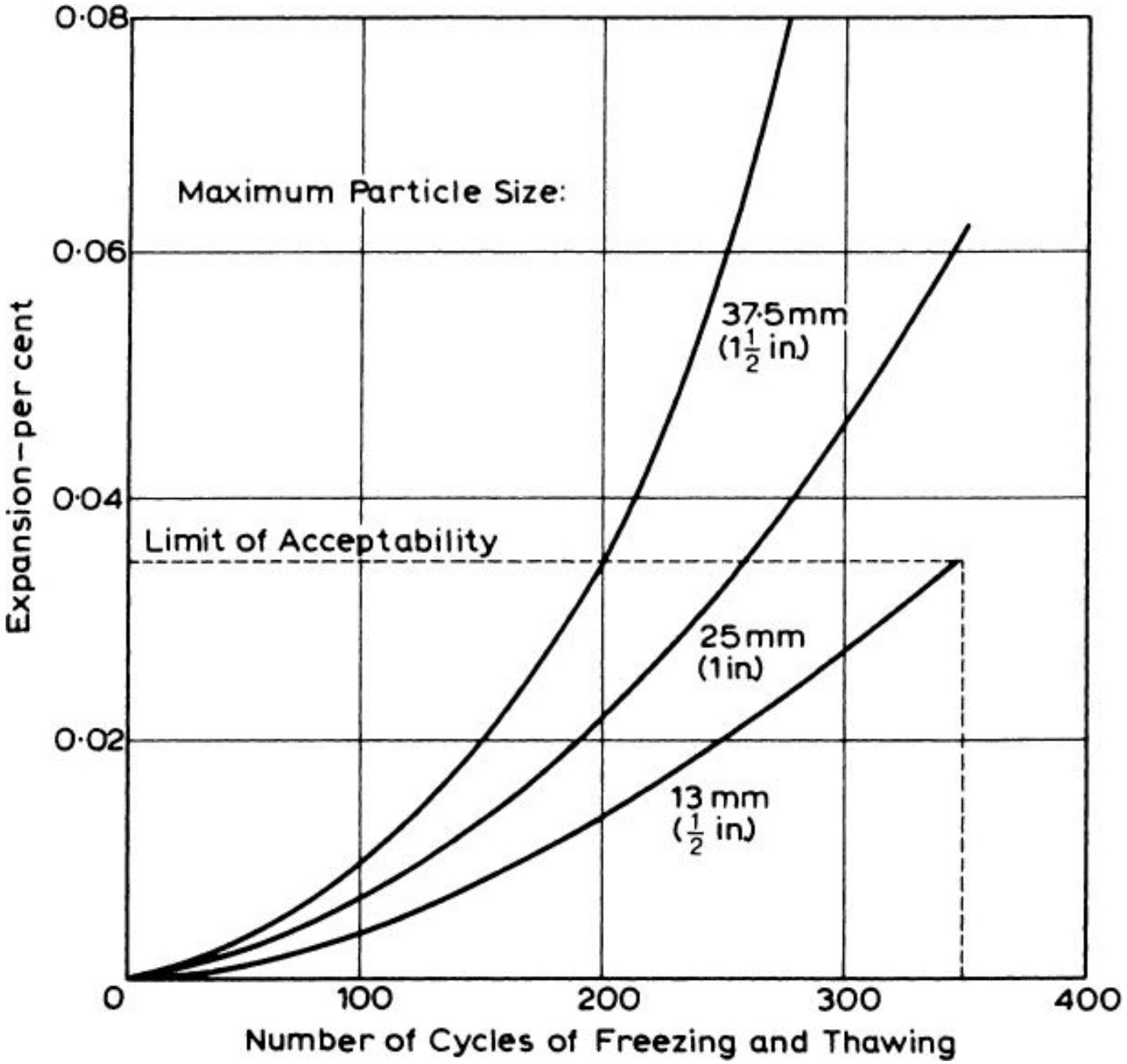


Fig. 11.8. Relation between maximum aggregate particle size and expansion in laboratory freezing and thawing tests. A failure criterion of 0.035 per cent expansion in 350 or fewer cycles is indicated[11.25]

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More generally, large aggregate particles are more vulnerable to frost.[11.34] Furthermore, the use of aggregate with a large maximum size or a large proportion of flat particles is inadvisable as pockets of bleed water may collect on the underside of the coarse aggregate. It is relevant to note that air entrainment reduces bleeding.

Air entrainment

Because the damaging action of freezing and thawing involves expansion of water on freezing, it is logical to expect that, if excess water can readily escape into adjacent *air-filled* voids, damage of concrete will not occur. This is the underlying principle of air entrainment. It should be emphasized, however, that the volume of capillary pores should be minimized in the first place as otherwise the volume of freezable water would exceed that which can be accommodated by the deliberately entrained air voids. This requirement translates into the need for an adequately low water/cement ratio, which also ensures a strength of

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concrete such that it can better resist the damaging forces induced by freezing. According to ACI 201.2R[11.92] to be resistant to freezing and thawing, concrete should have a water/cement ratio not greater than 0.50; this is reduced to 0.45 in thin sections including bridge decks and kerbs (curbs). Alternatively, concrete should not be exposed to cycles of freezing and thawing until its strength has reached 24 MPa (3500 psi).

Entrained air in concrete is defined as air intentionally incorporated by means of a suitable agent. This air should be clearly distinguished from accidentally entrapped air: the two kinds differ in the magnitude of the air bubbles, those of entrained air having typically a diameter of about 50 μm (0.002 in.), whereas accidental air usually forms very much larger bubbles, some as large as the familiar, albeit undesirable, pockmarks on the formed surface of concrete.

Entrained air produces discrete, nearly spherical, bubbles in the cement paste so that no channels for the flow of water are formed and the per-

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meability of the concrete is not increased. The voids never become filled with the products of hydration of cement as gel can form only in water.

The improved resistance of air-entrained concrete to frost attack was discovered accidentally when cement ground with beef tallow, added as a grinding aid, was observed to make more durable concrete than when no grinding aid was used. The main types of air-entraining agents are:

(a) salts of fatty acids derived from animal and vegetable fats and oils (beef tallow being an example of this group)

(b) alkali salts of wood resins, and

(c) alkali salts of sulfated and sulfonated organic compounds.

All these agents are surface-active agents, or surfactants, that is, long-chain molecules which orient themselves so as to reduce the surface tension of the water, the other end of the molecule being directed toward the air. Thus, the air

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bubbles formed during mixing become stabilized: they are covered by a sheath of air-entraining molecules repelling one another, and so preventing coalescence and ensuring a uniform dispersion of the entrained air.

Numerous types of air-entrained agents are available in the form of commercial admixtures, but the performance of unknown ones should be checked by trial mixes. ASTM C 260-06 and BS EN 934-2320 as well as BS EN 934-6 : 2001, lay down the performance requirements of air-entraining agents, usually called admixtures. The essential requirements of an air-entraining admixture are that it rapidly produces a system of finely divided and stable foam, the individual bubbles of which resist coalescence. The foam must have no harmful chemical effect on the cement.

The air-entraining admixture is normally dispensed into the mixer direct in the form of a solution. The timing of the discharge of the admixture into the mixer is of importance so as to en-

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sure a uniform distribution and adequate mixing for the formation of the foam. If other admixtures are also used, they should not come into contact with the air-entraining admixture prior to entering the mixer because their interaction could affect their performance.

Air-entraining agents can also be interground with cement but this allows no flexibility in the air content of the concrete, so that the use of air-entrained cements should generally be limited to minor construction.

Air-void system characteristics

Because the resistance to the movement of water through hardened cement paste must not be excessive to the point of preventing the flow, it follows that water, wherever it is located, must be sufficiently close to air-filled space, that is to the bubbles of entrained air. Thus, the fundamental requirement which ensures the efficacy of air entrainment is a limit on the maximum distance which the escaping water has to travel. The

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practical factor is the spacing of the air bubbles, i.e. the thickness of the hardened cement paste between adjacent air voids, which is twice the maximum distance referred to above. Powers[11.15] calculated that an average spacing of 250 μm (0.01 in.) between the voids is required for full protection from frost damage (Fig. 11.9); nowadays, 200 μm (0.008 in.) is usually recommended.[11.94]

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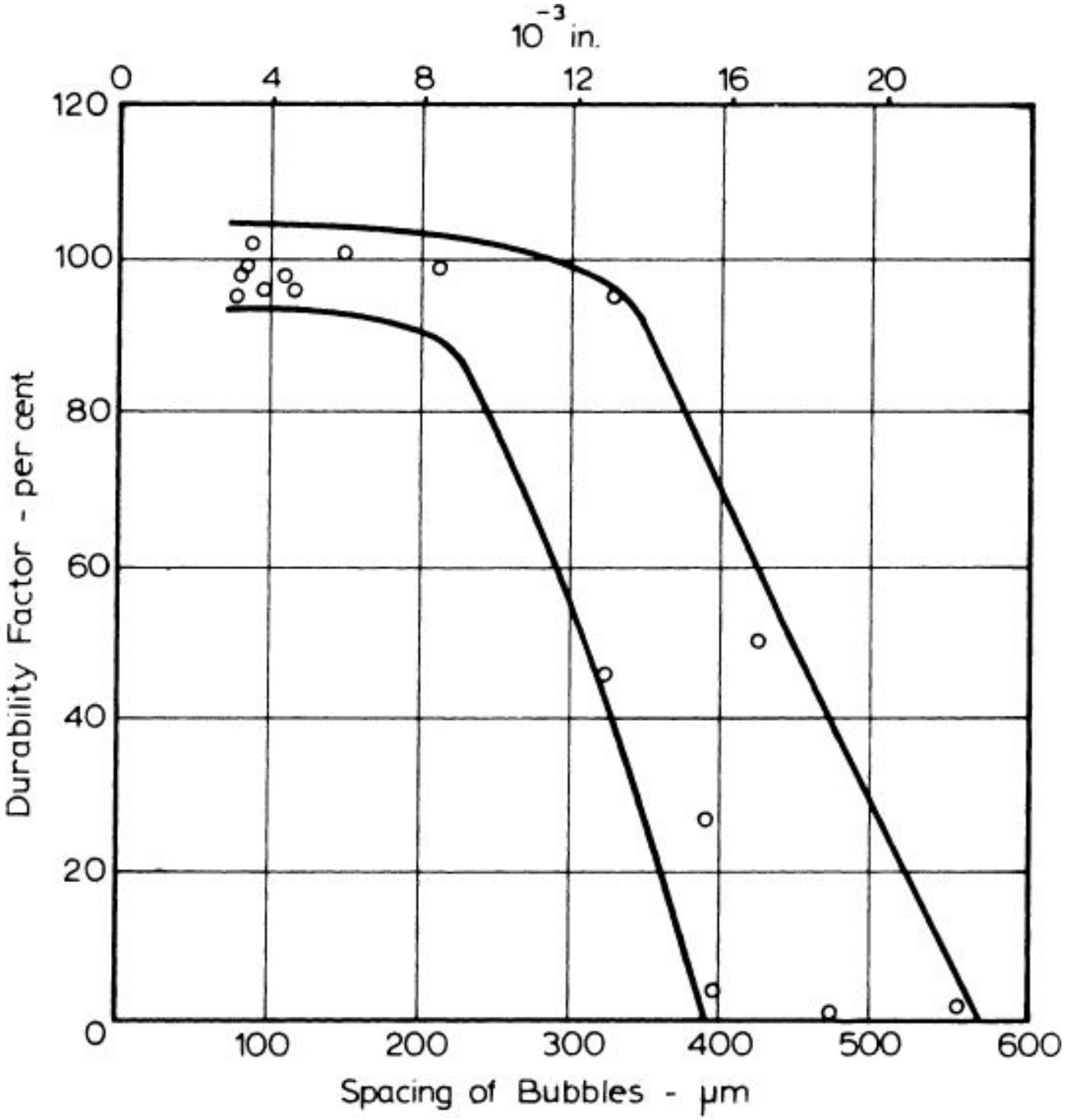


Fig. 11.9. Relation between durability and spacing of bubbles of entrained air[11.16]

Because the total volume of voids in a given volume of concrete affects the strength of con-

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crete (cf. p. 280) it follows that, for a given spacing, the air bubbles should be as small as possible. Their size depends to a large degree on the foaming process used. In fact, bubbles are not all of one size, and it is convenient to express their size in terms of specific surface (square millimetres per cubic millimetre or square inches per cubic inch).

It should not be forgotten that accidental (entrapped) air is present in any concrete, whether air-entrained or not, and, as the two kinds of voids cannot be distinguished other than by direct observation, the specific surface represents an average value for all voids in a given cement paste. For air-entrained concrete of satisfactory quality, the specific surface of voids is in the range of approximately 16 to 24 mm^{-1} (400 to 600 $in.^{-1}$), but sometimes it is as high as 32 mm^{-1} (800 $in.^{-1}$). By contrast, the specific surface of accidental air is less than 12 mm^{-1} (300 $in.^{-1}$).[11.15]

The adequacy of air entrainment in a given hardened concrete can be estimated by a spacing

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factor, $\bar{L}$, determined by a test method prescribed in ASTM C 457-10a. The spacing factor is a useful index of the maximum distance of any point in the hardened cement paste from the periphery of a nearby air void. The calculation of the factor is based on the assumption that all air voids are equal-sized spheres arranged in a simple cubic lattice. The calculation is laid down by ASTM C 457-10a and requires the knowledge of: the air content of the concrete, using a linear traverse microscope to determine the average number of air void sections per inch or the average chord intercept of the voids; and the hardened cement paste content by volume. The spacing factor is expressed in inches or millimetres; usually a value of not more than 200 μm (0.008 in.) is a maximum value required for satisfactory protection from freezing and thawing.

It may be useful to add that the water which has moved into air voids during freezing returns into the smaller capillary pores in the hardened cement paste during thawing. Thus, protection

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by air entrainment continues permanently for repeated freezing and thawing.[11.17] Rapid thawing followed by freezing is not harmful, as the water is already in the air voids; on the other hand, slow thawing followed by very rapid freezing may not allow sufficient movement of water to take place.

Entrained-air requirements

From the requirement of a maximum spacing of air voids, it is possible to calculate the minimum volume of entrained air in the hardened cement paste. For each mix, there is a minimum volume of voids required. Klieger[11.14] found this volume to correspond to 9 per cent of the volume of *mortar*. As the volume of hardened cement paste, in which alone the air is entrained, varies with the richness of the mix, the air content of *concrete* which is required depends on the mix proportions; in practice, the maximum size of aggregate is used as a parameter.

For a given air content, the spacing of air voids depends on the water/cement ratio of the

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mix as shown in Fig. 11.10. Specifically, the higher the water/cement ratio the larger the bubble spacing (and the lower the specific surface) because small bubbles coalesce.[11.42] The stability of air bubbles is considered on p. 553.

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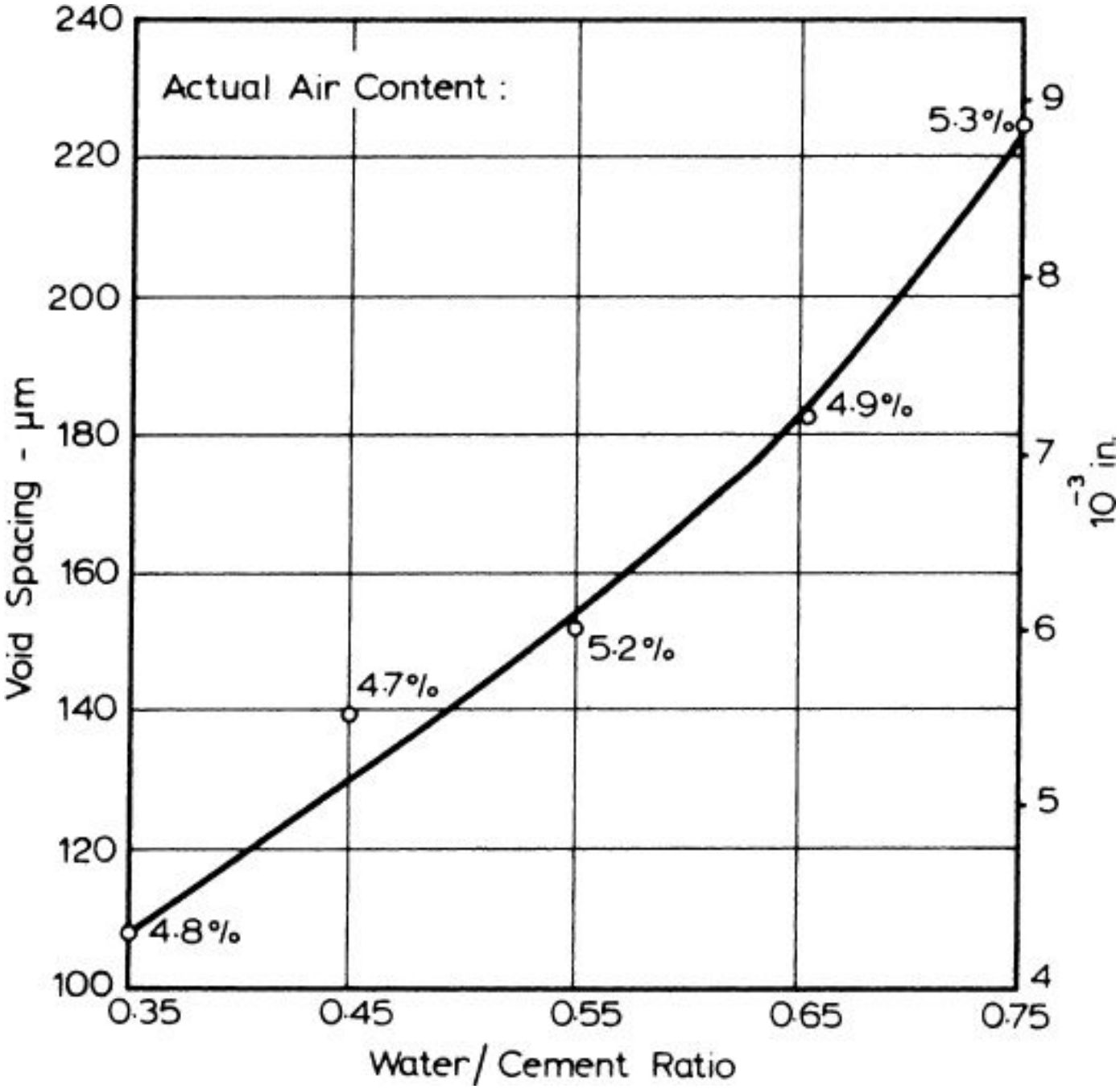


Fig. 11.10. Influence of the water/cement ratio on the void spacing in concrete with an average air content of 5 per cent[11.11]

Typical values of the amount of air required for 250 μm (0.01 in.) spacing for different mixes

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are given in Table 11.1, based on Powers' results.[11.15] A higher specific surface, which corresponds to smaller bubbles, is desirable so as to minimize the adverse effect of the air in concrete on its strength. Table 11.1 indicates that, for a particular value of the specific surface of the air voids, richer mixes require a greater volume of entrained air than lean ones. However, the richer the mix the greater the specific surface of the voids for a given air content. This is illustrated in Table 11.2, based on ref. 11.14.

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Table 11.1. Air Content Required for a Void Spacing of 250 μm (0.01 in.)[11.15]

Approximate cement content of concrete		*Water/cement ratio*	*Air requirement as a percentage of volume of concrete for specific surface of voids, mm^{-1} ($in.^{-1}$), of:*				
kg/m^3	*lb/yd^3*		*14 (350)*	*18 (450)*	*20 (500)*	*24 (600)*	*31 (800)*
445	750	0.35	8.5	6.4	5.0	3.4	1.8
390	660		7.5	5.6	4.4	3.0	1.6
330	560		6.4	4.8	3.8	2.5	1.3
445	750	0.49	10.2	7.6	6.0	4.0	2.1
390	660		8.9	6.7	5.3	3.5	1.9
330	560		7.6	5.7	4.5	3.0	1.6
280	470		6.4	4.8	3.8	2.5	1.3
445	750	0.66	12.4	9.4	7.4	5.0	2.6
390	660		10.9	8.2	6.4	4.3	2.3
330	560		9.3	7.0	5.5	3.7	1.9
280	470		7.8	5.8	4.6	3.1	1.6
225	380		6.2	4.7	3.7	2.5	1.3

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Table 11.2. Example of the Influence of the Cement Content of the Mix upon the Specific Surface of Air Voids in Concrete with a Maximum Size of Aggregate of 19 mm ($\frac{3}{4}$ in.) (based on ref. 11.14)

Cement content		Optimum air content per cent	Specific surface of voids	
kg/m^3	lb/yd^3		mm^{-1}	$in.^{-1}$
223	376	6.5	13	330
307	517	6.0	17	420
391	658	6.0	23	580

It can be noted that appropriately higher values may be required in grout in prestressed concrete ducts; the voids induced by aluminium powder which reacts with the alkalis, used to ensure complete filling of a duct, are insufficient for frost protection.

The severity of exposure of the concrete affects the value of the air content which should be specified,[11.92] as shown in Table 11.3 in which "severe exposure" describes conditions such that concrete may be in almost continuous contact

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with moisture prior to freezing or where de-icing salts are used; the air content in mortar is expected to be 9 per cent. “Moderate exposure” describes conditions when concrete is only occasionally exposed to moisture prior to freezing and when no de-icing salts are used; the air content in mortar is expected to be 7 per cent. A tolerance of $\pm 1\frac{1}{2}$ per cent is permitted on the values given in Table 11.3. Table 11.3 includes also the British requirements; these are less demanding than specified by ACI 201.2R-92.[11.92] On the other hand, Swedish requirements are similar to those of ACI 201.2R-92 but the tolerance permitted under very aggressive conditions is only ±1 per cent.[11.43]

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Table 11.3. Recommended Air Content of Concretes Containing Aggregates of Different Maximum Size

Maximum size of aggregate		*Recommended total air content of concrete (per cent) for level of exposure:*		
mm	*in.*	*ACI 201.2R-01*[11.92]		*British BS 8110 : 1 : 1997**
		Moderate	*Severe*	*Subject to de-icing salts*
9.5	$\frac{3}{8}$	6	$7\frac{1}{2}$	7
12.5	$\frac{1}{2}$	$5\frac{1}{2}$	7	—
14	—	—	—	6
19	$\frac{3}{4}$	5	6	5
25	1	5	6	—
37.5	$1\frac{1}{2}$	$4\frac{1}{2}$	$5\frac{1}{2}$	4
75	3	$3\frac{1}{2}$	$4\frac{1}{2}$	—
150	6	3	4	—

*replaced by BS EN 1992-1-1 : 2004

Some standards specifiy not only a maximum value of the bubble spacing but also a minimum value of the specific surface of the air in concrete so as to ensure the presence of small air bubbles. This gives the best protection from freezing and thawing, coupled with the least loss of strength due to the presence of voids in the concrete.

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Factors influencing air entrainment

The volume of air entrained in a given concrete is independent of the volume of entrapped air and depends primarily on the amount of air-entraining admixture added. The larger the quantity of the admixture the more air is entrained, but there is a maximum amount of any admixture beyond which there is no increase in the volume of voids.

To obtain a desired percentage of entrained air in concrete, for any given air-entraining admixture, there is a recommended dosage. However, the actual amount of air which becomes entrained is affected by a number of factors. Broadly speaking, for a given percentage of entrained air, more admixture is required under the following conditions:

- when the cement has a higher fineness;
- when the cement has a low alkali content;
- when fly ash is incorporated in the mix, the more so the higher the carbon content in the fly ash;

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when the aggregate has a high proportion of ultrafine material or when finely-divided pigments are used;

when the concrete temperature is high;

when the workability of the mix is low; and

when the mixing water is hard.

In connection with water, it can be mentioned that water used to wash truck mixers is very hard, especially if the mix used was air-entrained; the difficulty of entraining air is alleviated if the air-entraining admixture is not added with the wash water but with the additional clean water or with the sand.[11.95]

Mixes with high cement contents, about 500 kg/m^3 (840 lb/yd^3), and very low water/cement ratios (0.30 to 0.32), which are used in low-slump concrete overlays for bridge decks, require extremely high admixture dosages.[11.48]

Air entrainment can be used with various types of cement. However, there may be diffi-

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culties with mixes containing fly ash. The main reason for this is that carbon in the fly ash, arising from imperfect combustion, can absorb the surface-active air-entraining agent, thereby reducing its effectiveness.[11.38] In consequence, an increased dosage of the air-entraining admixture may be required but, if the active carbon content is not uniform, a variable air content may result.

In addition, it has sometimes been observed that properly entrained air can become destabilized in the presence of carbon particles in the fly ash. Thus, the air content of the mix decreases prior to placing. This may be due to the adsorption of the air bubbles on the highly active surface of the carbon particles.[11.38] Special air-entraining admixtures which contain a polar species, preferentially adsorbed by carbon, have been developed but they cannot remedy the difficulties unless there is no variation in the nature of the carbon.[11.38]

Air entrainment can be used when silica fume is incorporated in the mix; resistance to freezing

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and thawing is ensured by the usual spacing factor not larger than 200 μm.[11.35]

Air-entraining admixtures can be used when other admixtures are also included in the mix. When water-reducing admixtures are used at the same time as air-entraining admixtures, a lesser amount of the latter is frequently needed for a given percentage of air, even if the water-reducing admixture has no air-entraining properties *per se*. The explanation is that the physical or chemical environment is altered so as to permit the air-entraining admixture to operate more efficiently.[11.27] It should be noted that combinations of some admixtures may be incompatible, so that tests with the actual materials to be used should always be made. Indeed, trial mixes to determine the required dosage of any given air-entraining admixture are highly recommended.

Some superplasticizers, in combination with certain cements and air-entraining admixtures, may produce an unstable void system; it is, therefore, vital to check their compatibility.[11.44] Given

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such compatibility, satisfactory air entrainment of concrete containing a superplasticizer is possible, but there is usually a slight increase in the bubble size with a consequent increase in the bubble spacing factor.[11.52] For this reason, some increase in the dosage of the air-entraining admixture is required.[11.51] Nevertheless, at water/cement ratios below 0.4, concretes containing a superplasticizer exhibit good resistance to freezing and thawing when the spacing factor is somewhat larger than normally required, namely up to 240 μm.[11.100] Indeed, Canadian standards allow a maximum spacing factor of 230 μm.

The actual mixing operation also affects the resultant air content, and the loading sequence can have a significant effect. The cement should be well dispersed and the mix uniform before the air-entraining admixture is introduced.[11.46] If the mixing time is too short, the air-entraining admixture does not become sufficiently dispersed, but over-mixing gradually expels some air, so that there is an optimum value of mixing time.

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In practice, the mixing time is fixed from other considerations, usually at a value shorter than the minimum necessary for the admixture to become fully dispersed, and the amount of the air-entraining admixture must be adjusted accordingly. A very fast rotation of the mixer increases the amount of entrained air. Agitating up to 300 revolutions appears to lead only to a small loss of air (see Fig. 11.11)[11.28] but after 2 hours a loss of up to 20 per cent of the original air content can occur.[11.33] In some cases, a loss as high as 50 per cent was reported.[11.50]

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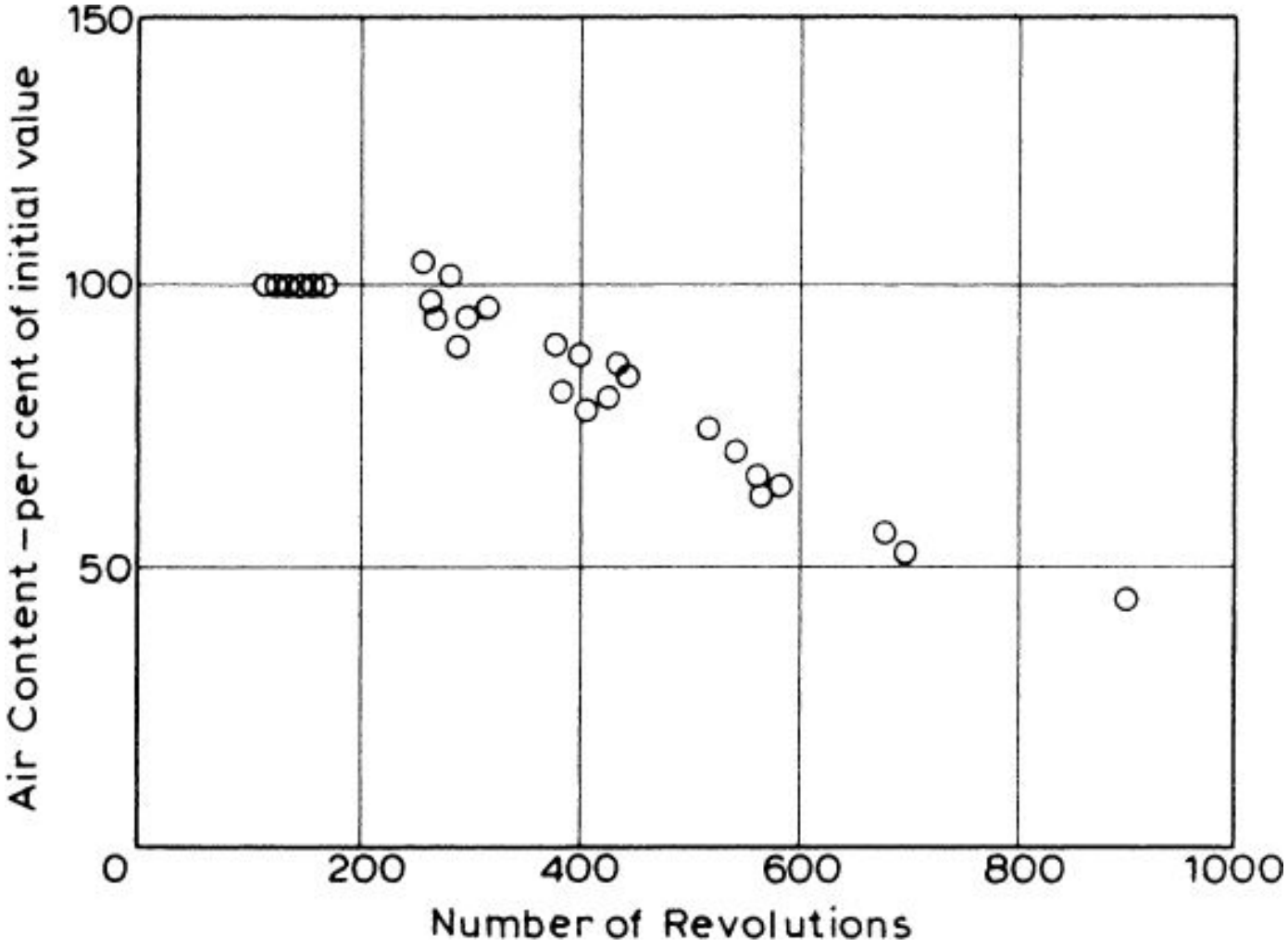


Fig. 11.11. Relation between air content and number of revolutions of the mixer. Batches of 6 m^3 (8 yd^3) were mixed at 18 rpm and agitated at 4 rpm[11.28]

Excessive finishing operations can result in a loss of entrained air from the surface zone of concrete, and it is this zone that is particularly vulnerable to freezing and thawing, as well as to the action of de-icing agents.

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Stability of entrained air

Assuring an adequate percentage of air in fresh concrete is not sufficient: the air voids must be stable so that they remain in position when the concrete hardens. Indeed, what is crucial is not the total air content but the spacing of the small air bubbles.

Three mechanisms of instability may operate.[11.42] In the first, during transporting and compacting of concrete, *large* bubbles move upwards by buoyancy (and also toward the side formwork) and are lost. This has little effect on the resistance to freezing and thawing, and can be even beneficial in that the loss of strength of concrete occasioned by the inclusion of voids is reduced.

The second mechanism involves the collapse of bubbles by pressure (arising from surface tension) which is largest in the *smallest* bubbles; the air becomes dissolved in the pore water. The loss of these bubbles has a detrimental effect on the resistance of concrete to freezing and thawing. It

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is likely that this mechanism of loss of the smallest bubbles is unavoidable and it explains the frequent absence of bubbles smaller than about 10 μm.[11.42]

The third mechanism consists of coalescence of small bubbles with larger ones, also in consequence of the relation between the solubility of air and the bubble size; the physics of this mechanism is rather complex.[11.42] The formation of larger bubbles, and therefore increased bubble spacing, is detrimental to resistance of concrete to freezing and thawing. Moreover, because the pressure in a larger bubble is smaller than in the original small bubble, the total volume of the coalesced bubble is larger. This can explain why, on occasion, the volume of entrained air in hardened concrete is higher than it was in the fresh concrete.[11.42] The increased total volume of air has a negative effect on the strength of concrete.

As far as the influence of cement on stability is concerned, it seems that stability increases with an increase in the content of alkalis in the ce-

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ment.[11.45] Silica fume, at least up to a 10 per cent content in the blended cement, does not affect the stability of the air-void system.[11.57]

In practice, loss of air occurs in transporting and during vibration of concrete: the loss is generally less than 1 percentage point, but slightly more in high-workability concrete. For the most part, it is the larger bubbles that are expelled, so that the effect on the resistance of concrete to freezing and thawing is small. Under normal conditions of pumping, the loss of air is between 1 and 1.5 percentage points.[11.54] However, a much larger loss can occur during pumping when a boom is used in a vertical position, so that the concrete in the pipeline can slide down under gravity: the air bubbles then expand but fail to re-form when the concrete leaves the pipeline. A remedy lies in providing resistance before discharge by an additional length of a horizontal flexible hose.[11.54]

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Because of the possible loss of air, air content should be determined on concrete as placed and not only at the point of discharge from the mixer; however, determination at the mixer may be of value as a means of batching control.

It may be noted that steam curing of air-entrained concrete may lead to incipient cracking because of the expansion of air.

Air entrainment by microspheres

The main difficulty with the use of air-entraining admixtures is that the air content of the concrete cannot be controlled directly: the quantity of the admixture is known but, as mentioned earlier, the actual air content in the hardened concrete and the spacing of air bubbles are affected by many factors. This difficulty is obviated if, instead of air bubbles, rigid-foam particles of suitable size are used. Such easily compressible hollow plastic microspheres (modelled on medication microcapsules) are manufactured.[11.29] They have a diameter of 10 to 60 μm (0.0004 to 0.002 in.), which

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is a narrower range of sizes than is the case with entrained air bubbles. In consequence, a lesser volume of microspheres can be used for the same protection from freezing and thawing, so that the loss of strength of concrete is smaller. Using 2.8 per cent of microspheres by volume of hardened cement paste gives a spacing factor of 70 μm (0.003 in.),[11.29] which is well below the value of 250 μm (0.01 in.) normally recommended with entrained air.

The specific gravity of the microspheres is 45 kg/m^3 (2.8 lb/ft^3) and they improve the workability of concrete to the same extent as entrained air, even though their total volume in the mix is smaller; the reason is that they are *all* small.

The microspheres are available pre-mixed with 90 per cent water in the form of a paste, and are stable except when concrete is over-mixed. They do not interact with other admixtures, but failure to perform in the presence of superplasticizers has been reported.[11.53] The main drawback

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of microspheres is their high cost so that their use is limited to special applications.

Use of highly porous particulate additives such as vermiculite, perlite or pumice,[11.49] although attractive when concrete is extruded or vacuum-dewatered, leads to a high loss of strength and is limited to high water/cement ratios.

Measurement of air content

There are three methods of measuring the *total* air content of fresh concrete. Because the entrained air cannot be distinguished in these tests from the large bubbles of accidental air, it is important that the concrete tested be properly compacted.

The *gravimetric method* is the oldest one. It relies simply on comparing the density of compacted concrete containing air, ρ_a, with the calculated density of air-free concrete of the same mix proportions, ρ. The air content, expressed as a percentage of the total volume of the concrete, is then $1 - \rho_a/\rho$. This method is covered by ASTM

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C 138-09 and can be used when the specific gravity of the aggregate and the mix proportions are constant. An error of 1 per cent in the calculated air content is not uncommon; this order of error would be expected from the simple experience of determining the density of nominally similar test specimens of non-air-entrained concrete.

In the *volumetric method* the difference in the volumes of a sample of compacted concrete before and after the air has been expelled is determined. The air is removed by agitating, inverting, rolling, and rocking, the operation being performed in a special two-part vessel. The details of the test are prescribed by ASTM C 173-10. The main difficulty lies in the fact that the mass of water replacing the air is small compared with the total mass of the concrete. The method is appropriate for concrete containing any type of aggregate.

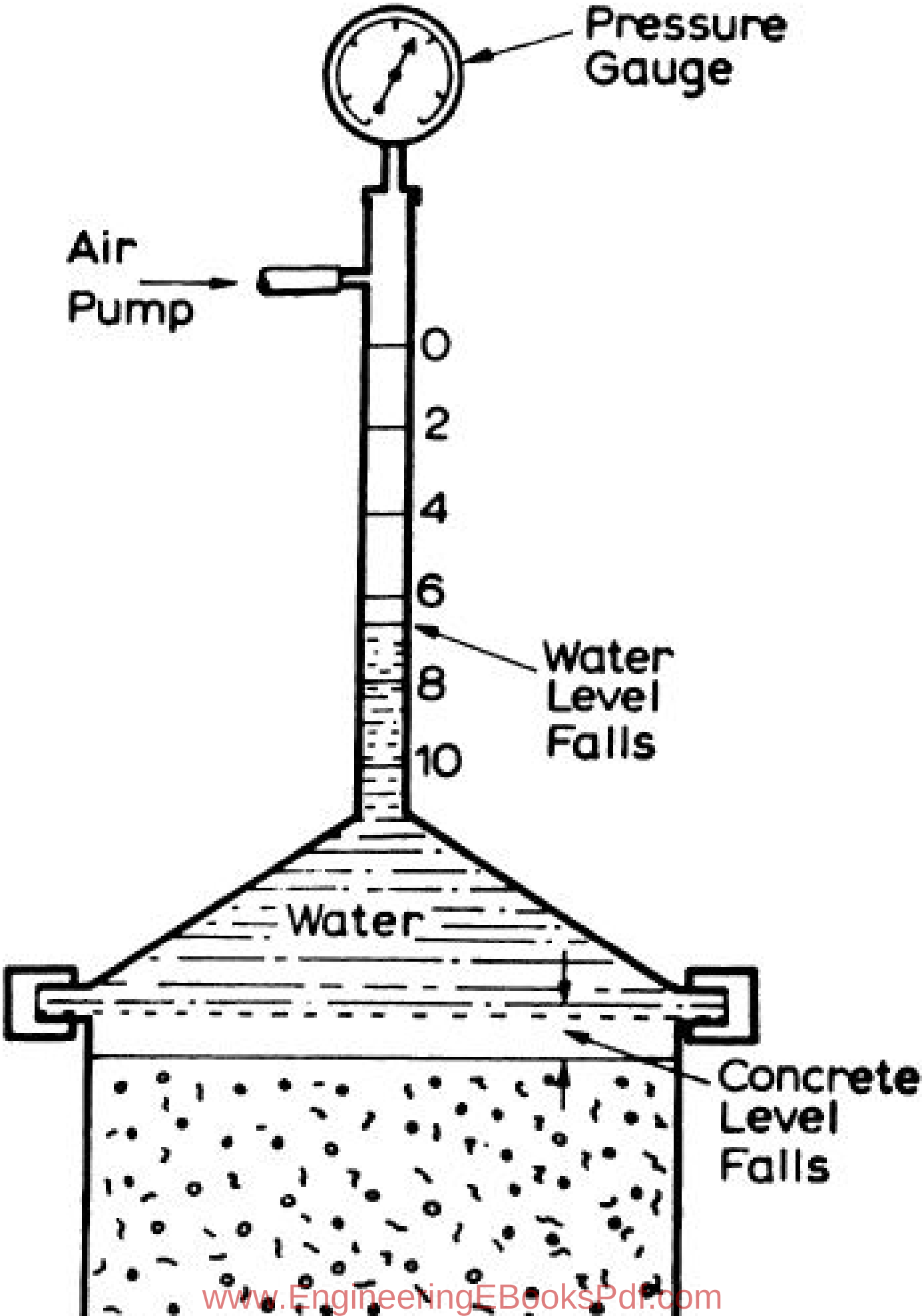
The most popular method, and one best suited for site use, is the *pressure method*. It is based on the relation between the volume of air and the ap-

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plied pressure (at a constant temperature) given by Boyle's law. The mix proportions or the properties of the materials need not be known and, when commercial air meters are used, no calculations are required as direct graduations in percentage of air are provided. However, at high altitudes, the pressure meter must be re-calibrated. The meter is not suitable for use with porous aggregates or with lightweight concrete.

A typical pressure-type air meter is shown in Fig. 11.12. The procedure consists essentially of observing the decrease in the volume of a sample of compacted concrete when subjected to a known pressure. The pressure is applied by a small pump, such as a bicycle pump, and measured by a pressure gauge. Due to the increase in pressure above atmospheric, the volume of air in the concrete decreases and this causes a fall in the level of the water above the concrete. By arranging the level of the water to vary within a calibrated tube, the air content can be read direct by an unskilled operator.

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Fig. 11.12. Pressure-type air meter

The test is covered by ASTM C 231-09 and BSEN 12350-7 : 2009, and provides the most dependable and accurate method of determining the air content of concrete.

The tests should be performed at the point of placing the concrete so as to exclude the air lost in transportation; preferably, concrete after compaction should be tested. It should be remembered that what is measured is the total volume of air in the concrete, and not just the entrained air with the desired air-void characteristics.

On the other hand, a detailed knowledge of the air-void system of *hardened* concrete can be obtained from polished sections of concrete by means of a microscope using the linear traverse technique[11.19] or a modified point-count method prescribed by ASTM C 457-10a.

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Tests of resistance of concrete to freezing and thawing

There exist no standard methods for the determination of the resistance of concrete to cycles of freezing and thawing such as may occur in service. However, ASTM C 666-03 (2008) prescribes two procedures for the determination of the resistance of concrete to rapidly repeated cycles of freezing and thawing; these procedures can be used to compare various mixes. In Procedure A, both freezing and thawing take place in water; in Procedure B, freezing takes place in air but thawing takes place in water. Freezing saturated concrete in water is much more severe than in air,[11.21] and the degree of saturation of the specimen at the beginning of the tests also affects the rate of deterioration. British Standard BS 5075 : 2 : 1982 also prescribes freezing in water.

The deterioration of concrete can be assessed in several ways. The most common method is to measure the change in the dynamic modulus of

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elasticity of the specimen, the reduction in the modulus after a number of cycles of freezing and thawing expressing the deterioration of the concrete. This method indicates damage before it has become apparent either visually or by other methods, although there are some doubts about this interpretation of the decrease in the modulus after the first few cycles of freezing and thawing.[11.20]

With the ASTM methods it is usual to continue freezing and thawing for 300 cycles or until the dynamic modulus of elasticity is reduced to 60 per cent of its original value, whichever occurs first. The durability can then be assessed as:

$$\text{durability factor} = \frac{\text{number of cycles at end of test} \times \text{percentage of original modulus}}{300}$$

There are no established criteria for acceptance or rejection of concrete in terms of the durability factor; its value is thus primarily in a comparison of different concretes, preferably when only one variable (e.g. aggregate) is changed. However, some guidance in interpretation can be obtained from the following: a factor smaller than 40

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means that the concrete is probably unsatisfactory with respect to resistance to freezing and thawing; 40 to 60 is the range for concretes with doubtful performance; above 60, the concrete is probably satisfactory; and around 100 it can be expected to be satisfactory.

The effects of freezing and thawing can also be assessed from measurements of the loss of compressive or flexural strength or from observations of the change in length[11.20] (used in ASTM C 666-03 (2008) and in BS 5075-2 : 1992) or in the mass of the specimen. A large change in length is an indication of internal cracking: a value of 200×10^{-6} for tests in water is taken to represent serious damage.[11.60]

Measurement of a decrease in the mass of the specimen is appropriate when damage takes place mainly at the surface of the specimen, but is not reliable in cases of internal failure; the results depend also on the size of the specimen. It may be noted that, if failure is primarily due to unsound aggregate, it is more rapid and more severe

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than when the hardened cement paste is disrupted first. It should be added that the tests of ASTM C 666-03 (2008) are useful in evaluating the possibility of the development of D-cracking due to the unsoundness of coarse aggregate.[11.36]

Another test method determined the dilation of concrete subjected to slow freezing, and was prescribed by ASTM C 671-94 (withdrawn); this is referred to on p. 544.

It can be seen that a number of tests and of means of assessing the results are available, and it is not surprising that the interpretation of test results is difficult. If the tests are to yield information indicative of the behaviour of concrete in practice, the test conditions must not be fundamentally different from the field conditions. A major difficulty lies in the fact that a test must be accelerated in comparison with the conditions of outdoor freezing, and it is not known at what stage acceleration affects the significance of the test results. One difference between the conditions in the laboratory and actual exposure lies in

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the fact that, in the latter case, there is seasonal drying during the summer months but, with permanent saturation imposed in some of the laboratory tests, all the air voids can eventually become saturated with a consequent failure of the concrete. Indeed, probably the most important factor influencing the resistance of concrete to cycles of freezing and thawing is the degree of its saturation,[11.58] and this may increase by prolonged accretion of ice during the freezing period; an example of such exposure occurs in Arctic waters. The duration of the freezing period in water is, therefore, of importance.

An important feature of the tests of ASTM C 666-03 (2008) is that cooling takes place at a rate of up to 11 °C/h (20 °F/h) whereas, in practice, 3 °C/h (5 °F/h) is more usual. The maximum rate of cooling of outdoor air in Europe was reported by Fagerlund[11.58] as 6 °C/h (11 °F/h). However, when radiation towards a clear sky can occur on a winter night, the surface temperature of concrete

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can cool at the rate of 12 °C/h (22 °F/h) even though the ambient air cools at 6 °C/h (11 °F/h).

The influence of the rate of freezing upon the resistance of concrete to cycles of freezing and thawing was demonstrated by Pigeon *et al.*;[11.59] as shown in Fig. 11.13, the higher the rate of freezing the smaller the spacing factor required for the protection of concrete.

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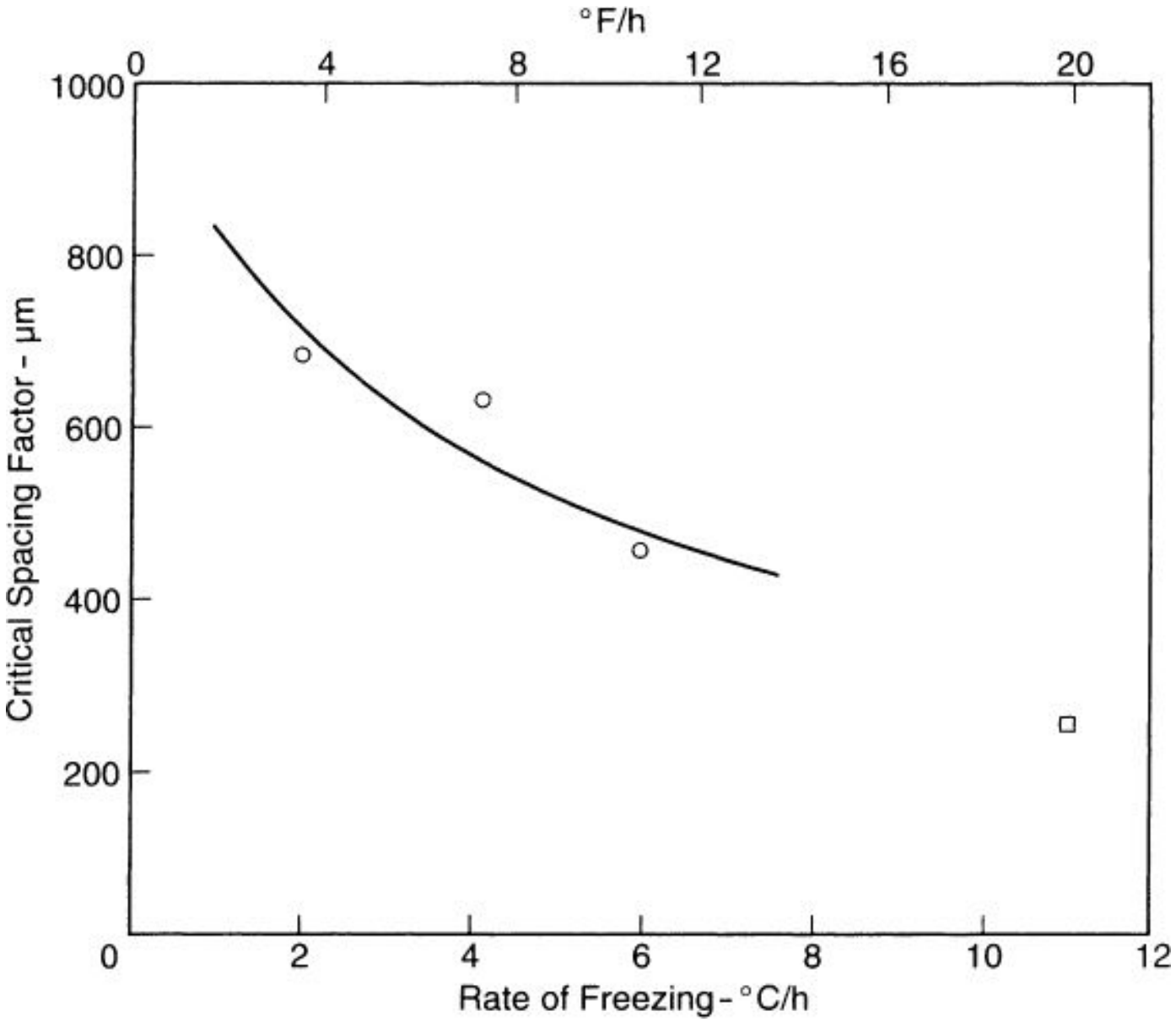


Fig. 11.13. Relation between the rate of freezing and the spacing factor required for the protection of concrete with a water/cement ratio of 0.5. The line represents data from ref. 11.59 and the points those from ref. 11.15

The vulnerability of concrete (with a water/cement ratio lower than 0.5) to freezing and thaw-

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ing in service depends on the degree of hydration of the cement paste: time is required for a dense pore structure to develop. The normal procedure of ASTM C 666-03 (2008) requires testing at the age of 14 days, which may be far too early. However, the test method provides for the choice of some other age.

It can be stated that some accelerated freezing and thawing tests result in the destruction of concrete that in practice could be satisfactory.[11.22] However, the ability of a concrete to withstand a considerable number of laboratory freezing and thawing cycles (say 150) is a probable indication of its high degree of durability under service conditions. The ASTM C 666-03 (2008) tests show, however, a high scatter in the middle range of durability. Whereas the numbers of cycles of freezing and thawing in a test and in actual concrete are not simply related, it may be interesting to note that, in much of the United States, there are more than 50 cycles per annum.

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The number of cycles of freezing and thawing to which a particular concrete element is exposed in service is not readily determined. The record of air temperature is inadequate. For example, the situation is complicated on a sunny day with passing clouds. The temperature of the surface of the concrete directly exposed to the sun can rise by 10 °C (18 °F) above the air temperature. When the sky clouds over, the concrete cools.[11.96] Thus, several cycles of freezing and thawing can occur in a day. These events are influenced by the angle of incidence of the solar radiation, so that a south-facing exposure may be most harmful. These rapid temperature changes at the surface of the concrete can also induce harmful temperature gradients.[11.96] It can be mentioned in passing that, in some northern locations, there is only one cycle of freezing and thawing per annum: its duration is six months.

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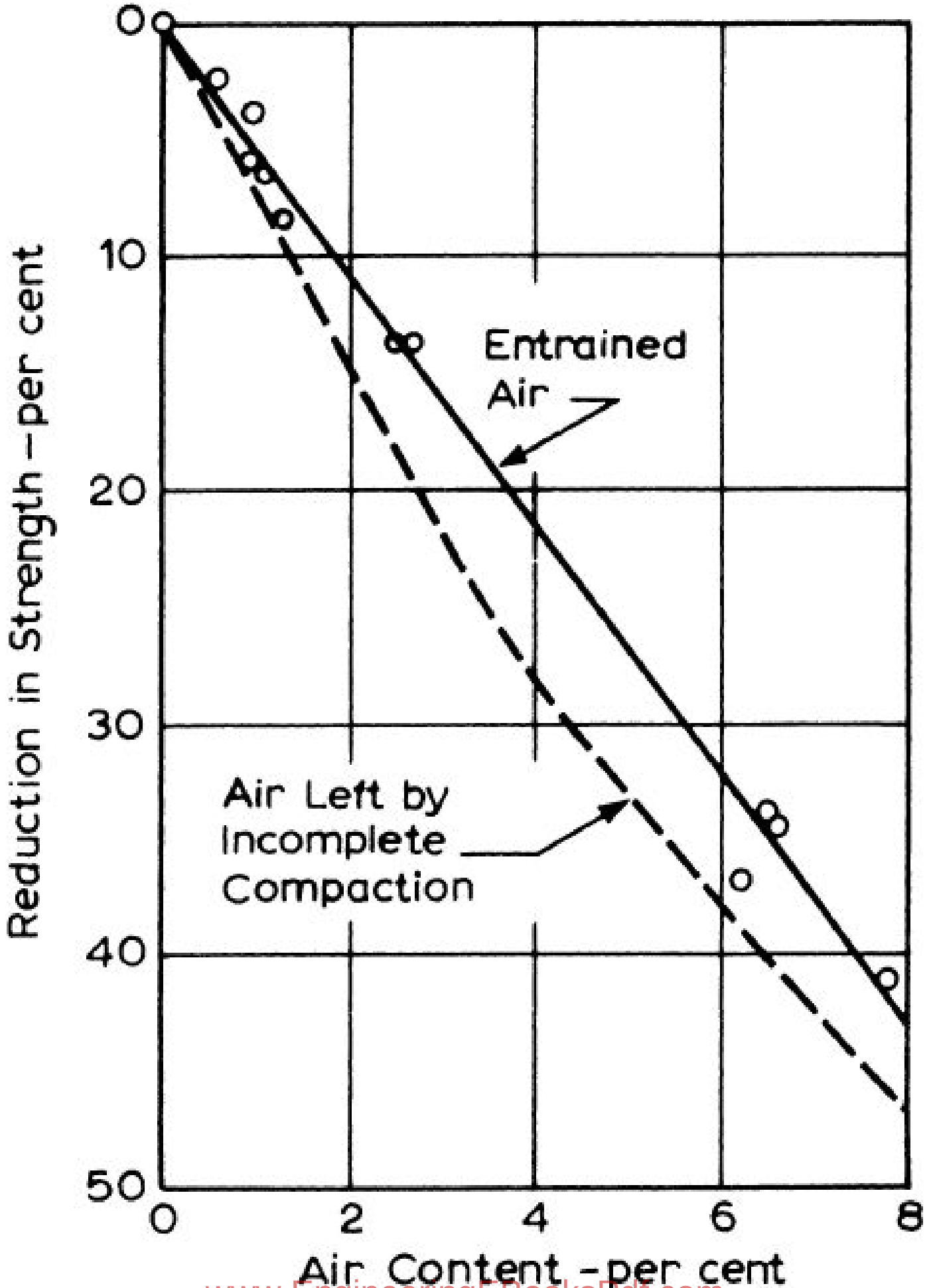
Further effects of air entrainment

The original purpose of air entrainment was to make concrete resistant to freezing and thawing. This is still the most common reason for incorporating entrained air in concrete, but there are some further effects of air entrainment on the properties of concrete, some beneficial, others not. One of the most important is the influence of voids on the strength of concrete at all ages. It will be remembered that the strength of concrete is a direct function of its density ratio, and voids caused by entrained air will affect the strength in the same way as voids of any other origin. Figure 11.14 shows that when entrained air is added to a mix, without any other change in the mix proportions being made, the decrease in the strength of concrete is proportional to the volume of air present. The range considered is up to 8 per cent of air and this is why the curved part of the strength–void ratio relation is not apparent (cf. Fig. 4.1). That the origin of the air is irrelevant is apparent from the dotted curve in Fig.

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11.14 which shows the strength–air content relation for the case when the voids are due to inadequate compaction, as well as when they are due to entrainment. The range of tests covered mixes with water/cement ratios between 0.45 and 0.72, and this shows that the loss of strength expressed as a fraction of the strength of air-free concrete is independent of the mix proportions. The average loss of compressive strength is 5.5 per cent for each percentage point of air present.[11.18] The effect on flexural strength is much smaller. The relation between the volume of voids in concrete and loss of strength was confirmed by Whiting *et al.*[11.55]

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Fig. 11.14. Effect of entrained and accidental air on the strength of concrete[11.18]

It should be noted that strength is affected by the total volume of all the voids present: entrapped air, entrained air, capillary pores, and gel pores. When entrained air is present in the concrete, the total volume of capillary pores is smaller because a part of the gross volume of the hardened cement paste consists of entrained air. This is not a negligible factor because the volume of entrained air represents a significant proportion of the gross volume of the hardened cement paste. For instance, in a 1:3.4:4.2 mix with a water/cement ratio of 0.80, the capillary pores at the age 7 days were found to occupy 13.1 per cent of the volume of concrete. With entrained air in a mix of the same workability (1:3.0:4.2 with a water/cement ratio of 0.68), the capillary pores occupied 10.7 per cent, but the volume of air (entrained and entrapped) was 6.8 per cent (compared with 2.3 per cent in the former mix).[11.24]

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This is one reason why air entrainment does not cause as large a loss of strength as might be expected. But a more important reason is that the entrainment of air has a considerable beneficial effect on the workability of the concrete. As a result, in order to keep the workability constant, the addition of entrained air can be accompanied by a reduction in the water/cement ratio, compared with a similar mix without entrained air. For very lean mixes, say, with an aggregate/cement ratio of 8 or more, and particularly when angular aggregate is used, the improvement in workability due to air entrainment is such that the resultant decrease in the water/cement ratio compensates fully for the loss of strength due to the presence of the voids. In the case of massive structures, where the development of heat of hydration of cement, and not strength, is often of primary importance, air entrainment permits the use of mixes with low cement contents and, therefore, a low temperature rise. In richer mixes, the effect of air entrainment on workability is smaller, so that the water/ce-

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ment ratio can be lowered only a little, and there is a net loss in strength. In general terms, entrainment of 5 per cent of air increases the compacting factor of concrete by about 0.03 to 0.07, and the slump by 15 to 50 mm ($\frac{1}{2}$ to 2 in.),[11.18] but actual values vary with the properties of the mix. Air entrainment is also effective in improving the workability of the rather harsh mixes made with lightweight aggregate.

The reason for the improvement of workability by the entrained air is probably that the air bubbles, kept spherical by surface tension, act as a fine aggregate of very low surface friction and considerable elasticity. Entraining air in the mix makes it actually behave like an over-sanded mix and, for this reason, the addition of entrained air should be accompanied by a reduction in the sand content. The latter change allows a further reduction in the water content of the mix, i.e. a further compensation of the loss of strength due to the presence of voids is possible.

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It is interesting to note that air entrainment affects the consistency or 'mobility' of the mix in a qualitative manner: the mix can be said to be more 'plastic', so that for the *same* workability, as measured, say, by the compacting factor, the mix containing entrained air is easier to place and compact than an air-free mix.

The presence of entrained air is also beneficial in reducing bleeding: the air bubbles appear to keep the solid particles in suspension so that sedimentation is reduced and water is not expelled. For this reason, permeability and the formation of laitance are also reduced, and this results in an improved resistance to freezing and thawing of the top layer of a slab or a lift. This is relevant to the beneficial effect of air entrainment on the destructive action of de-icing agents. Air entrainment reduces segregation during handling and transporting as the mix is more cohesive, but segregation due to over-vibration is still possible, particularly as, under those conditions, the air bubbles are expelled.

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The addition of entrained air lowers the density of the concrete and makes cement and aggregate 'go further'. This offers an economic advantage but is offset by the cost of the air-entraining admixture and the associated operations.

Effects of de-icing agents

Horizontal surfaces, such as road slabs and bridge decks which are subjected to freezing and thawing, are often also treated by de-icing agents for the purpose of removing snow and ice. These agents have an adverse effect on concrete, leading to surface scaling and sometimes to corrosion of reinforcement. The latter topic is dealt with later in this chapter.

The salts commonly used are NaCl and $CaCl_2$, the latter being more expensive. The salts produce osmotic pressure and cause movement of water toward the top layer of the slab where freezing takes place,[11.4] and hydraulic pressure is developed.[11.92] Thus, the action is similar to ordinary freezing and thawing, but is more severe.

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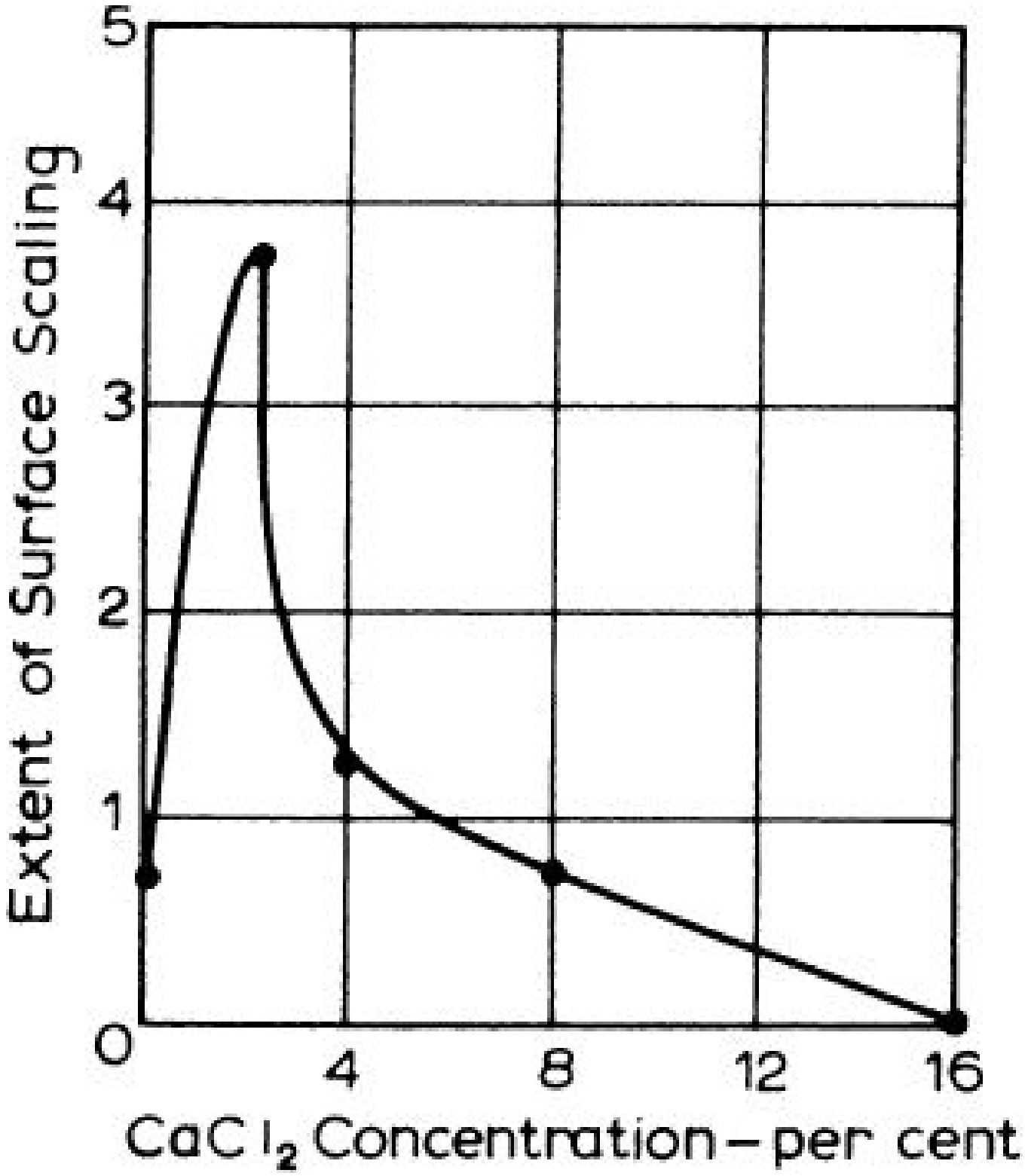
Indeed, the damage caused by the de-icing agents is primarily physical, and not chemical, in nature[11.13] and is independent of whether the de-icer is organic or not, or is a salt or not.[11.31] However, there is also some possibility of leaching of $Ca(OH)_2$ which has a greater solubility in a chloride solution than in water;[11.32] it is also possible for chloroaluminates to form under wetting and drying.[11.32]

Mather[11.30] suggested the following sequence. The de-icing agent melts the snow or ice, the resulting water being often ponded by adjacent ice. The water is actually a salt solution and, therefore, has a lowered freezing point. Some of this solution is absorbed by the concrete which may become saturated. As more ice melts, the melt water becomes diluted until its freezing point rises to near the freezing point of water. Freezing then occurs again. Thus, freezing and thawing occur as often as without the use of de-icing agents, or even more often because a potentially insu-

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lating layer of ice has been destroyed. In consequence, de-icing agents can be said to increase saturation, and possibly also to increase the number of cycles of freezing and thawing. An indirect confirmation of this behaviour is offered by the fact that the greatest damage occurs when concrete is exposed to relatively low concentrations of salts (2 to 4 per cent solution)[11.13] (Fig. 11.15).

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Fig. 11.15. Effect of concentration of $CaCl_2$ on scaling of non-air-entrained concrete after 50 cycles of freezing and thawing (without removal of the solution).[11.13] The extent of surface scaling is rated from 0 = no scaling to 5 = severe scaling

An additional factor contributing to the damage of concrete is the sudden drop in temperature of the subsurface concrete when ice melts and extracts the latent heat; this is a form of thermal shock which can result in very rapid freezing.

Air entrainment makes concrete very much more resistant to surface scaling in the same way as it provides resistance to freezing and thawing without the use of de-icing agents. The concrete should have a water/cement ratio not higher than 0.40 and a cement content of at least 310 kg/m^3 (520 lb/yd^3).[11.56] High-strength concrete shows very good resistance to scaling.[11.61]

Numerous tests on salt scaling have shown that the extent of damage is sensitive to the pro-

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cedure adopted. For instance, air drying of the concrete after wet curing but prior to exposure cycles, increases the resistance to surface scaling.[11.31] The drying out must, however, be preceded by moist curing of sufficient duration for the cement paste to hydrate extensively. Therefore, in practice, concreting should take place at a time of the year such that good curing can be applied, followed by a period of drying out. Excessive bleeding and laitance must be avoided.

The most severe damage occurs when concrete is subjected to alternating freezing and thawing with the de-icer solution remaining on top of the specimen, rather than being replaced with fresh water prior to each re-freezing.[11.13] On the other hand, if the liquid is removed from the surface of the concrete prior to re-freezing, no scaling takes place, even with non-air-entrained concrete.[11.13]

The resistance of concrete to de-icing agents can be ascertained by the test method of ASTM C 672-03 in which specimens are subjected to

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cycles of freezing when covered with a calcium chloride solution followed by thawing in air. The assessment of scaling is made visually.

Because chlorides which penetrate to the reinforcing steel lead to corrosion, the use of chloride-free de-icing agents is desirable. One of these is urea, which, however, pollutes water and is less effective in removing ice. Calcium magnesium acetate is effective, albeit slow to act, but is very expensive.

Some protection of concrete from the deleterious action of de-icing agents can be obtained by sealing it with linseed oil. Boiled linseed oil, diluted in equal parts with kerosene or mineral spirits, is applied to the surface of concrete, which must be dry, two coats being used. The oil slows down the ingress of the de-icer solution but does not seal the surface of the concrete so as to prevent evaporation. The linseed oil darkens the colour of concrete, and non-uniform application may produce an unsightly surface. Re-sealing after a

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few years is necessary. Silane and siloxane can also be used, but this is a specialized topic.

Chloride attack*

*The sections on chloride attack of reinforced concrete were substantially published in ref. 11.37.

Chloride attack is distinct in that the primary action is the corrosion of steel reinforcement, and it is only as a consequence of this corrosion that the surrounding concrete is damaged. Corrosion of reinforcement is one of the major causes of deterioration of reinforced concrete structures in many locations. The broad topic of corrosion of steel, as well as of other metals, embedded in concrete (see ACI 222R-89)[11.82] is outside the scope of the present book, and it is intended to limit this discussion to the consideration of those properties of concrete which influence corrosion, with emphasis on the transport of chloride ions

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through the concrete in the cover to the reinforcement.

Nevertheless, a brief description of the mechanism of chloride-induced corrosion will be helpful in understanding the processes involved.

Mechanism of chloride-induced corrosion

The protective passivity layer on the surface of embedded steel was mentioned on p. 499. This layer, which is self-generated soon after the hydration of cement has started, consists of γ-Fe_2O_3 tightly adhering to the steel. As long as that oxide film is present, the steel remains intact. However, chloride ions destroy the film and, in the presence of water and oxygen, corrosion occurs. Chloride ions were described by Verbeck[11.63] as "a specific and unique destroyer".

It may be useful to add that, provided the surface of the reinforcing steel is free from loose rust (a condition which is always specified), the presence of rust at the time when the steel is embedded in concrete does not influence corrosion.[11.78]

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A brief description of the corrosion phenomenon is as follows. When there exists a difference in electrical potential along the steel in concrete, an electrochemical cell is set up: there form anodic and cathodic regions, connected by the electrolyte in the form of the pore water in the hardened cement paste. The positively charged ferrous ions Fe^{++} at the anode pass into solution while the negatively charged free electrons e^{-} pass through the steel into the cathode where they are absorbed by the constituents of the electrolyte and combine with water and oxygen to form hydroxyl ions $(OH)^{-}$. These travel through the electrolyte and combine with the ferrous ions to form ferric hydroxide which is converted by further oxidation to rust (see Fig. 11.16). The reactions involved are as follows:

anodic reactions:

$$Fe \rightarrow Fe^{++} + 2e^{-}$$

$$Fe^{++} + 2(OH)^{-} \rightarrow Fe(OH)_2 \quad \text{(ferrous hydroxide)}$$

$$4Fe(OH)_2 + 2H_2O + O_2 \rightarrow 4Fe(OH)_3 \quad \text{(ferric hydroxide)}$$

cathodic reaction:

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$$4e^- + O_2 + 2H_2O \rightarrow 4(OH)^-.$$

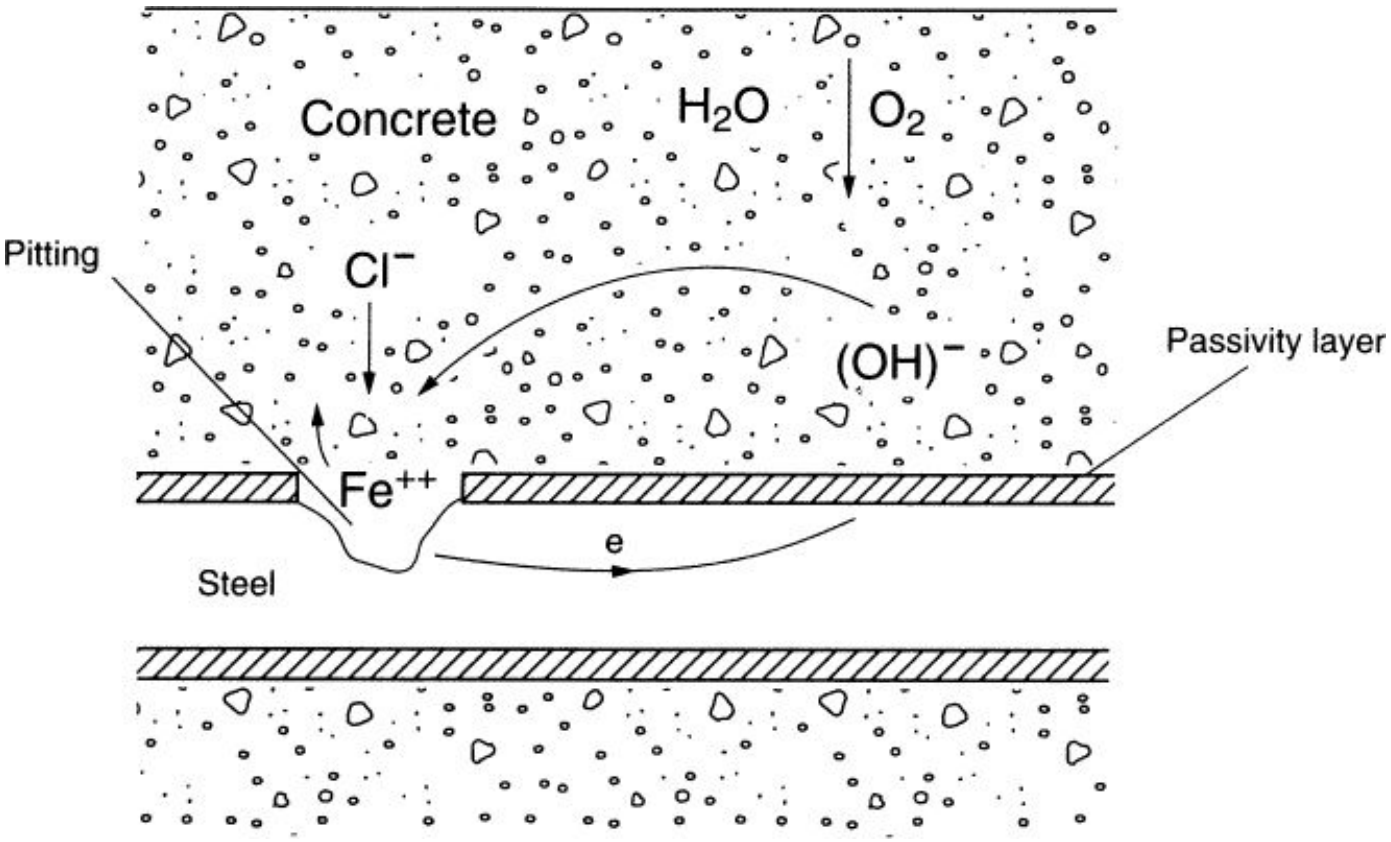


Fig. 11.16. Schematic representation of electro-chemical corrosion in the presence of chlorides

It can be seen that oxygen is consumed and water is regenerated but it is needed for the process to continue. Thus, there is no corrosion in, dry concrete, probably below a relative humidity of 60 per cent; nor is there corrosion in concrete fully immersed in water, except when water can

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entrain air, for example by wave action. The optimum relative humidity for corrosion is 70 to 80 per cent. At higher relative humidities, the diffusion of oxygen through the concrete is considerably reduced.

The differences in electrochemical potential can arise from differences in the environment of the concrete, for example when a part of it is permanently submerged in sea water and a part is exposed to periodic wetting and drying. A similar situation can arise when there is a substantial difference in the thickness of cover to a steel system which is electrically connected. Electrochemical cells form also due to a variation in salt concentration in the pore water or due to a non-uniform access to oxygen.

For corrosion to be initiated, the passivity layer must be penetrated. Chloride ions activate the surface of the steel to form an anode, the passivated surface being the cathode. The reactions involved are as follows:

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$$Fe^{++} + 2Cl^{-} \rightarrow FeCl_2$$
$$FeCl_2 + 2H_2O \rightarrow Fe(OH)_2 + 2HCl.$$

Thus, Cl^{-} is regenerated so that the rust contains no chloride, although ferrous chloride is formed at the intermediate stage.

Because the electrochemical cell requires a connection between the anode and the cathode by the pore water, as well as by the reinforcing steel itself, the pore system in hardened cement paste is a major factor influencing corrosion. In electrical terms, it is the resistance of the 'connection' through the concrete that controls the flow of the current. The electrical resistivity of concrete is greatly influenced by its moisture content, by the ionic composition of the pore water, and by the continuity of the pore system in the hardened cement paste.

There are two consequences of corrosion of steel. First, the products of corrosion occupy a volume several times larger than the original steel so that their formation results in cracking (char-

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acteristically parallel to the reinforcement), spalling or in delamination of concrete (see Fig. 11.17). This makes it easier for aggressive agents to ingress toward the steel, with a consequent increase in the rate of corrosion. Second, the progress of corrosion at the anode reduces the cross-sectional area of the steel, thus reducing its load-carrying capacity. In this connection, it should be pointed out that chloride-induced corrosion is highly localized at a small anode, pitting of the steel taking place.

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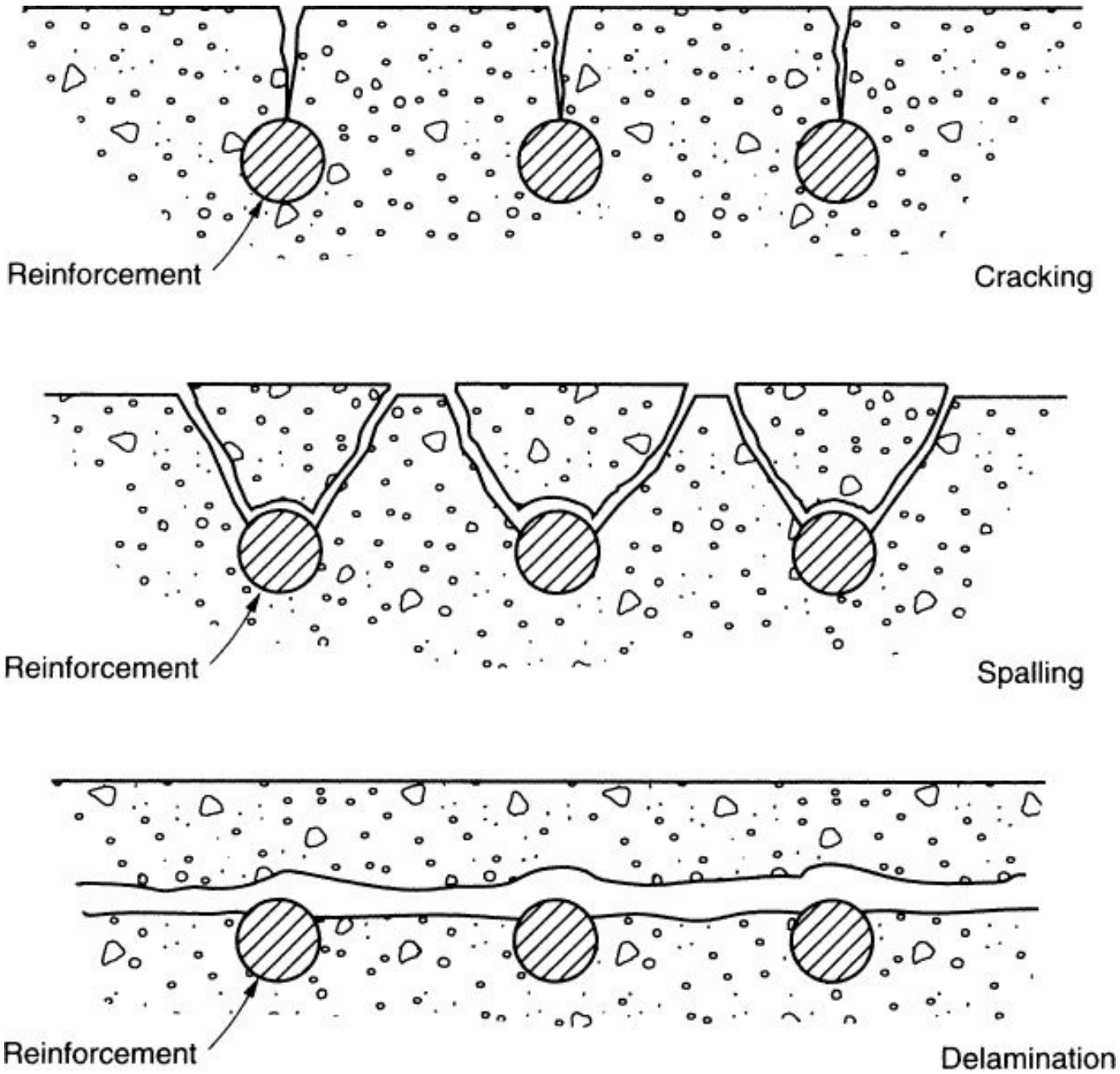


Fig. 11.17. Diagrammatic representation of damage induced by corrosion: cracking, spalling, and delamination

When the supply of oxygen is severely limited, corrosion at a slow rate can occur. The

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products of corrosion, which are less voluminous than under normal circumstances, may travel into voids in the concrete without a progressive development of cracking or spalling.

Chlorides in the mix

Chlorides can be present in concrete because they have been incorporated in the mix through the use of contaminated aggregate or of sea water or brackish water, or by admixtures containing chlorides. None of these materials should be permitted in reinforced concrete, and standards generally prescribe strict limits on the chloride content of the concrete from all sources. For example, BS 8110-1 : 1997 limits the *total* chloride-ion content in reinforced concrete to 0.40 per cent by mass of cement. The same limit is prescribed by European Standard BS EN 206-1 : 2000 (now BS EN 1992-1 : 2004). The approach of ACI 318-02[11.56] is to consider *water-soluble* chloride ions only. On that basis, the chloride-ion content of reinforced concrete is limited to 0.15

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per cent by mass of cement. The two values are not substantially different from one another because water-soluble chlorides are only a part of the total chloride content, namely, the free chlorides in pore water. The distinction between *free* and *bound* chlorides is considered on p. 571, but, at this stage, it can be noted that the total chloride content is determined as the *acid-soluble* chloride content, using ASTM C 1152-04 or BS 1881-124 : 1988. In the presence of some admixtures, potentiometric titration gives a higher value of chloride content than reliance on colour change. There exist several techniques for the determination of the content of water-soluble chlorides.

As a possible source of chlorides in the mix, Portland cement itself contains only a very small amount: normally, no more than 0.01 per cent by mass. However, ground granulated blastfurnace slag may have a significant chloride content if its processing involved quenching with sea water.[11.92] Drinking water may well contain 250 ppm of chloride ions; at a water/cement ratio of

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0.4, the water would contribute the same amount of chloride ions as Portland cement. As far as aggregate is concerned, BS 882 : 1992 (withdrawn) gives guidance on the maximum total chloride ion content; compliance with this guidance is likely to satisfy the requirements for concrete of BS 5328-1 : 1997 (withdrawn) and of BS 8110-1 : 1997 (now Eurocode 2 : 2004). For reinforced concrete, the chloride content of the aggregate should not exceed 0.05 per cent by mass of the total aggregate; this is reduced to 0.03 per cent when sulfate-resisting cement is used. For prestressed concrete, the corresponding figure is 0.01 per cent. Limits on impurities in water are given in BS EN 1008 : 2002 and ASTM C 1602-06.

The various limits on chlorides referred to in this section are generally conservative so that compliance with them should ensure no chloride-induced corrosion unless more chlorides ingress into the concrete in service. The view that the limits are conservative is disputed by Pfeifer.[11.40]

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Ingress of chlorides

The problem of chloride attack arises usually when chloride ions ingress from outside. This can be caused by de-icing salts – a topic discussed on p. 563. Another, particularly important, source of chloride ions is sea water in contact with concrete. Chlorides can also be deposited on the surface of concrete in the form of air-borne very fine droplets of sea water (raised from the sea by turbulence and carried by wind) or of air-borne dust which subsequently becomes wetted by dew. It is useful to point out that air-borne chlorides can travel substantial distances: 2 km has been reported,[11.75] but travel over even greater distances is possible, depending on wind and topography. The configuration of structures also affects the movement of air-borne salts: when eddies occur in the air, salts can reach the landward faces of structures.

Brackish groundwater in contact with concrete is also a source of chlorides.

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Although this is a rare occurrence, it may be mentioned that chlorides can ingress into concrete from conflagration of organic materials containing chlorides. Hydrochloric acid is formed and deposited on the surface of concrete where it reacts with calcium ions in the pore water. Ingress of chloride ions can follow.[11.83]

Whatever their external origin, chlorides penetrate concrete by transport of water containing the chlorides, as well as by diffusion of the ions in the water, and by absorption. Prolonged or repeated ingress can, with time, result in a high concentration of chloride ions at the surface of the reinforcing steel.

When concrete is permanently submerged, chlorides ingress to a considerable depth but, unless oxygen is present at the cathode, there will be no corrosion. In concrete which is sometimes exposed to sea water and is sometimes dry, the ingress of chlorides is progressive. The following is a description of a situation often found in structures on the coast in a hot climate.

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Dry concrete imbibes salt water by absorption and, under some conditions, may continue to do so until the concrete has become saturated. If the external conditions then change to dry, the direction of movement of water becomes reversed and water evaporates from the ends of capillary pores open to the ambient air. It is, however, only pure water that evaporates, the salts being left behind. Thus, the concentration of salts in the water still in the concrete increases near the surface of the concrete. The concentration gradient thus established drives the salts in the water near the surface of the concrete towards the zones of lower concentration, i.e. inwards; this is transport by diffusion. Depending on the external relative humidity and on the duration of the drying period, it is possible for most of the water in the outer zone of the concrete to evaporate so that the water remaining in the interior will become saturated with salt and the excess salt will precipitate out as crystals.

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It can be seen thus that, in effect, the water moves outwards and the salt inwards. The next cycle of wetting with salt water will bring more salt present in solution into the capillary pores. The concentration gradient now decreases outwards from a peak value at a certain depth from the surface, and some salts may diffuse toward the surface of the concrete. If, however, the wetting period is short and drying restarts quickly, the ingress of salt water will carry the salts well into the interior of the concrete; subsequent drying will remove pure water, leaving the salts behind.

The exact extent of the movement of salt depends on the length of the wetting and drying periods. It may be recalled that wetting of concrete occurs very rapidly and drying is very much slower; the interior of the concrete never dries out. It should also be noted that the diffusion of ions during the wet periods is fairly slow.

It is apparent thus that a progressive ingress of salts toward the reinforcing steel takes place un-

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der alternating wetting and drying, and a chloride profile of the kind shown in Fig. 11.18 is established. The profile is determined by chemical analysis of dust samples obtained by incremental drilling to various depths from the surface. Sometimes, there is a lower concentration of chlorides in the outermost 5 mm ($\frac{1}{4}$ in.) or so of the concrete where rapid movement of water takes place so that the salts are quickly carried a small distance inwards. The maximum chloride ion content in pore water can be in excess of the concentration in sea water; this was observed after 10 years' exposure.[11.71] The crucial fact is that, with the passage of time, a sufficient amount of chloride ions will reach the surface of the reinforcing steel. What constitutes a 'sufficient' amount will be discussed in the subequent section.

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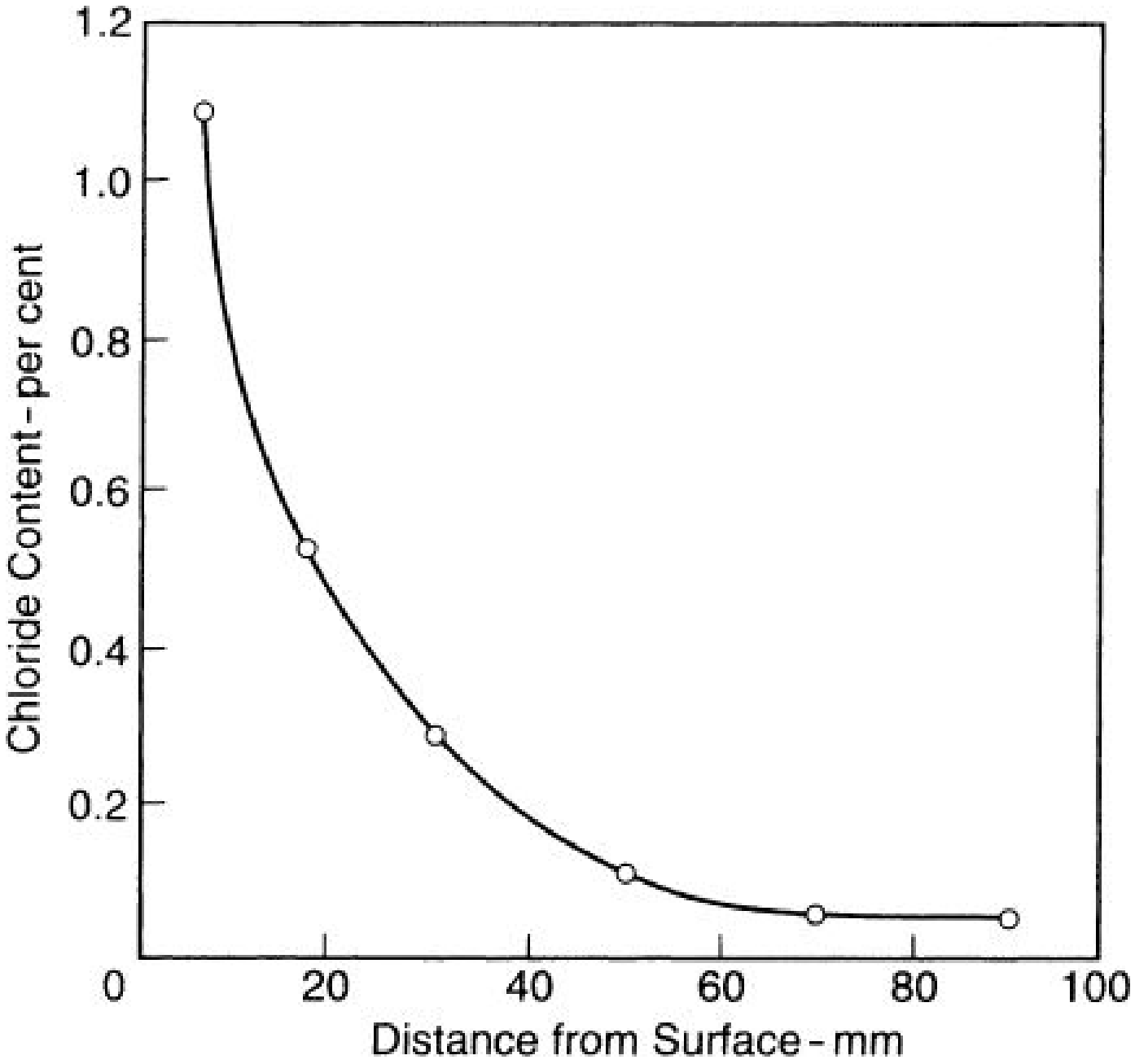


Fig. 11.18. An example of the profile of total chloride ion content as a percentage of the mass of cement; points show averages over 10 or 20 mm increments

As just mentioned, the ingress of chlorides into concrete is strongly influenced by the exact se-

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quence of wetting and drying. This sequence varies from location to location, depending on the movement of the sea and on the wind, on exposure to the sun, and on the usage of the structure. Thus, even different parts of the same structure may undergo a different pattern of wetting and drying; this explains why, sometimes, there is a considerable variation in the extent of corrosion damage in a single structure.

It is not only wetting and drying of the surface zone of the concrete that influences the ingress of chlorides; drying to a greater depth allows subsequent wetting to carry the chlorides well into the concrete, thus speeding up the ingress of chloride ions. For this reason, concrete in the tidal zone (where the period of drying is short) is less vulnerable to corrosion than concrete in the splash zone (where wetting may occur only when the sea is high or the wind is strong). The most vulnerable is the concrete wetted by sea water only occasionally, such as areas around bollards (where wet ropes are coiled) or in the vicinity of fire

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hydrants (using sea water), or in industrial areas subjected to periodic washdown with sea water, but at other times exposed to the drying effects of the sun and of a high temperature.

Threshold content of chloride ions

It was mentioned earlier that, for corrosion to be initiated, there has to be a certain minimum concentration of chloride ions at the surface of the steel. However, no universally valid threshold concentration exists. As far as chlorides incorporated in the original mix are concerned, the threshold concentration was considered on p. 568. It is useful to add that the presence of a given excessive amount of chlorides in the original mix results in a more aggressive action, and therefore a higher corrosion rate, than when the same amount of chlorides has ingressed into the concrete in service.[11.64]

As far as chlorides which have ingressed into the concrete are concerned, it is even more difficult to establish a threshold concentration of

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chloride ions below which there is no corrosion. This threshold depends on a number of factors, many of which are still imperfectly understood. Moreover, the distribution of chlorides within the hardened cement paste is not uniform, as found in chloride profiles in actual structures. For practical purposes, prevention of corrosion lies in controlling the ingress of chlorides by the thickness of cover to reinforcement and by the penetrability of the concrete in the cover.

While, under any given circumstances, there may be a threshold chloride content for corrosion to be initiated, its progress depends on the resistivity of the hardened cement paste, which varies with humidity, and on the availability of oxygen, which is affected by the immersion of concrete.

In any case, it is not the total chloride content that is relevant to corrosion. A part of the chlorides are chemically bound, being incorporated in the products of hydration of cement. Another part of the chlorides are physically bound, being ad-

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sorbed on the surface of the gel pores. It is only the third part of the chlorides, namely, free chlorides, that are available for the aggressive reaction with steel. However, the distribution of the chloride ions among the three forms is not permanent as there is an equilibrium situation such that some free chloride ions are always present in the pore water. It follows that only the chloride ions in excess of those needed for this equilibrium can become bound.

Binding of chloride ions

The main form of binding of the chloride ions is by reaction with C_3A to form calcium chloroaluminate, $3CaO.Al_2O_3.CaCl_2.10H_2O$, sometimes referred to as *Friedel's salt*. A similar reaction with C_4AF results in calcium chloroferrite, $3CaO.Fe_2O_3.CaCl_2.10H_2O$. It follows that more chloride ions are bound when the C_3A content of the cement is higher, and also when the cement content of the mix is higher. For this reason, it used to be thought that cements with a high C_3A

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content are conducive to good resistance to corrosion.

This may be true when chloride ions are present at the time of mixing (a situation which should not be permitted) because they can rapidly react with C_3A. However, when chloride ions ingress into concrete, a smaller amount of chloroaluminates is formed and, under some future circumstances, they may become dissociated, releasing chloride ions so as to replenish those removed from the pore water by transport to the surface of the steel.

A further factor in deciding on the desirable C_3A content of the cement is the possibility of sulfate attack on some parts of the given structure, other than those subject to the ingress of sea water. As mentioned on p. 76, sulfate resistance requires a low C_3A content in the cement. For these various reasons, it is nowadays thought that a moderately sulfate-resisting cement, Type II, offers the best compromise.

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In the case of cements containing ground granulated blastfurnace slag, it has been suggested that binding of chlorides takes place also by the aluminates in the slag, but this has not been fully confirmed.[11.91]

In connection with a possible use of cement with a high C_3A content, it should be remembered that a high C_3A content results in a higher early rate of heat evolution, and therefore a temperature rise. This behaviour can be harmful in moderately large concrete masses often associated with structures exposed to the sea.[11.88]

Some standards, for example BS 8110-1 : 1985 (replaced by Eurocode 2-2004), severely limit the chloride content when sulfate-resisting cement (Type V) is used, on the assumption that chlorides adversely affect sulfate resistance. This has now been proven not to be the case.[11.76] What happens is that sulfate attack results in a decomposition of calcium chloroaluminate, thus making

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some chloride ions available for corrosion; calcium sulfoaluminate is formed.[11.79]

Carbonation of hardened cement paste in which bound chlorides are present has a similar effect of freeing the bound chlorides and thus increasing the risk of corrosion. Ho and Lewis[11.80] cite Tuutti as having found an increased concentration of chloride ions in pore water to occur 15 mm in advance of the carbonation front. This harmful effect of carbonation is in addition to the lowering of the pH value of the pore water, so that severe corrosion may well follow. It has also been found in laboratory tests[11.85] that the presence of even a small amount of chlorides in carbonated concrete enhances the rate of corrosion induced by the low alkalinity of carbonated concrete.

In considering both carbonation and ingress of chloride ions, it is important to remember that the optimum relative humidity for carbonation is between 50 and 70 per cent, whereas corrosion progresses rapidly only at higher humidities. The

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occurrence of both of these relative humidities, one after another, is possible when concrete is exposed to long periods of alternating wetting and drying. Another occurrence of both chloride ingress and carbonation was observed in thin cladding panels of a building: air-borne chlorides ingressed from outside and reached the reinforcing steel; carbonation progressed from the relatively dry inside of the building.

Returning to the topic of the chloride ion concentration present in the pore water in an equilibrium situation, it should be noted that the chloride ion concentration depends on the other ions present in the pore water; for example, at a given total chloride ion content, the higher the hydroxyl (OH^-) concentration the more free chloride ions are present.[11.66] For this reason, the Cl^-/OH^- ratio is considered to affect the progress of corrosion, but no generally valid statements can be made. It has also been found that, for given amount of chloride ions in the mix, there are significantly

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more free chloride ions with NaCl than with $CaCl_2$.[11.67]

Because of these various factors, the proportion of bound chloride ions varies from 80 per cent to well below 50 per cent of the total chloride ion content. Therefore, there may not exist a fixed and unique value of the total amount of chloride ions below which corrosion will not occur. Tests[11.66,11.68] have shown that, in consequence of the various equilibrium requirements of the pore water, the mass of bound chlorides in relation to the mass of cement is independent of the water/cement ratio.

Influence of blended cements on corrosion

While the preceding discussion was concerned with the influence of the type of Portland cement on the chemical aspects of chloride ions, it is also important, indeed more so, to consider the influence of the type of blended cement on the pore structure of the hardened cement paste and

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on its penetrability, as well as on resistivity. This was largely done in Chapter 10, but those aspects of various cementitious materials which are particularly relevant to the movement of chloride ions will be considered here. It should be added that the same properties of hardened cement paste which influence the transport of chlorides also influence the supply of oxygen and the availability of moisture, both of which are necessary for corrosion to occur. However, the locations on steel where chlorides are present and where oxygen is needed are different: the former is at the anode, and the latter at the cathode.

The cementitious materials of interest are fly ash, ground granulated blast-furnace slag, and silica fume. All three, when properly proportioned in the mix, significantly reduce the penetrability of concrete and increase its resistivity, thereby reducing the rate of corrosion.[11.70,11.87,11.90] As far as silica fume is concerned, its positive effect is through improvement of the pore structure of hardened cement paste,

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which increases resistivity, even though silica fume reduces somewhat the pH value of the pore water in consequence of reaction with $Ca(OH)_2$.[11.98] Gjørv *et al.*[11.97] showed that 9 per cent of silica fume by mass in the cement reduced the chloride diffusivity by a factor of about 5.

It should be remembered that, because of its effect on workability, with the use of silica fume there is usually associated the inclusion of a superplasticizer. Superplasticizers *per se* do not affect the pore structure and, therefore, do not alter the process of corrosion.

The beneficial effects of the various cementitious materials are so significant that their use in reinforced concrete liable to corrosion in hot climates is virtually necessary: Portland cement alone should not be used.[11.89]

Tests on chloride ion diffusion through mortar indicate that fillers do not affect the movement of chlorides.[11.77]

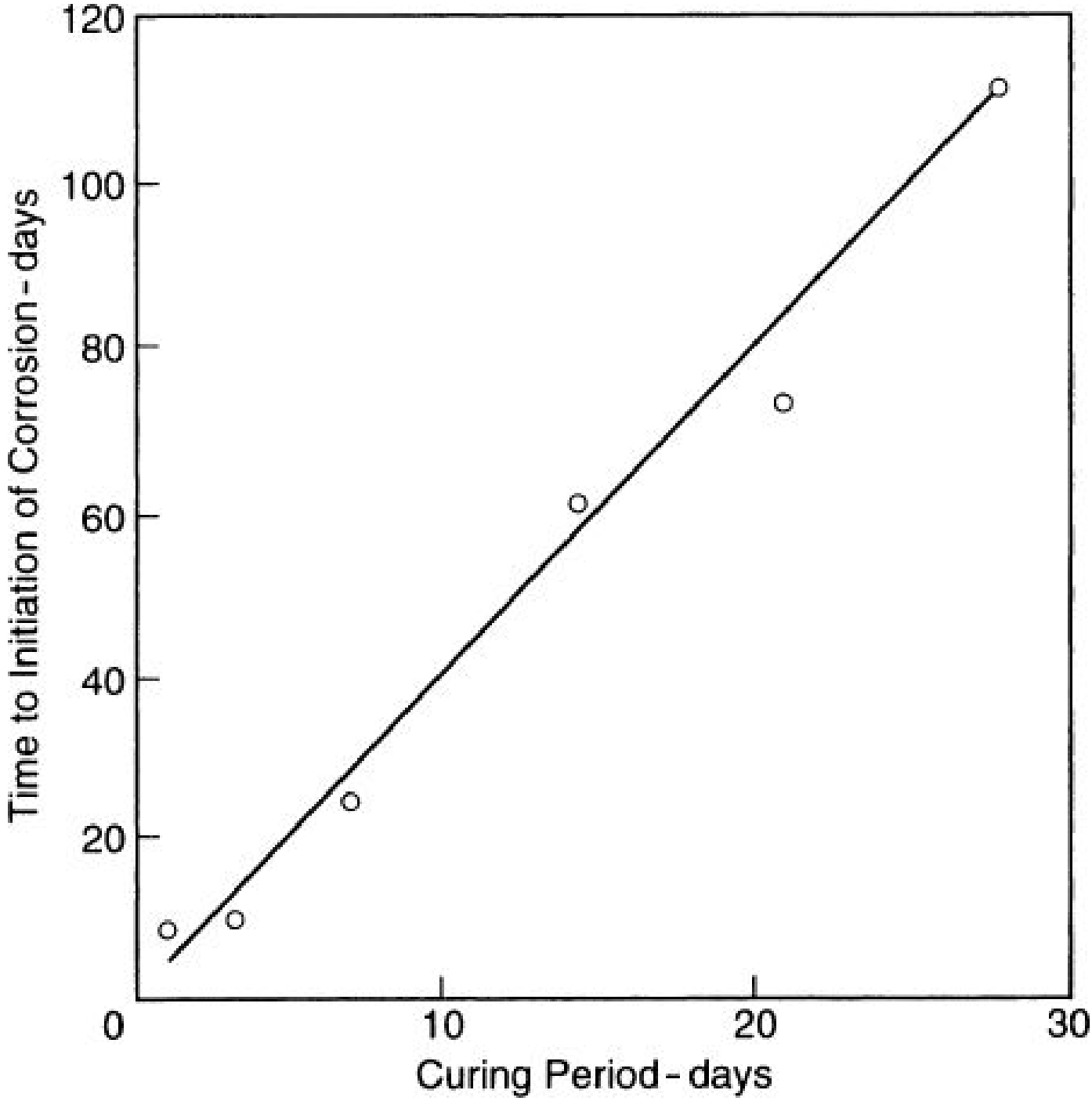
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Chloride ions in concrete made with high-alumina cement lead to a more aggressive situation than with Portland cement,[11.81] the comparison being made at the same chloride ion content. It can be recalled that the pH value in high-alumina cement concrete is lower than with Portland cement so that the passive state of the steel may be less stable.[11.81]

Further factors influencing corrosion

The preceding discussion of the influence of the composition of concrete upon its resistance to corrosion should be complemented by re-emphasizing the importance of good curing, whose effect is primarily upon the concrete in the cover zone. The time to *initiation* of corrosion is substantially increased by prolonged curing[11.69] (see Fig. 11.19). However, only fresh water must be used for curing because brackish water greatly increases the ingress of chlorides.[11.69]

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Fig. 11.19. Influence of the length of moist curing on time to initiation of corrosion of reinforcement; water/cement ratio of 0.5, cement content 330 kg/m^3 (550 lb/yd^2), Type V cement; specimens partially immersed in a 5 per cent solution of sodium chloride (based on ref. 11.69)

Once the corrosion has been initiated, its continuation is not inevitable: the progress of corrosion is influenced by the resistivity of the concrete between the anode and the cathode and by the continuing supply of the oxygen at the cathode. On the one hand, it is very doubtful that the supply of oxygen can be completely and reliably stopped by the application of a membrane, although developments in this field continue. On the other hand, the resistivity of concrete is a function of its moisture condition so that drying out would halt the corrosion, which, however, can re-start upon subsequent wetting.

Cracking of concrete in the cover facilitates the ingress of chlorides and, therefore, enhances

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corrosion. Although virtually all reinforced concrete in service exhibits some cracks, cracking can be controlled by appropriate structural design, detailing, and construction procedures. Cracks wider than about 0.2 to 0.4 mm (0.008 to 0.016 in.) are harmful. It may be worth mentioning that, although prestressed concrete is crack-free, the prestressing steel is more vulnerable to corrosion because of its nature; also, the small cross-sectional area of prestressing wires means that pitting corrosion greatly reduced their load-carrying capacity.

Higher temperature has several effects on corrosion. First, the content of free chlorides in the pore water increases; the effect is more pronounced with cements having a high C_3A content and with lower chloride concentrations in the original mix[11.62] (Fig. 11.20).

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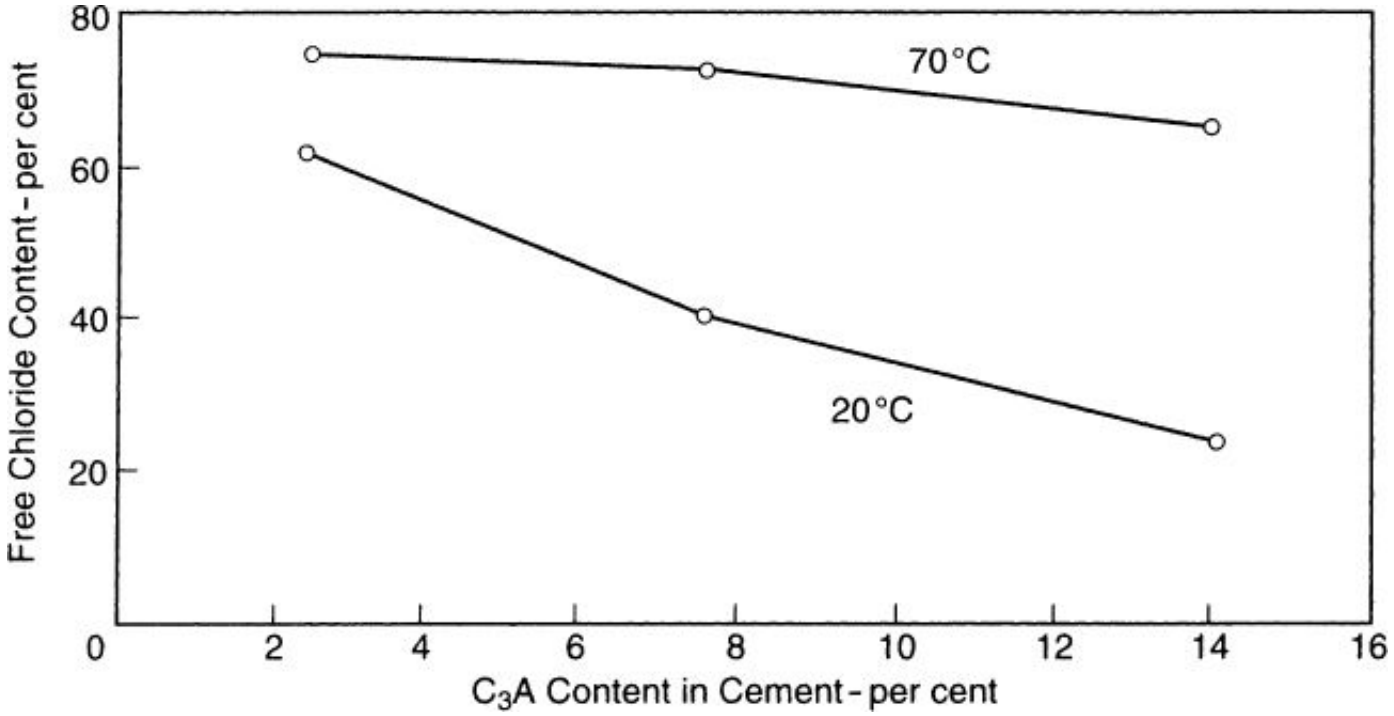


Fig. 11.20. Influence of C_3A content in cement on the amount of free chloride ions (expressed as a percentage of total chloride ions of 1.2 per cent of the mass of cement) at 20 and 70 °C (68 and 158 °F) (based on ref. 11.62 with kind permission of Elsevier Science Ltd, Kidlington, U.K.)

More importantly, the reactions of corrosion, like many chemical reactions, occur faster at higher temperatures. It is usually assumed that a rise of temperature of 10 °C (18 °F) doubles the rate of reaction, but there is some evidence that the increase is only 1.6-fold.[11.93] Whatever

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the exact factor, the accelerating effect of temperature explains why there is so much more corrosion-damaged concrete in hot coastal areas than in temperate parts of the world.

It may also be recalled that initial hardening of concrete at high temperatures results in a coarser pore structure (see p. 361), a consequence of which is a lower resistance to the diffusion of chloride ions.[11.39] The temperature differential between the surface of the concrete and its interior affects the diffusion; direct exposure to the sun can result in a significant rise in the temperature of the surface concrete above the ambient value.

Thickness of cover to reinforcement

The thickness of cover to reinforcement is an important factor controlling the transport of chloride ions: the greater the cover the longer the time interval before the chloride ion concentration at the surface of the steel reaches the threshold value. Thus, the quality of the concrete (in terms of its low penetrability) and the thickness of cover

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work together and can, therefore, to some extent, be traded off one against the other. For this reason, standards often specify combinations of cover and strength of concrete such that a lower thickness of cover requires a higher strength, and vice versa.

However, there are limitations to this approach. First of all, thick cover is of no avail if the concrete is highly penetrable. Moreover, the purpose of cover is not only to provide protection of reinforcement, but also to ensure composite structural action of steel and concrete, as well as, in some cases, to provide fire protection or resistance to abrasion. Unduly large thickness of cover would result in the presence of a considerable volume of concrete devoid of reinforcement. And yet, the presence of steel is required to control shrinkage and thermal stresses, and to prevent cracking due to those stresses. Were cracking to occur, the large thickness of cover would be proved to be detrimental. In practical terms, the cover thickness should not exceed 80 to 100

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mm (3 to 4 in.) but the decision on cover forms part of structural design.

Too small a thickness of cover should not be used either, because, however low the penetrability of the concrete, cracking, for whatever reason, or local damage or misplaced reinforcement can result in a situation where chloride ions can rapidly be transported to the surface of the steel.

Tests for penetrability of concrete to chlorides

A rapid test for the penetrability of concrete to chloride ions is prescribed by ASTM C 1202-10, which determines the electrical conductance, expressed as the total electrical charge in coulombs (ampere-seconds) passed during a certain time interval through a concrete disc between solutions of sodium chloride and sodium hydroxide when a potential difference of 60 V d.c. is maintained. The charge is related to the penetrability of the concrete to chloride ions, so that the test can be of help, in a comparative manner, in selecting a suit-

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able concrete mix. A somewhat similar test determines the a.c. impedance of specimens of various shapes.[11.86]

Tests of the kind just described do not necessarily replicate the transport of chloride ions in a real-life situation, nor do they have a sound scientific basis. Nevertheless, they are useful and certainly preferable to the assumption that resistance to chloride ion ingress is simply related to the strength of concrete; this assumption has been shown not to be valid[11.41] except in the most general manner.

Stopping corrosion

Simplified statements about methods of controlling or remedying corrosion which has been initiated may be unhelpful. All that should be stated here is that the progress of corrosion would be reduced by drying the concrete or by the prevention of oxygen supply through the application of surface barriers. This is a specialized field, and *ad hoc* solutions may, in fact, prove harmful; for

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instance, applying a barrier at the anode (rather than the cathode) would increase the ratio of the size of the cathode to the anode, which would increase the rate of corrosion.

It is reasonable to raise the question of whether there exist integral corrosion inhibitors, that is, substances which, while not preventing ingress of chlorides into concrete, inhibit the corrosion of steel. Nitrites of sodium[11.74] and calcium[11.72] have been found to be effective in laboratory tests. The action of the nitrite is to convert ferrous ions at the anode into a stable passive layer of Fe_2O_3, the nitrite ion reacting preferentially to the chloride ion. The concentration of nitrites must be sufficient to cope with a continuing ingress of chloride ions. Indeed, it is not certain that corrosion inhibitors are effective indefinitely, and do not simply delay corrosion.

The accelerating effect of the nitrites can be offset by the use of a retarding admixture, if need be. The search for other corrosion inhibitors continues.[11.73]

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Being incorporated in the mix, the inhibitors protect all the embedded steel. Nevertheless, inhibitors are no substitute for concrete of low penetrability: they are merely an additional safeguard. Moreover, sodium nitrite increases the hydroxyl ion concentration in the pore water, and this may increase the risk of alkali–aggregate reaction. Thus, the beneficial effect of an increased hydroxyl ion concentration upon the risk of corrosion of steel is accompanied by a negative effect on the risk of alkali–aggregate reaction. Of course, this is relevant only if the aggregate is susceptible to such a reaction in the first place.

A discussion of prevention of the corrosion of steel in concrete would be incomplete without a mention of the protection of steel by epoxy coating and by cathodic protection which makes the entire steel surface cathodic. Epoxy coating of steel is a specialized technique which can be helpful *in addition* to an adequate thickness of cover concrete of low permeability. In special cases, reinforcement made of stainless steel, or

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coated with stainless steel, can be used, but this is very expensive. Cathodic protection has been shown to be effective in some applications, but its use in a new structure is an admission of defeat in that the particular reinforced concrete structure is manifestly not durable.

A question which has to be faced occasionally is: can chloride ions be removed from the surface of the steel. Within the confines of this book, only a very brief answer can be given.

There has been developed a technique for desalinating concrete, in which chloride is removed by passing a heavy direct current between the corroding reinforcing steel (now acting as a cathode) and an external anode in electrolytic contact with the concrete; chloride ions migrate towards the external anode, thus moving away from the surface of the reinforcement.[11.84] It seems that only about one-half of the chloride in the concrete can be removed and, with time, corrosion is likely to re-start. Some negative consequences of the process may follow;[11.65] for example, the concen-

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tration of sodium ions which enter the pore water can become so high that aggregate which, under normal circumstances, is non-reactive with the alkalis may become reactive.

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Chapter 12. Testing of hardened concrete

We have seen that the properties of concrete are a function of time and ambient humidity, and this is why, in order to be of value, tests on concrete have to be performed under specified or known conditions. Different test methods and techniques are used in different countries and sometimes even in the same country. Because many of these tests are used in laboratory work, and especially in research, a knowledge of the influence of the test methods on the *measured* property is of importance. It is, of course, essential to distinguish between the effects of test conditions and the intrinsic differences in the concretes being tested.

Tests can be made for different purposes but the main two objectives of testing are quality control and compliance, now called conformity, with specifications. Additional tests can be made for specific purposes, e.g. compressive strength tests

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to determine the strength of concrete at transfer of prestress or at the time of the removal of formwork. It should be remembered that tests are not an end in themselves: in many practical cases, they do not lend themselves to a neat, concise interpretation, so that, in order to be of real value, tests should always be used against the background of experience. Nevertheless, because tests are generally performed for the purpose of comparison with a specified, or some other, value, any departure from the standard procedure is undesirable as it may lead to a dispute or to confusion.

Tests can be broadly classified into mechanical tests to destruction and non-destructive tests which allow repeated testing of the same specimen and thus make possible a study of the change in properties with time. Non-destructive tests also permit testing concrete in an actual structure.

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Tests for strength in compression

The most common of all tests on hardened concrete is the compressive strength test, partly because it is an easy test to perform, and partly because many, though not all, of the desirable characteristics of concrete are qualitatively related to its strength; but mainly because of the intrinsic importance of the compressive strength of concrete in structural design. Although invariably used in construction, the compressive strength test has some disadvantages, but it has become, in French parlance,[12.80] a part of the engineer's *bagage culturel*.

The strength test results may be affected by variation in: type of test specimen; specimen size; type of mould; curing; preparation of the end surface; rigidity of the testing machine; and rate of application of stress. For this reason, testing should follow a single standard, with no departure from prescribed procedures.

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Compressive strength tests on specimens treated in a standard manner which includes full compaction and wet curing for a specified period give results representing the *potential* quality of the concrete. Of course, the concrete in the structure may actually be inferior, for example, due to inadequate compaction, segregation, or poor curing. These effects are of importance if we want to know when the formwork may be removed, or when further construction may continue, or the structure be put into service. For this purpose, the test specimens are cured under conditions as nearly similar as possible to those existing in the actual structure. Even then, the effects of temperature and moisture would not be the same in a test specimen as in a relatively large mass of concrete. The age at which *service* specimens are tested is governed by the information required. On the other hand, *standard* specimens are tested at prescribed ages, generally 28 days, with additional tests often made at 3 and 7 days. Two types of compression test specimens are used: cubes and cylinders. Cubes are used in Great Britain,

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Germany, and many other countries in Europe. Cylinders are the standard specimens in the United States, France, Canada, Australia, and New Zealand. In Scandinavia, tests are made on both cubes and cylinders. The use of one or other type of specimen in a given country is so ingrained that the European Standard BS EN 206:1996 allows the use of both cylinders and cubes. Indeed, European standards use both types of strength.

Cube test

The specimens are cast in steel or cast-iron moulds of robust construction, generally 150 mm (or 6 in.) cubes, which should conform within narrow tolerances to the cubical shape, prescribed dimensions and planeness. The mould and its base must be clamped together during casting in order to prevent leakage of mortar. Before assembling the mould, its mating surfaces should be covered with mineral oil, and a thin layer of similar oil must be applied to the inside surfaces of the mould in order to prevent the development of bond between the mould and the concrete.

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The standard practice prescribed by BS EN 12390-1 : 2000 is to fill the mould in one or more layers. Each layer of concrete is compacted by a vibrating hammer, or using a vibrating table, or by a square steel punner. Ramming should continue until full compaction without segregation or laitance has been achieved because it is essential that the concrete in the cube be fully compacted if the test result is to be representative of the properties of fully-compacted concrete. If, on the other hand, a check on the properties of the concrete *as placed* is required, then the degree of compaction of the concrete in the cube should simulate that of the concrete in the structure. Thus, in the case of precast members compacted on a vibrating table, the test cube and the member may be vibrated simultaneously, but the disparity of the two masses makes the achievement of the same degree of compaction extremely difficult, and this method is not recommended.

According to BS EN 12390-2 : 2009, after the top surface of the cube has been finished by

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means of a float, the cube is stored undisturbed for 16 to 72 hours at a temperature of 20 ± 5 °C (68 ± 9 °F) and a relative humidity preventing dehydration. At the end of this period, the mould is stripped and the cube is further cured in water or in a chamber with a relative humidity of not less than 95 per cent and at 20 ± 2 °C (68 ± 4 °F).

In the compression test, the cube, while still wet, is placed with the cast faces in contact with the platens of the testing machine, i.e. the position of the cube when tested is at right angles to that as-cast. According to BS 1881-116 : 1983 (withdrawn), the load on the cube should be applied at a constant rate of stress equal to 0.2 to 0.4 MPa/second (30 to 60 psi/second). ASTM C 39-09a prescribes a rate of 0.25 ± 0.05 MPa/sec (35 ± 7 psi/sec). Because of the non-linearity of the stress–strain relation of concrete at high stresses, the rate of increase in strain must be increased progressively as failure is approached, i.e. the speed of the movement of the head of the testing machine has to be increased. The require-

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ments for testing machines are discussed on p. 590.

From the preceding discussion it can be seen that there is no unique simple relation between the strengths of cubes and cylinders made from the same concrete. And yet, European legislation, which made it possible for a contractor in any European Union country to bid for, and build, concrete structures in any country, made it highly desirable to describe the strength of concrete in a manner that allowed such construction in an unequivocal manner. Consequently, European Standards chose to assume that the strength of a cube is 5/4 of the strength of the cylinder and all mixes therefore describe strength by a double number, e.g. 40/32, meaning that a cube strength of 40 MPa is equivalent to a cylinder strength of 32 MPa. However, in concrete made with light-weight aggregate, the ratio is much higher than 5/4.[12.150]

The compressive strength, known also as the crushing strength, is reported to the nearest 0.5

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MPa or 50 psi; a greater precision is usually only apparent.

Cylinder test

The standard cylinder is 6 in. in diameter, 12 in. long, or 150 by 300 mm, but in France the size is 159.6 by 320 mm; the diameter of 159.6 mm gives a cross-sectional area of 20 000 mm^2. Cylinders are cast in a mould generally made of steel or cast iron, with a clamped base; cylinder moulds are specified by ASTM C 470-09, which allows also the use of single-use moulds, made of plastic, sheet metal and treated cardboard.

Details of moulds may seem to be trivial but non-standard moulds can result in a misleading test result. For example, if the mould has a low rigidity, some of the compaction effort is dissipated so that the compaction of the concrete in the mould may be inadequate; a lower strength would be recorded. Conversely, if the mould allows leakage of mix water, the strength of concrete would increase. Excessive re-use of moulds

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intended for single use or for limited re-use leads to their distortion and to an apparent loss of strength.[12.55]

The method of making test cylinders is prescribed by BS EN 12390-2 : 2009 and by ASTM C 192-07. The procedure is similar to that used with cubes, but there are differences in detail between the British and American standards.

The testing of a cylinder in compression requires that the top surface of the cylinder be in contact with the platen of the testing machine. This surface, when finished with a float, is not smooth enough for testing and requires further preparation; this is a disadvantage of cylinders tested in compression. Treatment of the top end of cylinders by capping is considered in a later section, but even though the cylinders will be capped, ASTM C 192-07 and C 31-09 do not allow depressions or excrescences greater than 3 mm ($\frac{1}{8}$ in.); these could result in air pockets.[12.55]

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Equivalent cube test

Sometimes, the compressive strength of concrete is determined using parts of a beam tested in flexure. The end parts of such a beam are left intact after failure in flexure and, because the beam is usually of square cross-section, an ‘equivalent’ or ‘modified’ cube can be obtained by applying the load through square steel plates of the same size as the cross-section of the beam. It is important that the two plates be accurately placed vertically above one another; a suitable jig is shown in Fig. 12.1. The specimen should be placed so that the as-cast top surface of the beam is not in contact with either plate.

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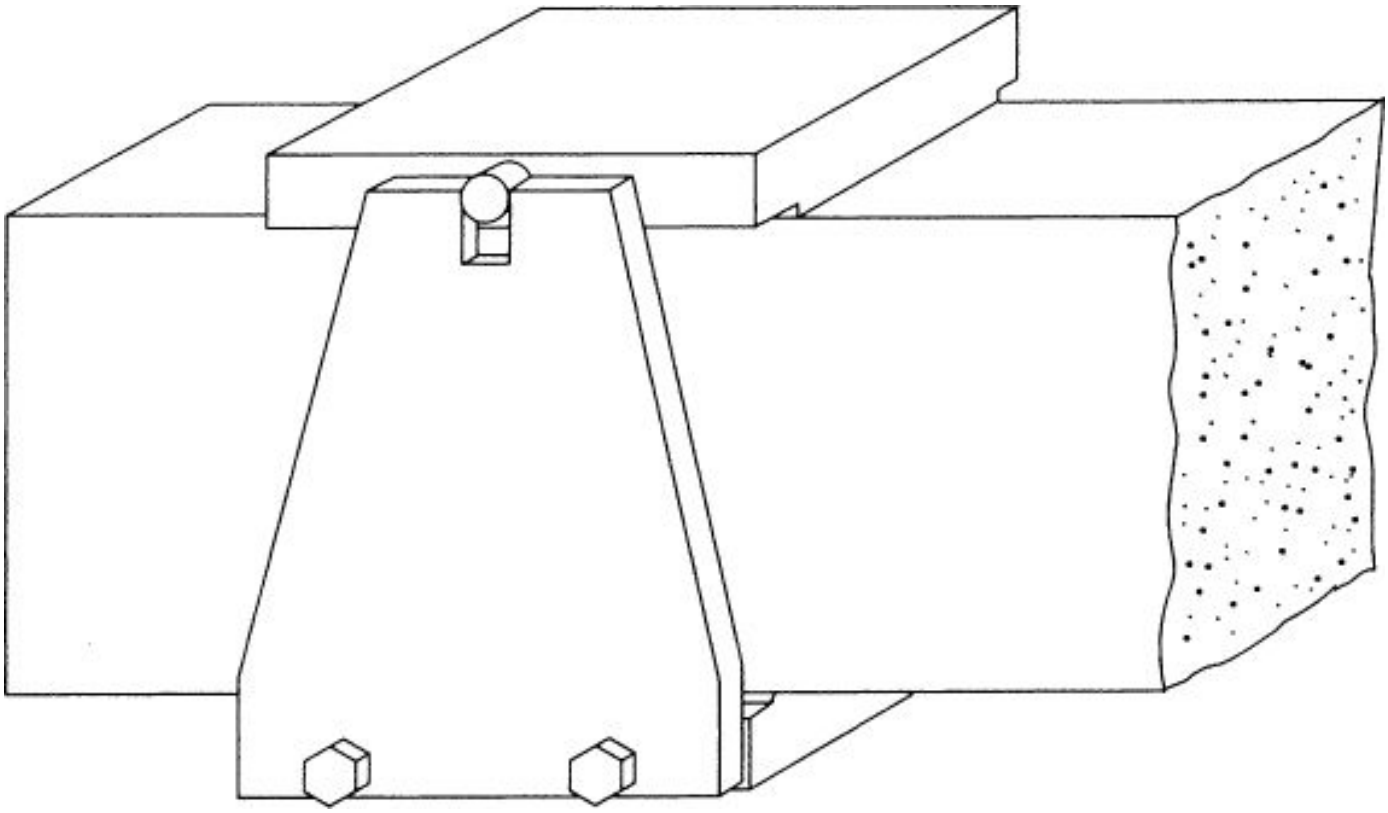


Fig. 12.1. Jig for testing equivalent cubes

The test is prescribed by BS 1881-119 : 1983 and ASTM C 116-90. The latter, now withdrawn, allowed the use of beams whose cross-section is rectangular.

The strength of a modified cube is approximately the same as the strength of a standard cube of the same size: actually, the restraint of the overhanging parts of the 'cube' may result in a slight increase in ultimate strength[12.4] so that it is reasonable to assume the strength of a modified

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cube to be, on average, 5 per cent higher than that of a cast cube of the same size.

Effect of end condition of specimen and capping

When tested in compression, the top surface of the test cylinder is brought into contact with the platen of the testing machine and, because this surface is not obtained by casting against a machined plate but finished by means of a float, the top surface is somewhat rough and not truly plane. Under such circumstances, stress concentrations are introduced and the apparent strength of the concrete is reduced. Convex end surfaces cause a greater reduction than concave ones as they generally lead to higher stress concentrations. The reduction in the measured loss in strength is particularly high in high strength concrete.[12.5]

To avoid this reduction in strength, plane end surfaces are essential: ASTM C 617-09a requires the end surfaces of a cylinder to be plane within

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0.05 mm (0.002 in.), as determined by a straight edge and a feeler gauge, and to be perpendicular to the axis of the cylinder within 0.5°. A method of testing concrete cylinders for planeness and parallelism of end and perpendicularity of sides is prescribed in the U.S. Army Corps Engineers Handbook for Concrete and Cement.[12.81] While the procedures are not unduly complex, such a test is most likely to be of interest in research work. A limitation on planeness of the platens of the testing machine is prescribed by ASTM C 39-09a.

In addition to the absence of 'high spots', the contact surfaces should be free from grains of sand or other debris (from a previous test), which would lead to premature failure and, in extreme cases, to sudden splitting.

There are three possible means of overcoming the ill-effects of an uneven end surface of the specimen: capping, grinding, and packing with a bedding material.

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Packing is not recommended because it results in an appreciable lowering of the apparent mean strength of concrete, compared with capped, and often even with smooth-trowelled, specimens (see Fig. 12.6). At the same time, the scatter of strength results is appreciably reduced because the influence of the defects in planeness (responsible for the large variation in strength) is eliminated.

The reduction in strength introduced by packing, usually of softboard, cardboard, or lead, arises from lateral strains induced in the cylinder by the Poisson's ratio effect in the packing material. Poisson's ratio of this material is generally higher than that of concrete so that splitting is induced. This effect is similar to, although usually greater than, that of lubricating the ends of the cylinder in order to eliminate the restraining influence of the friction between the specimen and the platen on lateral spread of the concrete. Such lubrication has been found to reduce the strength of the specimen.

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Capping with a suitable material does not adversely affect the measured strength and reduces its scatter compared with uncapped specimens. An ideal capping material should have strength and elastic properties similar to those of the concrete in the specimen; there is then no enhanced tendency to splitting, and a reasonably uniform distribution of stress over the cross-section of the specimen is achieved.

The capping operation may be performed either just before testing or alternatively soon after the specimen has been cast. Different materials are used in the two cases but, whatever the capping material, it is essential that the cap be thin, preferably 1.5 to 3 mm ($\frac{1}{16}$ to $\frac{1}{8}$ in.) thick. The capping material must be no weaker than the concrete in the specimen; however, the strength of the cap is affected by its thickness. Too great a difference in strength is thought to be undesirable because a very strong cap may produce a large lateral restraint and thus lead to an apparent increase in strength. The influence of the capping

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material on strength is much greater in the case of high- or medium-strength concrete than in low-strength concrete;[12.6,12.82] in the latter case, Poisson's ratio of the capping material is also of no influence. With 48 MPa (7000 psi) concrete, high-strength capping leads to strengths 7 to 11 per cent higher than low-strength capping. For 69 MPa (10 000 psi) concrete, the difference can be as high as 17 per cent. These differences are smaller when the thickness of the cap is very small.[12.82]

Capping procedures are prescribed in ASTM C 617-09a. When the capping operation is to be performed soon after casting, Portland cement paste is used. Before its application, it is preferable to allow two to four hours' delay after casting so that the plastic shrinkage of the concrete and the resulting settlement of the top surface of the material in the mould can take place. It is convenient to finish the original concrete about 1.5 to 3 mm ($\frac{1}{16}$ to $\frac{1}{8}$ in.) short of the top of the mould. During capping, this space is filled with a stiff

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cement paste which has been allowed partially to shrink and, by working down a glass or machined steel plate, a plane surface is obtained. Experience is necessary to make this operation successful and particularly to obtain a clean break between the cement paste and the plate: greasing the plate with a mixture of lard oil and paraffin[12.7] or covering with a thin film of graphite grease[12.6] has been found helpful. Following capping, moist curing must be continued.

The alternative method is to cap the cylinder shortly before it is tested: the actual time depends on the hardening properties of the capping material. The cap should be 3 to 8 mm ($\frac{1}{16}$ to $\frac{5}{16}$ in.) thick and it must bond well to the underlying concrete. Suitable capping materials are high-strength gypsum plaster and molten sulfur mortar, but regulated-set cement[12.82] has also been used.

The sulfur mortar consists of sulfur and a granular material such as milled fire clay. The

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mixture is applied in a molten state and allowed to harden with the specimen in a jig which ensures a plane and square end surface. The use of a fume cupboard is necessary because toxic fumes are produced. The sulfur mixture from tested cylinders can be re-used up to five times, but care is required in selecting and using sulfur mortar as otherwise the strength of the test cylinders can be significantly affected.[12.53] Moist curing must be resumed after capping.

An alternative to capping is to grind (using silicon carbide abrasion) the bearing surface of the specimen until it is plane and square. This method produces very satisfactory results but is rather expensive. It has been suggested that grinding leads to a higher strength than capping in that any loss of strength associated with capping is absent.[12.84] Thus, ground specimens have the same strength as those with 'perfect' cast test-surfaces.

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Non-bonded caps

Although sulfur-mortar capping is satisfactory for concretes with strengths up to about 100 MPa (or 14 000 psi), the capping operation is tedious and potentially slightly dangerous. For this reason, a number of attempts have been made to develop non-bonded caps. These are in the form of an elastomeric pad inserted into a restraining rigid metal cap of the type shown in Fig. 12.2. Neoprene pads in steel caps have been found to be satisfactory.[12.74] The pad should fit snugly in the cap whose internal diameter should be about 6 mm ($\frac{1}{4}$ in.) larger than the diameter of the concrete cylinder. It is important that the cylinder be concentric with the cap.

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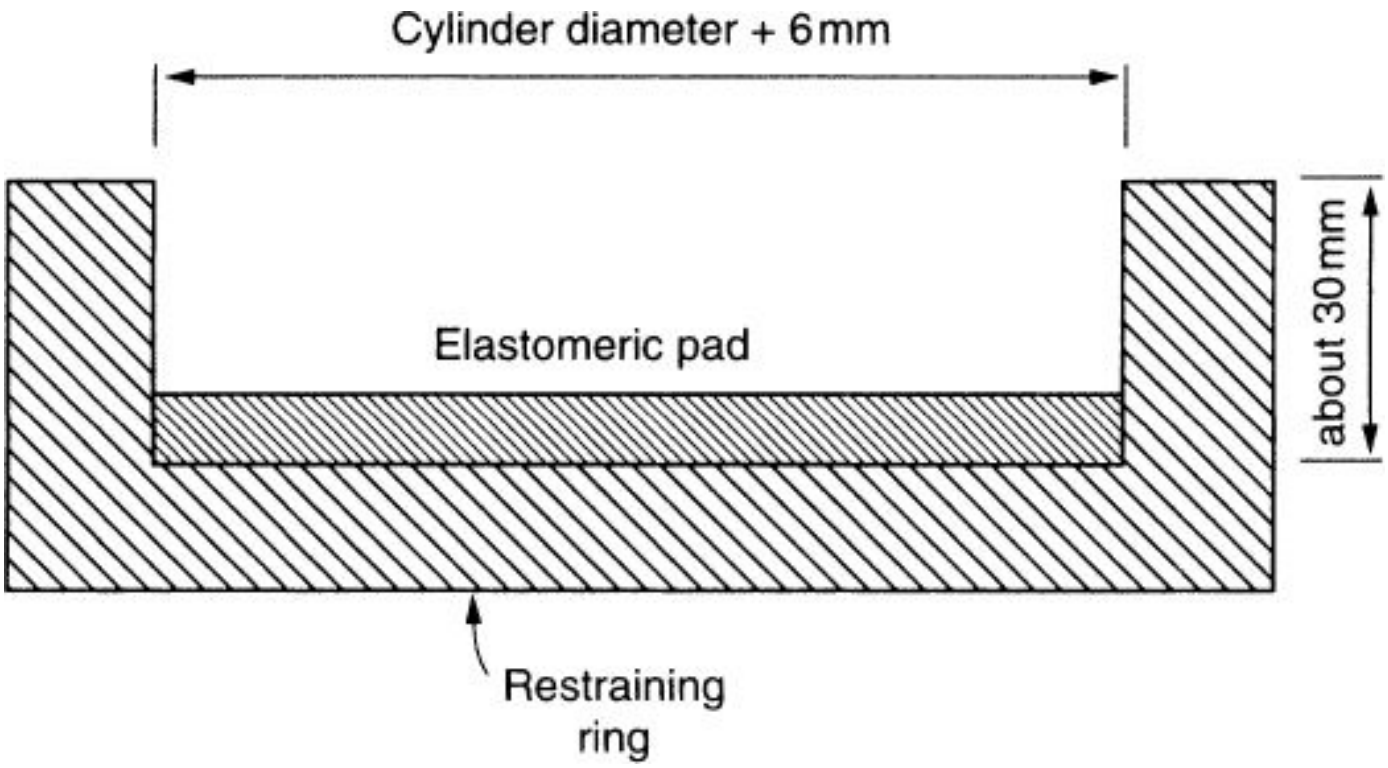


Fig. 12.2. Cross-section of a typical non-bonded capping system

The use of non-bonded rubber caps is permitted in Australia.[12.75] It has been found that the caps have to be fully moulded (and not punched) and that rubber of different hardness has to be used according to the strength of concrete.[12.75] This is a complicating factor if the approximate strength of the cylinder cannot be anticipated. Furthermore, rubber caps should not be used with low-strength concrete: limiting values are 20 MPa (or 3000 psi)[12.75] and 30 MPa (or 4500

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psi)[12.73] have been suggested because, at lower strengths, non-bonded caps lead to lower strength values than those obtained with conventional sulfur-mortar capping.

The use of non-bonded caps has been limited in other countries and, therefore, a reliable comparison of strengths obtained using these caps with values of strength of sulfur-mortar capped cylinders is not available. However, even if there is a small systematic difference in strength compared with the strength of sulfur-mortar capped cylinders, this is not important because every method of capping introduces a systematic influence on the observed strength so that there is no 'true' strength of concrete. What is important is that a single method is used on a given construction project.

The variability of test results on cylinders with non-bonded caps is smaller than with standard caps. This may be due to the beneficial effect of non-bonded caps in reducing the consequences of the roughness of cylinder ends.[12.72]

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Capping very high strength concrete presents a special problem in that such concrete has a higher strength than sulfur-mortar caps. Non-bonded caps are also unsatisfactory because the pads can become seriously damaged and even extruded from the cap.[12.71] Grinding the cylinder ends gives very good results but it is slow and expensive. Moreover, a high quality of grinding and lapping must be rigorously ensured.

To avoid grinding, the use of a sand-filled restraining steel cap has been developed: dry, fine siliceous sand is compacted in the cap; the cylinder is placed on top of the sand; and molten paraffin is poured in order to form a seal confining the sand and maintaining the centring of the cylinder.[12.71] The compressive strengths of concretes up to 120 MPa (or 17 000 psi) using sand capping agreed well with those of ground specimens.[12.71]

For research purposes, the application of a truly uniform compressive stress may be desirable. This has been achieved by loading through a mat of thin rubber strips with gaps in-

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between,[12.12] or through a stiff wire brush.[12.56] A brush 'platen' consists of filaments, about 5 by 3 mm (0.20 by 0.12 in.) in cross-section with gaps 0.2 mm (0.008 in.) wide. This combination allows the free lateral deformation of concrete to develop but the filaments do not buckle. The use of brush platens on 100 mm (4 in.) cubes has been found to yield a strength equal to about 80 per cent of the strength with rigid platens at a constant strain rate (for concrete strength of about 45 MPa (6500 psi)).[12.85]

Testing of compression specimens

In addition to being plane, the end surfaces of the cylinder should be normal to its axis, and this guarantees also that the end planes are parallel to one another. A small tolerance is permitted: an inclination of the axis of the specimen to the axis of the testing machine of 6 mm in 300 mm ($\frac{1}{4}$ in. in 12 in.) has been found to cause no loss of strength.[12.5] The axis of the specimen when placed in the testing machine should be as near

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to the axis of the platen as possible, but errors up to 6 mm ($\frac{1}{4}$ in.) do not affect the strength of cylinders made with low strength concrete.[12.5] However, BS 1881-115 : 1986 (withdrawn) required a provision for positive and accurate location of the test specimens. Likewise, a small lack of parallelism between the end surfaces of the specimen does not adversely affect its strength, provided the testing machine is equipped with a seating which can align freely with the end plane of the specimens. The current British Standard for testing machines is BS EN 12390-4 : 2000.

Free alignment is achieved by a spherical seat. This can act not only when the platens are brought into contact with the specimen but also when the load is being applied. At this stage, some parts of the specimen may deform more than others. This is the case in a cube in which, due to bleeding, the properties of different layers (as-cast) may not be the same. In the testing position, the cube is at right angles to the as-cast position so that the weaker and the stronger parts

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(parallel to one another) extend from platen to platen. Under load, the weaker concrete, having a lower modulus of elasticity, deforms more. With an effective spherical seat, the platen will follow the deformation so that the stress on all parts of the cube is the same and failure occurs when this stress reaches the strength of the weaker part of the cube. On the other hand, if the platen does not change its inclination under load (i.e. moves parallel to itself) a greater load is carried by the stronger part of the cube. The weaker part still fails first, but the maximum load on the cube is reached only when the stronger part of the cube carries its maximum load, too: thus the total load on the cube is greater than when the platen is free to rotate. This behaviour was confirmed experimentally.[12.9]

To make the spherical seat of a testing machine effective under load, a highly polar lubricant has to be used to reduce the coefficient of friction to a value as low as 0.04 (compared with 0.15 when a graphite lubricant is used).[12.10]

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ASTM C 39-09a specifies the use of petroleum-type oil such as conventional motor oil. It is not clear, however, whether making this movement of the platen possible results in the observed strength being more representative of the concrete under test. There are indications that a machine with a platen that does not change inclination under load gives more reproducible results when nominally similar cubes are tested.[12.11] For this reason the seat must not move under load. In any case, the observed strength is seriously affected by the friction at the surface of the ball seat, so that, for tests to be comparable, it is essential to maintain this surface in a standardized condition.

The loading of a platen through a spherical seat induces bending and distortion of the platen, which depend on the thickness of the platen. ASTM C 39-09a prescribes the thickness of the platen in relation to the size of the spherical seat, the latter being governed by the size of the specimen.

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Figure 12.3(a) indicates schematically the normal stress distribution at the platen-concrete interface when a ‘hard’ platen is used: the compressive stress is then higher near the perimeter than at the centre of the specimen. The same distribution exists when the specimen or the platen are slightly concave. Conversely, when a ‘soft’ platen is used (Fig. 12.3(b)), the compressive stress is higher near the centre of the specimen than around the perimeter. This condition is also produced by a slightly convex specimen face or platen. In addition to the stress distributions of Fig. 12.3, some local variations in stress exist due to the heterogeneity of concrete, and specifically due to the presence of coarse aggregate particles near the end faces.

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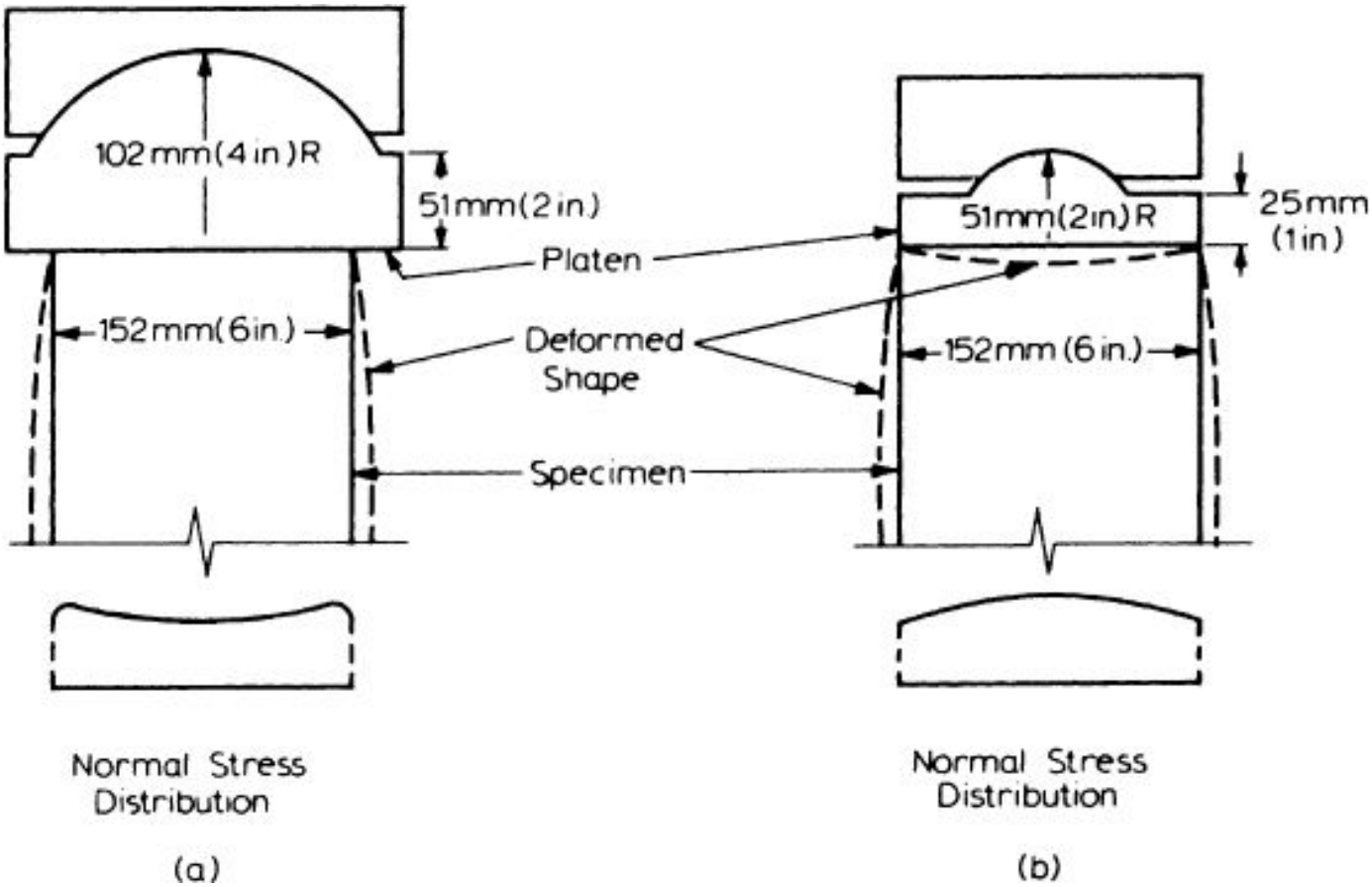


Fig. 12.3. Normal stress distribution near ends of specimens when tested in a machine with: (*a*) hard platens; (*b*) soft platens

A description of the different types of testing machines is outside the scope of this book, but it ought to be mentioned that the failure of the specimen is affected by the design of the machine, especially by the energy stored in it. With a very rigid machine, the high deformation of the specimen under loads approaching the ultimate load is not followed by the movement of the ma-

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chine head, so that the rate at which the load is applied decreases and a higher strength is recorded. On the other hand, in a less rigid machine, the load follows more nearly the load-deformation curve for the specimen and, when cracking commences, the energy stored by the machine is released rapidly. This leads to failure under a lower load than would occur in a more rigid machine, often accompanied by a violent explosion.[12.8] The exact behaviour depends on the detailed characteristics of the machine, not only its longitudinal stiffness, but also its lateral stiffness being relevant.[12.53] Proper and regular calibration of testing machines is essential; this is prescribed in BS EN 12390-4 : 2000. The same standard gives a method of verifying the performance of a testing machine.

Failure of compression specimens

On p. 293, we considered the failure of concrete subjected to uniaxial compression. The compression test imposes, however, a rather more com-

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plex system of stress, tangential forces being developed between the end surfaces of the concrete specimen and the adjacent steel platens of the testing machine. In each material, the vertical compression acting (the nominal stress on the specimen) results in a lateral expansion due to the Poisson's ratio effect. However, the modulus of elasticity of steel is 5 to 15 times greater, and Poisson's ratio no more than twice greater, than the corresponding values for concrete, so that the lateral strain in the platen is small compared with the transverse expansion of the concrete if it were *free* to move. For instance, Newman and Lachance[12.57] found the lateral strain in a steel platen to be 0.4 of the lateral strain in the concrete at a distance from the interface sufficient to remove the restraining effect.

It can be seen then that the platen restrains the lateral expansion of the concrete in the parts of the specimen near its ends: the degree of restraint exercised depends on the friction actually developed. When the friction is eliminated, e.g. by ap-

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plying a layer of graphite or paraffin wax to the bearing surfaces, the specimen exhibits a large lateral expansion and eventually splits along its full length.

With friction acting, i.e. under normal conditions of test, an element within the specimen is subjected to a shearing stress as well as to compression. The magnitude of the shearing stress decreases, and the lateral expansion increases, with an increase in distance from the platen. As a result of the restraint, in a specimen tested to failure there is a relatively undamaged cone or pyramid of height approximately equal to $\frac{1}{2}d\sqrt{3}$ (where d is the lateral dimension of the specimen).[12.4] If the specimen is longer than about $1.7d$, a part of it will be free from the restraining effect of the platens. We can note that specimens whose length is less than $1.5d$ show a considerably higher strength than those with a greater length (see Fig. 12.5).

It seems then, that, when a shearing stress acts in addition to the uniaxial compression, failure

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is delayed, and it can, therefore, be inferred that it is not the principal compressive stress that induces cracking and failure but probably the lateral tensile strain. The actual collapse may be due, at least in some cases, to the disintegration of the core of the specimen. The lateral strain is induced by the Poisson's ratio effect and, assuming this ratio to be approximately 0.2, the lateral strain is $\frac{1}{5}$ of the axial compressive strain. Now, we do not know the exact criteria of failure of concrete but there are strong indications that failure occurs at a limiting strain of 0.002 to 0.004 in compression or 0.0001 to 0.0002 in tension. Because the ratio of the latter of these strains to the former is less than Poisson's ratio of concrete, it follows that conditions of failure in circumferential tension are achieved before the limiting compressive strain has been reached.

Vertical splitting has been observed in numerous tests on cylinders, particularly in high-strength specimens made of mortar or neat cement paste, and also in sulfur-infiltrated concrete.

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The effect is less common in ordinary concrete when coarse aggregate is present because it provides lateral continuity.[12.4] The presence of vertical cracks has also been confirmed by measurements of ultrasonic pulse velocity along and across the specimen.[12.13]

The observations on the actual stress distribution in a nominally uniaxial situation do not necessarily detract from the value of the compression test as a comparative test, but we should be wary of interpreting it as a true measure of the compressive strength of concrete.

Effect of height/diameter ratio on strength of cylinders

Standard cylinders are of height h equal to twice the diameter d, but sometimes specimens of other proportions are encountered. This is particularly the case with cores cut from in situ concrete: the diameter depends on the size of the core-cutting tool whereas the height of the core varies with the thickness of the slab or member. If the core is too

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long, it can be trimmed to the *h/d* ratio of 2 before testing but, with too short a core, it is necessary to estimate the strength of the same concrete as if it had been determined on a specimen with *h/d* = 2.

ASTM C 42-04 and BS EN 12504-1 : 2009 give correction factors (Table 12.1) but Murdock and Kesler[12.14] found that the correction depends also on the level of strength of the concrete (Fig. 12.4). High strength concrete is less affected by the height/diameter ratio of the specimen, and such a concrete is also less influenced by the shape of the specimen; the two factors should be related as there is comparatively little difference between the strengths of a cube and of a cylinder with *h/d* = 1.

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Table 12.1. Standard Correction Factors for Strength of Cylinders with Different Ratios of Height to Diameter

Height to diameter ratio (h/d)	*Strength correction factor*	
	ASTM C 42-04	*BS EN 12504-1 : 2009*
2.00	1.00	1.00
1.75	0.98	0.97
1.50	0.96	0.92
1.25	0.93	0.87
1.00	0.87	0.80

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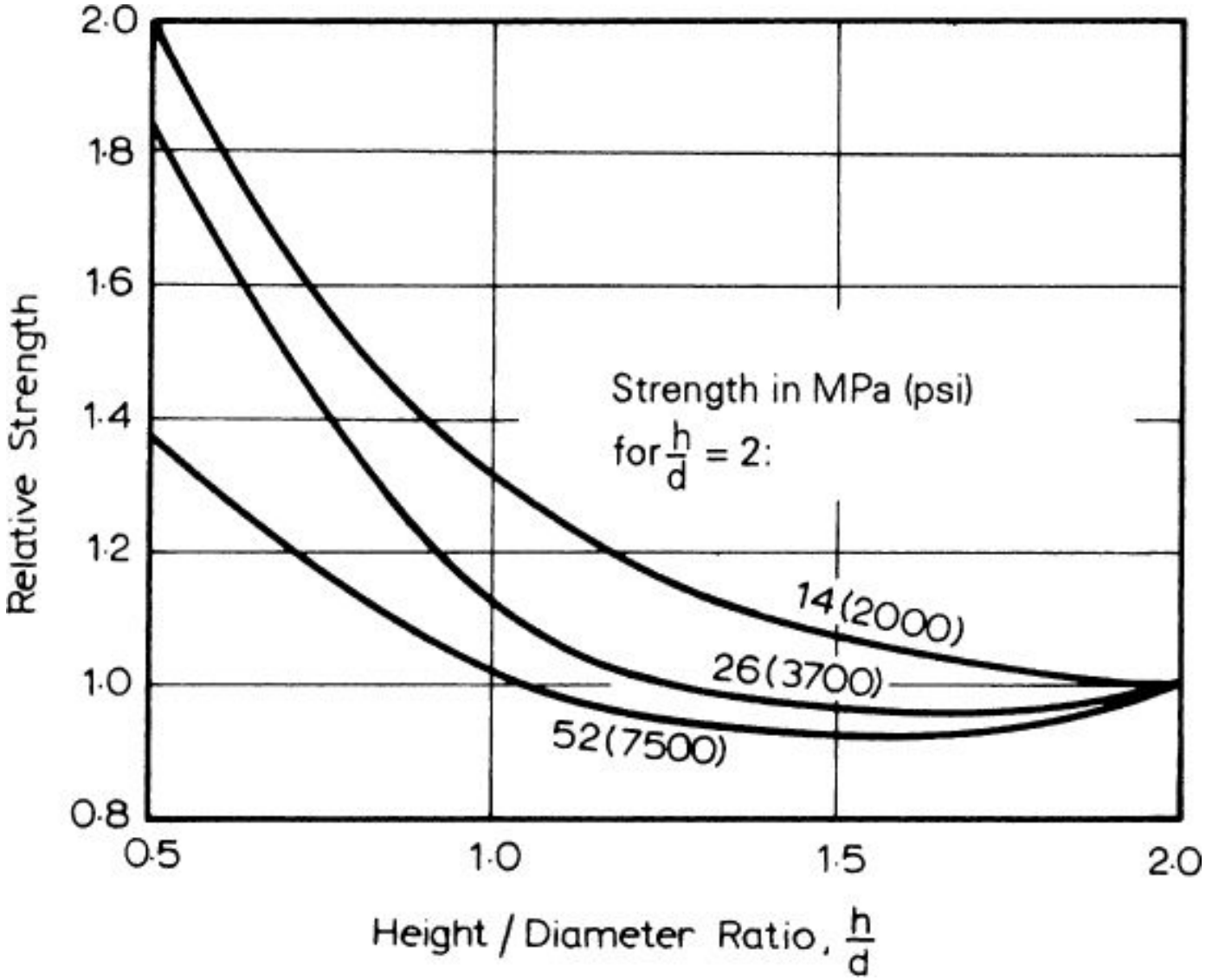


Fig. 12.4. Influence of the height/diameter ratio on the apparent strength of a cylinder for different strength levels[12.14]

The influence of strength on the conversion factor is of practical significance in the case of low strength concrete, if cores with *h/d* smaller than 2 are tested. Using ASTM C 42-04 and, even more so, BS EN 12504-1 : 2009, factors the

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strength that would be obtained with an *h/d* ratio of 2 would be overestimated; yet, it is in the case of concrete of low strength, or suspected of having too low a strength, that a correct estimate of strength is often particularly important.

The general pattern of influence of *h/d* on the strength of low- and medium-strength concrete is shown in Fig. 12.5. For values of *h/d* smaller than 1.5 the measured strength increases rapidly due to the restraining effect of the platens of the testing machine. When *h/d* varies between about 1.5 and 4, strength is affected only little and, for *h/d* values between 1.5 and 2.5, the strength is within 5 per cent of the strength of standard specimens (*h/d* = 2). For values of *h/d* above 5, strength falls off more rapidly, the effect of the slenderness ratio becoming apparent.

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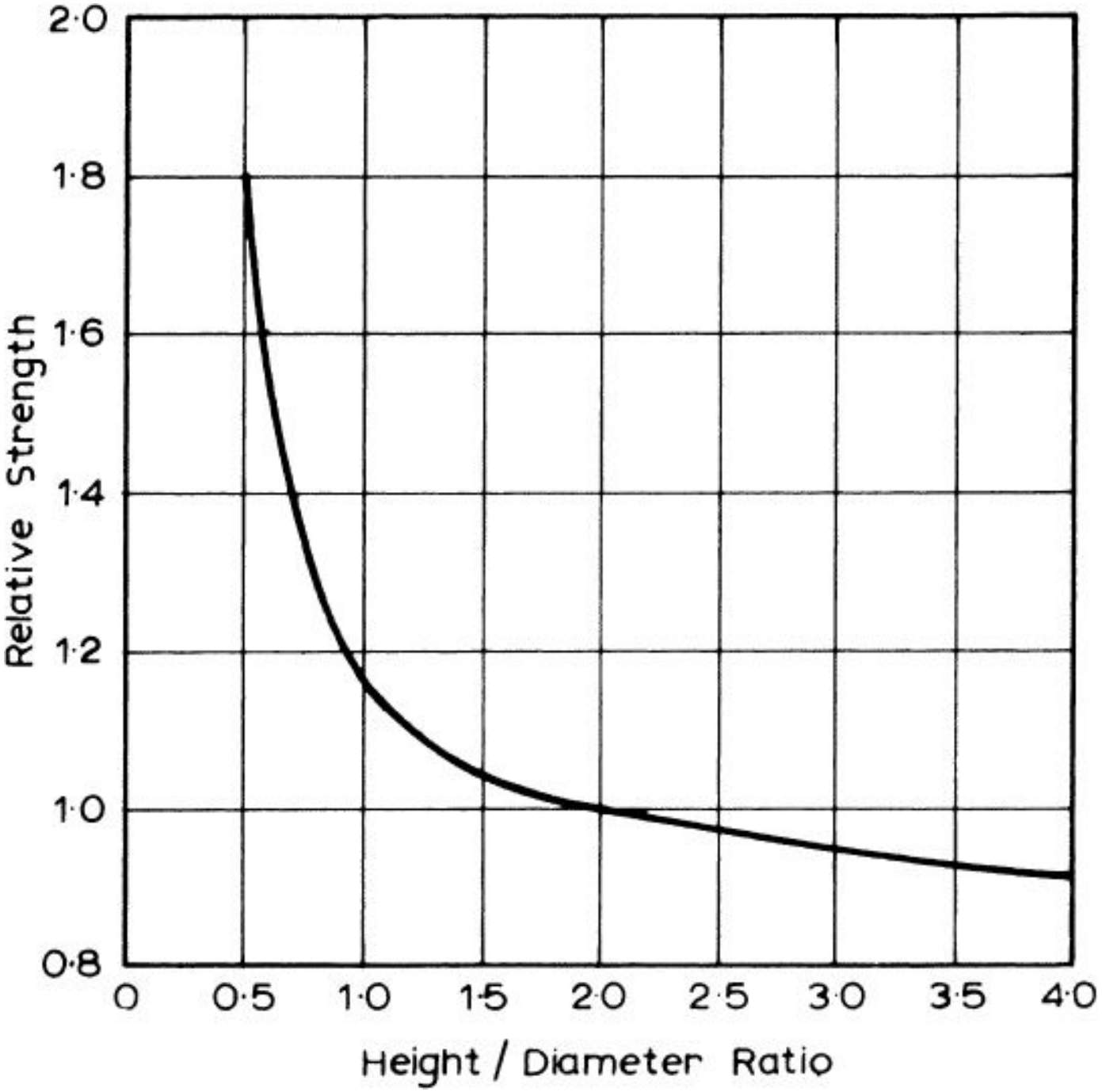


Fig. 12.5. General pattern of influence of the height/diameter ratio on the apparent strength of a cylinder[12.40]

It seems thus that the choice of the standard height/diameter ratio of 2 is suitable, not only be-

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cause the end effect is largely eliminated and a zone of uniaxial compression exists within the specimen, but also because a slight departure from this ratio does not seriously affect the measured value of strength. No correction is required for values of *h/d* between 1.94 and 2.10.

The influence on strength of the ratio of height to the least lateral dimension applies also in the case of prisms.

Of course, if the end friction is eliminated, the effect of *h/d* on strength disappears but this is very difficult to achieve in a routine test. The general pattern of the influence of packing between the platen and the specimen on the strength of cylinders with different values of *h/d* is shown in Fig. 12.6.

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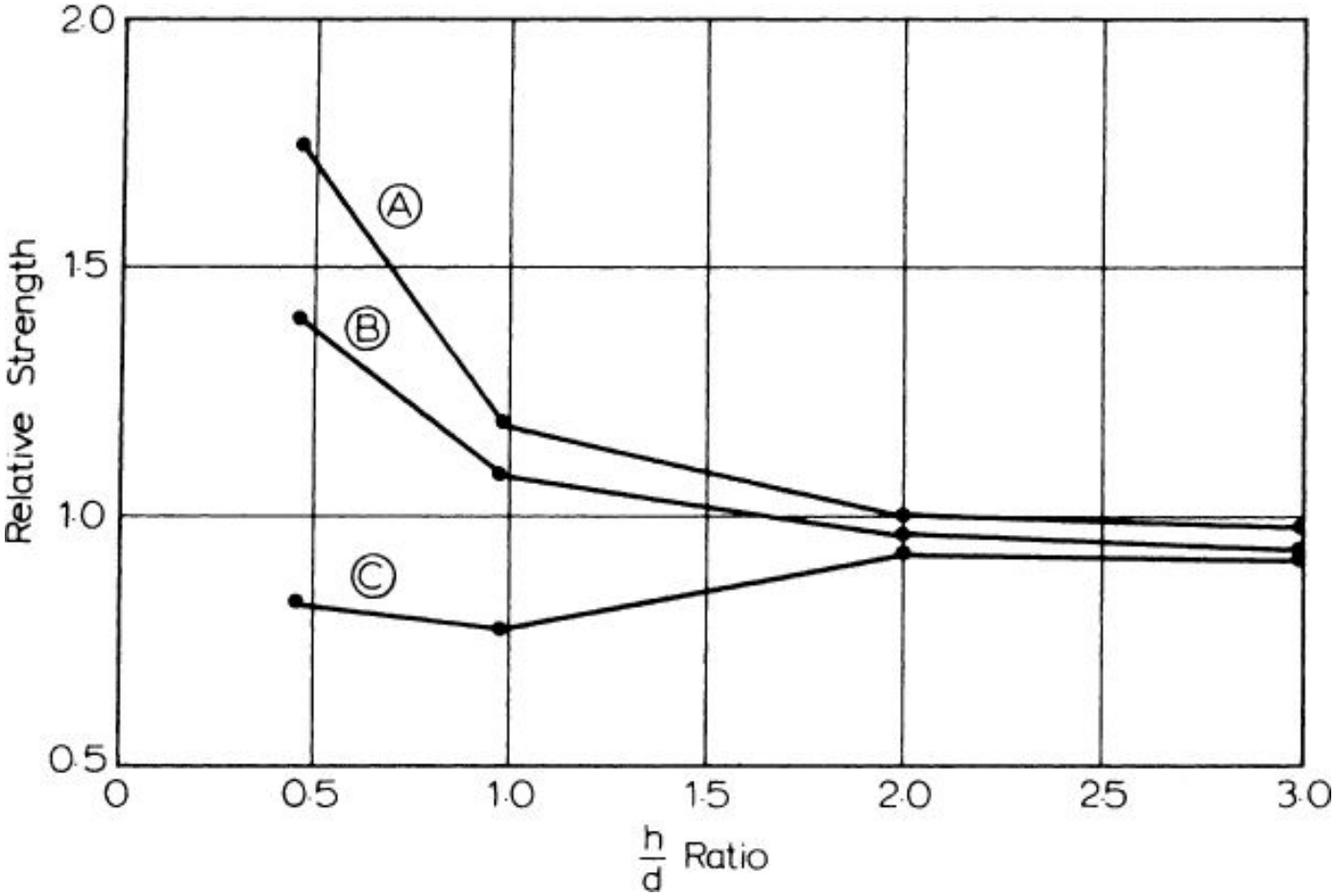


Fig. 12.6. Relative strength of cylinders of different height/diameter ratios with various types of packing between the platens and the specimen.[12.58] (Strength of cylinder with *h/d* = 2 and no packing taken as 1.0): (*A*) no packing; (*B*) 8 mm ($\frac{5}{16}$ in.) soft wallboard; (C) 25 mm (1 in.) plastic board

The end effect decreases more rapidly the more homogeneous the material; it is thus less noticeable in mortars and probably also in light-

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weight aggregate concrete of low or moderate strength where a lower heterogeneity arises from the smaller difference between the elastic moduli of the cement paste and the aggregate than is the case with normal weight aggregate. It has been found that, with lightweight aggregate concrete, the value of the ratio of strengths of a standard cylinder to a cylinder with a height-diameter ratio of 1 is between 0.95 and 0.97.[12.15,12.60] This has, however, not been confirmed in Russian tests on concrete made with expanded clay aggregate where a ratio of about 0.77 was reported.[12.59]

Comparison of strengths of cubes and cylinders

We have seen that the restraining effect of the platens of the testing machine extends over the entire height of a cube but leaves unaffected a part of a test cylinder. It is, therefore, to be expected that the strengths of cubes and cylinders made from the same concrete differ from one another.

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According to the expressions converting the strength of cores into the strength of equivalent cubes in BS 1881 : 1983 (withdrawn), the strength of cylinder is equal to 0.8 of the strength of a cube but, in reality, there is no simple relation between the strengths of the specimens of the two shapes. The ratio of the strengths of the cylinder to the cube increases strongly with an increase in strength[12.16] and is nearly 1 at strengths of more than 100 MPa (or 14 000 psi). Some other factors, for example, the moisture condition of the specimen at the time of testing, have also been found to affect the ratio of strengths of the two types of specimens.

Because European Standard BS EN 206-1 : 2000 recognizes the use of both cylinders and cubes, it includes a table of equivalence of strengths of the two types of compression specimens up to 50 MPa (measured on cylinders). The values of the cylinder/cube strength ratio are all around 0.8. The CEB–FIP Design Code[12.1] gives a similar table of equivalence but, above 50 MPa,

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the cylinder/cube strength ratio rises progressively, reaching 0.89 when the cylinder strength is 80 MPa. Neither of these tables should be used for purposes of conversion of a measured strength of one type of specimen to the strength of the other type. For any one construction project, a single type of compressive strength test specimen should be used.

It is difficult to say which type of specimen, cylinder or cube, is 'better' but, even in countries where cubes are the standard specimen, there seems to be a tendency, at least for research purposes, to use cylinders rather than cubes, and this has been recommended by RILEM (Réunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions) – an international organization of testing laboratories. Cylinders are believed to give a greater uniformity of results for nominally similar specimens because their failure is less affected by the end restraint of the specimen; their strength is less influenced by the properties of the

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coarse aggregate used in the mix; and the stress distribution on horizontal planes in a cylinder is more uniform than on a specimen of square cross-section.

It may be recalled that cylinders are cast and tested in the same position, whereas in a cube the line of action of the load is at right angles to the axis of the cube as-cast. In structural compression members, the situation is similar to that existing in a test cylinder, and it has been suggested that, for this reason, tests on cylinders are more realistic. The relation between the directions as-cast and as-tested has, however, been shown not to affect appreciably the strength of cubes made with unsegregated and homogeneous concrete[12.3] (Fig. 12.7). Moreover, as shown earlier, the stress distribution in any compression test is such that the test is only comparative and offers no quantitative data on the strength of a structural member.

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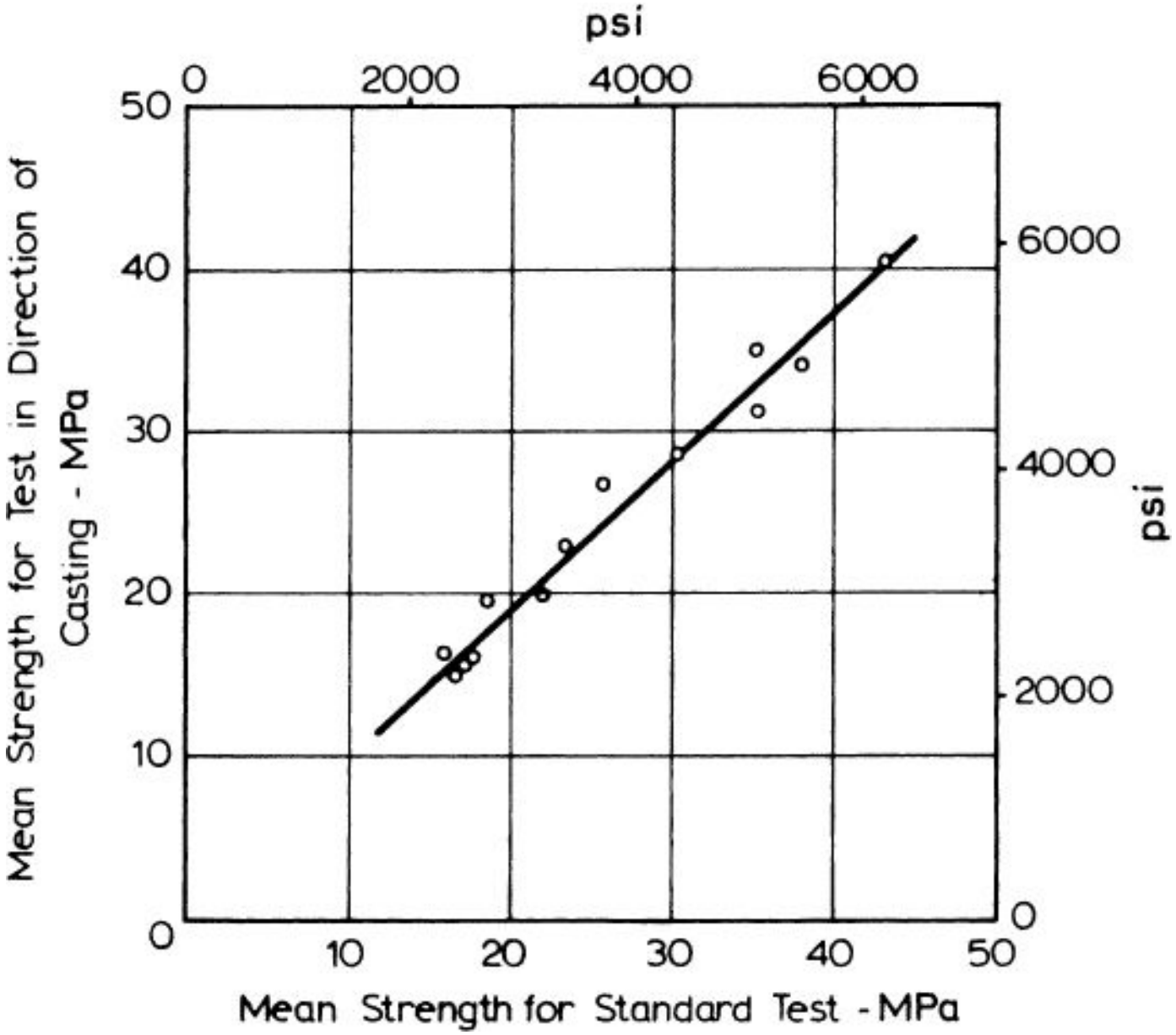


Fig. 12.7. Relation between mean strength of concrete cubes loaded in the direction of casting and in the standard manner[12.3]

Tests for strength in tension

Although concrete is not normally designed to resist direct tension, the knowledge of tensile

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strength is of value in estimating the load under which cracking will develop. The absence of cracking is of considerable importance in maintaining the continuity of a concrete structure and in many cases in the prevention of corrosion of reinforcement. Cracking problems occur when diagonal tension arising from shearing stresses develops, but the most frequent case of cracking is due to restrained shrinkage and temperature gradients. An appreciation of the tensile strength of concrete helps in understanding the behaviour of reinforced concrete even though the actual design calculations do not in many cases explicitly take the tensile strength into account. The rather extensive topic of cracking is considered in Chapter 10.

Strength in tension is of interest also in unreinforced concrete structures, such as dams, under earthquake conditions. Other structures, such as highway and airfield pavements, are designed on the basis of flexural strength, which involves strength in tension.

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There are three types of test for strength in tension: direct tension test, flexure test, and splitting tension test.

A direct application of a pure tension force, free from eccentricity, is very difficult. Despite some success with the use of lazy-tong grips,[12.19] it is difficult to avoid secondary stresses such as those induced by grips or by embedded studs. A direct tension test, using bonded end plates, is prescribed by the U.S. Bureau of Reclamation.[12.17] The other two types of test for strength in tension are considered below.

Flexural strength tests

In these tests, a plain (unreinforced) concrete beam is subjected to flexure using symmetrical *two-point loading* until failure occurs. Because the load points are spaced at one-third of the span, the test is called a *third-point loading* test. The theoretical maximum tensile stress reached in the bottom fibre of the test beam is known as the *modulus of rupture.*

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Beams are normally tested on their side in relation to the as-cast position but, provided the concrete is unsegregated, the position of the beam as tested relative to the as-cast position does not affect the modulus of rupture.[12.22,12.23]

British Standard BS EN 12390-5 : 2000 prescribes third-point loading on 150 by 150 by 750 mm (6 by 6 by 30 in.) beams supported over a span of 450 mm (18 in.) but 100 by 100 mm (4 by 4 in.) beams can also be used, provided the beam side is at least three times the maximum size of the aggregate.

The requirements of ASTM C 78-09 are similar to those of BS EN 12390-5 : 2009. If fracture occurs within the central one-third of the beam, the modulus of rupture is calculated on the basis of ordinary elastic theory, and is thus equal to:

$$PL/(bd^2),$$

where P = maximum total load on the beam

L = span

b = width of the beam, and

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d = depth of the beam.

If, however, fracture occurs outside the load points, say, at a distance a from the near support, a being the average distance measured on the tension surface of the beam, but not more than 5 per cent of the span, then the modulus of rupture is given by $3Pa/(bd^2)$. This means that the maximum tensile stress at the critical section, and not the maximum stress on the beam, is considered in the calculations. The British approach is to disregard failure outside the middle-third of the beam.

There exists also a test for flexural strength under *centre-point loading*, prescribed in ASTM C 293-08, and also in British Standard BS EN 12390-5 : 2000. In this test, failure occurs when the tensile strength of concrete in the extreme fibre immediately under the load point is exhausted. On the other hand, under third-point loading, one-third of the length of the extreme fibre in the beam is subjected to the maximum stress, so that the critical crack may develop at

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any section in one-third of the beam length. Because the probability of a weak element (of any specified strength) being subjected to the critical stress is considerably greater under two-point loading than when a central load acts, the centre-point loading test gives a higher value of the modulus of rupture,[12.20] but also a more variable one. In consequence, the centre-point loading test is very rarely used.

The expression for the modulus of rupture, given earlier in this section, was qualified by the term 'theoretical' because it is based on the *elastic* beam theory, in which the stress–strain relation is assumed to be linear, so that the tensile stress in the beam is assumed to be proportional to the distance from its neutral axis. In fact, as discussed in Chapter 9, there is a gradual increase in strain with an increase in stress above about one-half of the tensile strength. In consequence, the shape of the actual stress block under loads nearing failure is parabolic, and not triangular. The modulus of rupture thus overestimates the tensile strength

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of concrete: Raphael[12.52] showed that the correct value of tensile strength is about $\frac{3}{4}$ of the theoretical modulus of rupture (see Fig. 12.8).

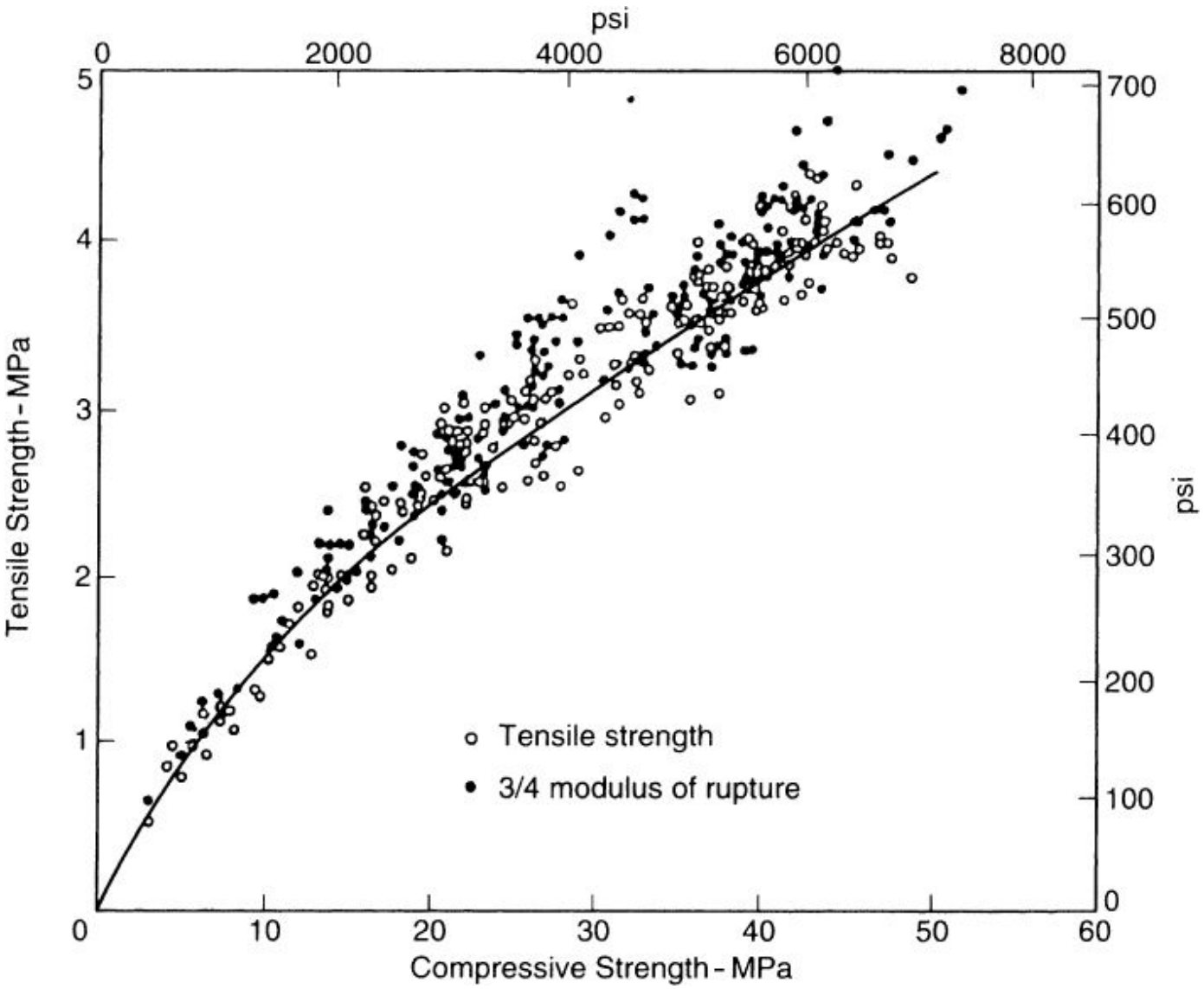


Fig. 12.8. Plot of the splitting-tension strength and of of the modulus of rupture against compressive strength of concrete (based on ref. 12.52)

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There exist additional possible reasons why the modulus of rupture test gives a higher value of strength than a direct tension test made on the same concrete. First, any accidental eccentricity in a direct tension test results in a lower apparent strength of the concrete. The second argument is similar to that justifying the influence of the loading arrangement on the value of the modulus of rupture: under direct tension, the entire volume of the specimen is subjected to the maximum stress, so that the probability of a weak element occurring is high. Third, in the flexure test, the maximum fibre stress reached may be higher than in direct tension because the propagation of a crack is blocked by less-stressed material nearer to the neutral axis. Thus, the energy available is below that necessary for the formation of new crack surfaces. These various reasons for the difference between the modulus of rupture and the direct tensile strength are not all of equal importance.

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At the beginning of this chapter, it was mentioned that flexural strength of concrete is of interest in the design of pavement slabs. However, the flexure test is not convenient for control or compliance purposes because the test specimens are heavy and are easily damaged. Also, the outcome of the flexure test is strongly affected by the moisture conditions of the specimen (see p. 602) and, more generally, the variability of the modulus of rupture is large.[12.115] It is, therefore, convenient to establish experimentally a relation between the modulus of rupture and cylinder compressive strength, and to use the latter in routine testing.[12.2] The relation between tensile and compressive strengths is discussed on p. 310.

A wide range of tests[12.131] has shown a linear relation between the modulus of rupture and the splitting tensile strength at a given age. This finding is of value if the strength of pavement concrete in situ has to be determined: cutting cores as well as testing them, in compression or in splitting tension, is very much easier than cutting out

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beams for the modulus of rupture test. Moreover, cores are frequently cut anyway for the purpose of verifying the thickness of pavement.

Splitting tension test

In this test, a concrete cylinder, of the type used for compression tests, is placed with its axis horizontal between the platens of a testing machine, and the load is increased until failure by indirect tension in the form of splitting along the vertical diameter takes place.

If the load is applied along the generatrix, then an element on the vertical diameter of the cylinder (Fig. 12.9) is subjected to a vertical compressive stress of:

$$\frac{2P}{\pi LD}\left[\frac{D^2}{r(D-r)}-1\right]$$

and a horizontal tensile stress of $2P/(\pi LD)$

where P = compressive load on the cylinder

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L = length of the cylinder

D = diameter, and

r and $(D - r)$ = distances of the element from the two loads respectively.

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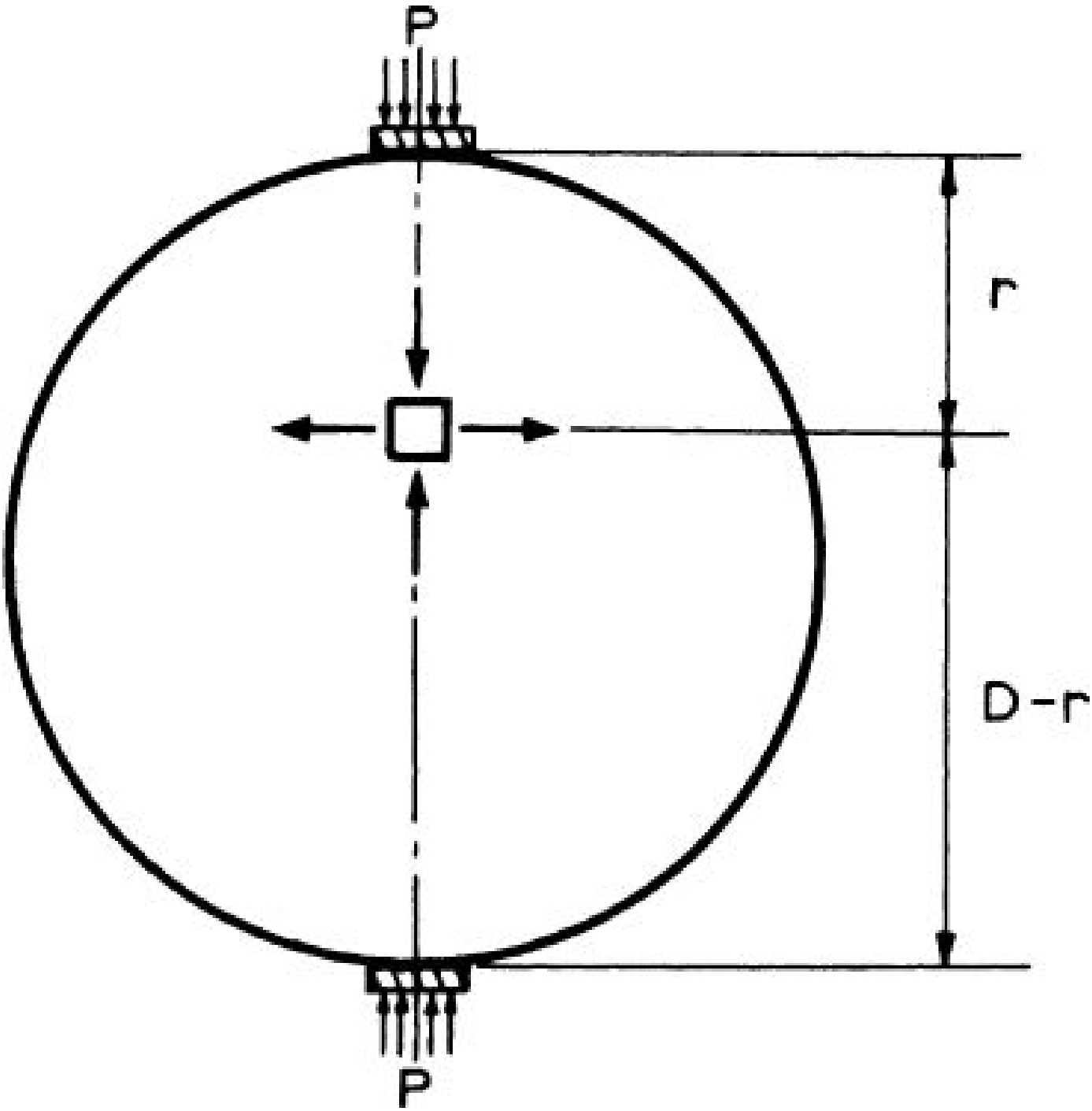


Fig. 12.9. The splitting test

However, immediately under the load, a high compressive stress would be induced and, in practice, narrow strips of a packing material, such as plywood, are interposed between the cylinder

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and the platens. Without packing strips, the recorded strength is lower, typically by 8 per cent. ASTM C 496-04 prescribes plywood strips, 3 mm ($\frac{1}{8}$ in.) thick and 25 mm (1 in.) wide. British Standard BS EN 12390-6 : 2009 specifies hardboard strips, 4 mm thick and 15 mm wide. With such an arrangement, the distribution of the horizontal stress on a section containing the vertical diameter is as shown in Fig. 12.10. The stress is expressed in terms of $2P/(\pi LD)$, and it can be seen that a high horizontal compressive stress exists in the vicinity of the loads but, as this is accompanied by a vertical compressive stress of comparable magnitude, thus producing a state of biaxial stress, failure in compression does not take place.

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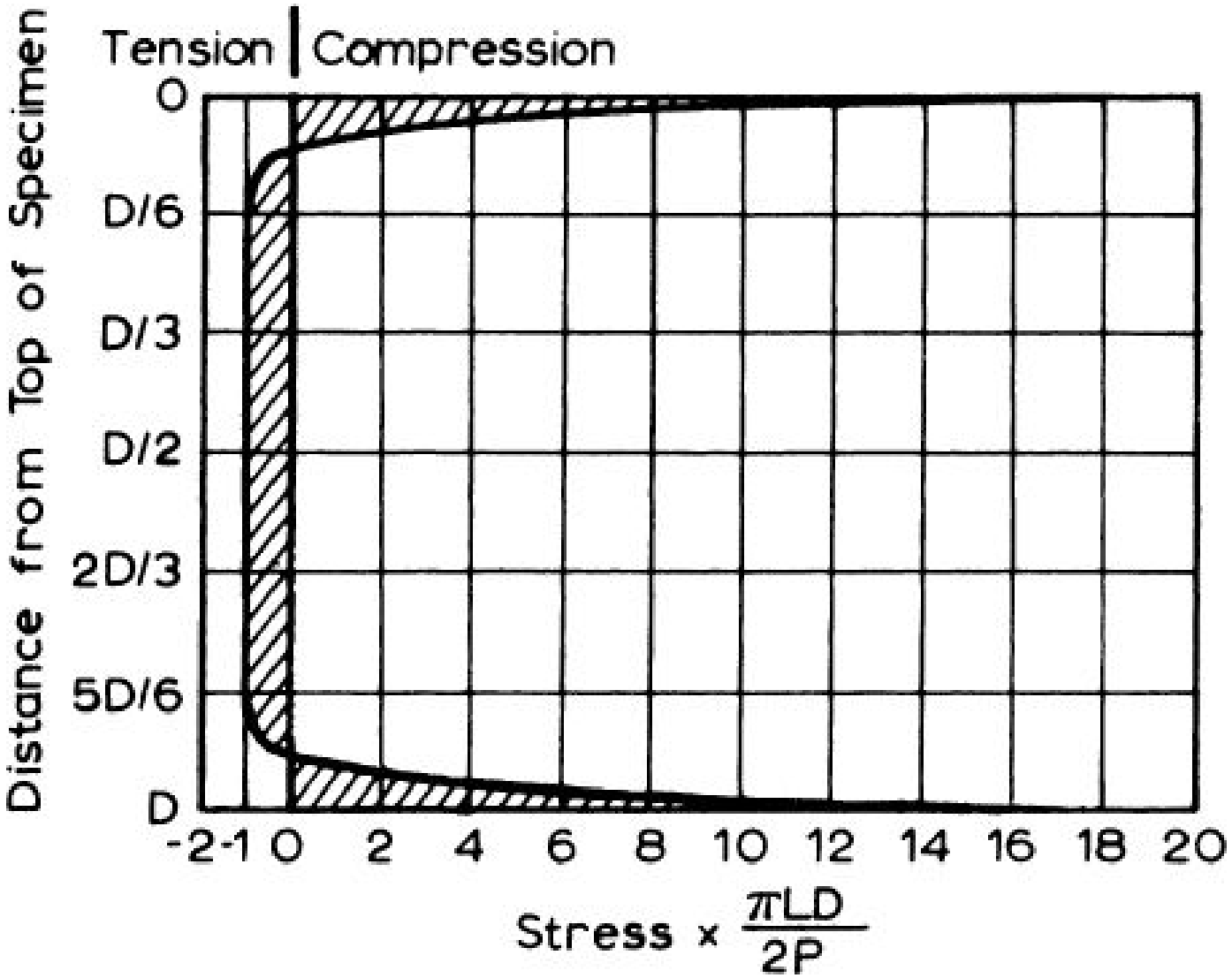


Fig. 12.10. Distribution of horizontal stress in a cylinder loaded over a width equal to $\frac{1}{12}$ of the diameter[12.24] (Crown copyright)

During the splitting test, the platens of the testing machine should not be allowed to rotate in a plane perpendicular to the axis of the cylinder, but a slight movement in the vertical plane containing the axis should be permitted in order

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to accommodate a possible non-parallelism of the generatrices of the cylinder. This can be achieved by means of a simple roller arrangement interposed between one platen and the cylinder. The rate of loading is prescribed by ASTM C 496-04 and by BS EN 12390-6 : 2009.

Cubes and prisms can also be subjected to the splitting test, the load being applied through loading pieces resting against the cube on centre lines of two opposing faces. The cube test, covered by BS 1881 : 117 : 1983 (replaced by BS EN 12390-6 : 2009), gives the same result as the splitting test on a cylinder,[12.144] viz. the horizontal tensile stress is equal to $2P/(\pi a^2)$ where a is the side of the cube. This means that only the concrete within a cylinder inscribed in the cube resists the applied load.

An advantage of the splitting test is that the same type of specimen can be used for both the compression and the tension tests. Therefore, the splitting cube test is of interest only in countries where the cube and not the cylinder is used as

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a standard compression specimen; few data are available on the performance of the splitting cube test.

The splitting test is simple to perform and gives more uniform results than other tension tests.[12.24] The strength determined in the splitting test is believed to be close to the direct tensile strength of concrete, being 5 to 12 per cent higher. It has been suggested, however, that, in the case of mortar and lightweight aggregate concrete, the splitting test yields too low a result. With normal aggregate, the presence of large particles near the surface to which the load is applied may influence the behaviour.[12.86]

It may be noted that, according to ACI 318-08[12.124] splitting tensile strength should not be used for the purpose of establishing conformity.

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Influence on strength of moisture condition during test

The British as well as ASTM Standards require that all the test specimens be tested in a 'wet' or 'moist' condition. This condition has the advantage of being better reproducible than a 'dry condition' which includes widely varying degrees of dryness.

Occasionally, a test specimen may not be in a wet condition, and it is of interest to consider what are the consequences of such departure from the standard. It should be emphasized that only the condition immediately prior to the test is considered, it being assumed that usual curing has been applied in all cases.

As far as compressive strength specimens are concerned, testing in a dry condition leads to a higher strength. It has been suggested[12.51] that drying shrinkage at the surface induces a biaxial compression on the core of the specimen, thus increasing its strength in the third direction, that

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is, in the direction of the applied load. However, tests have shown that well-cured mortar prisms[12.50] and concrete cores,[12.121] when completely dried, had a higher compressive strength than when tested wet. These specimens were not subject to differential shrinkage so that there was no biaxial stress system induced. The behaviour of the specimens, described above, accords also with the suggestion[12.32] that the loss of strength due to wetting of a compression test specimen is caused by the dilation of the cement gel by adsorbed water: the forces of cohesion of the solid particles are then decreased. Conversely, when on drying the wedge-action of water ceases, an apparent increase in strength of the specimen is recorded. The effects of water are not merely superficial as dipping the specimens in water has much less influence on strength than soaking. On the other hand, soaking concrete in benzene or paraffin, known not to be adsorbed by the cement gel, has no influence on strength. Re-soaking oven-dried specimens in water reduces their

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strength to the value of continuously wet-cured specimens, provided they have hydrated to the same degree.[12.32] The variation in strength due to drying appears thus to be a reversible phenomenon.

The quantitative influence of drying varies: with 34 MPa (5000 psi) concrete, an increase in compressive strength up to 10 per cent has been reported[12.33] on thorough drying, but if the drying period is less than 6 hours, the increase is generally less than 5 per cent. Other tests have shown the decrease in strength, in consequence of 48-hour wetting prior to test, to be between 9 and 21 per cent.[12.49]

Beam specimens tested in flexure exhibit a behaviour opposite to that of compression test specimens: a beam which has been allowed to dry before testing has a lower modulus of rupture than a similar specimen tested in a wet condition.[12.109] This difference is due to the tensile stresses induced by restrained shrinkage prior to

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the application of the load which induces tension in the extreme fibre. The magnitude of the apparent loss of strength depends on the rate at which moisture evaporates from the surface of the specimen. It should be emphasized that this effect is distinct from the influence of curing on strength.

If, however, the test specimen is small and drying takes place very slowly, so that internal stresses can be redistributed and alleviated by creep, an increase in strength is observed. This was found in tests on concrete beams,[12.31] and also on mortar briquettes.[12.30] Conversely, wetting a completely dry specimen prior to testing reduces its strength;[12.31] interpretation of this phenomenon is controversial.[12.128]

The strength of cylinders tested in splitting tension is not affected by the moisture condition because failure occurs in a plane remote from the surface subjected to wetting or drying.

The temperature of the specimen at the time of testing (as distinct from the curing temperature)

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affects the strength, a higher temperature leading to a lower indicated strength, both in the case of compression and of flexure specimens (Fig. 12.11).

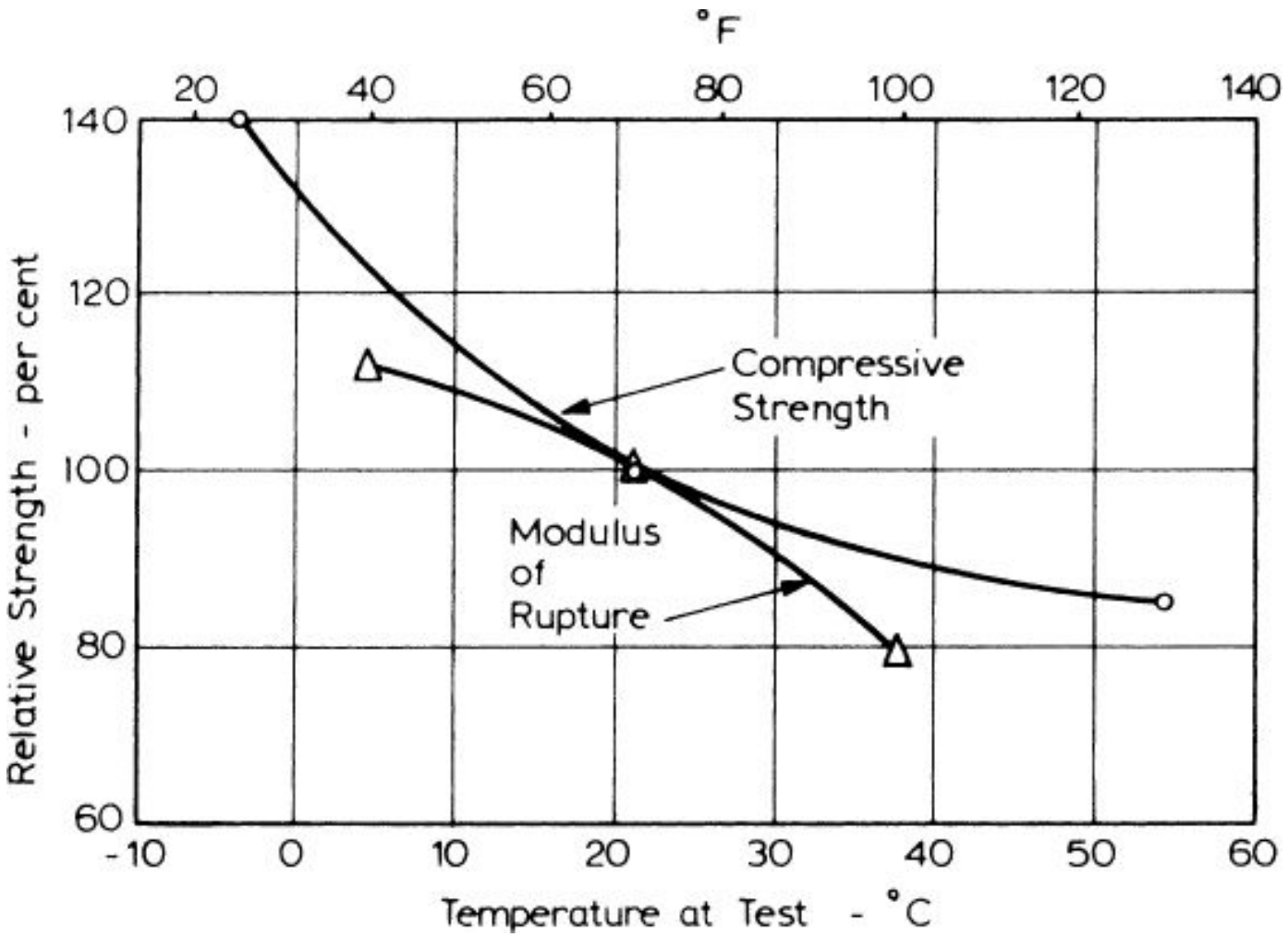


Fig. 12.11. Influence of temperature at the time of testing on strength

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Influence of size of specimen on strength

The size of test specimens for strength testing is prescribed in the relevant standards, but occasionally more than one size is permitted. Moreover, from time to time arguments in favour of use of smaller specimens are advanced. These point out their advantages: smaller specimens are easier to handle and are less likely to be accidentally damaged; the moulds are cheaper; a lower capacity testing machine is needed; and less concrete is used, which in the laboratory means less storage and curing space, and also a smaller quantity of aggregate to be processed.[12.41] On the other hand, the size of the test specimen may affect the resulting strength and also the variability of test results. For these reasons, it is important to consider in detail the influence of the size of specimen on strength test results.

The discussion on p. 292 showed that concrete is composed of elements of variable strength so that it is reasonable to assume that the larger the

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volume of the concrete subjected to stress the more likely it is to contain an element of a given extreme (low) strength. As a result, the measured strength of a specimen decreases with an increase in its size, and so does the variability in strength of geometrically similar specimens. Because the influence of size on strength depends on the standard deviation of strength (Fig. 12.12) it follows that the size effects are smaller the greater the homogeneity of the concrete. Thus, the size effect in lightweight aggregate concrete should be smaller, but this has not been confirmed with any degree of certainty, although there is some support for this suggestion in the available data.[12.76] Figure 12.12 can also explain why the size effect virtually disappears beyond a certain size of the specimen: for each successive ten fold increase in size of the specimen it loses progressively a smaller amount of strength.

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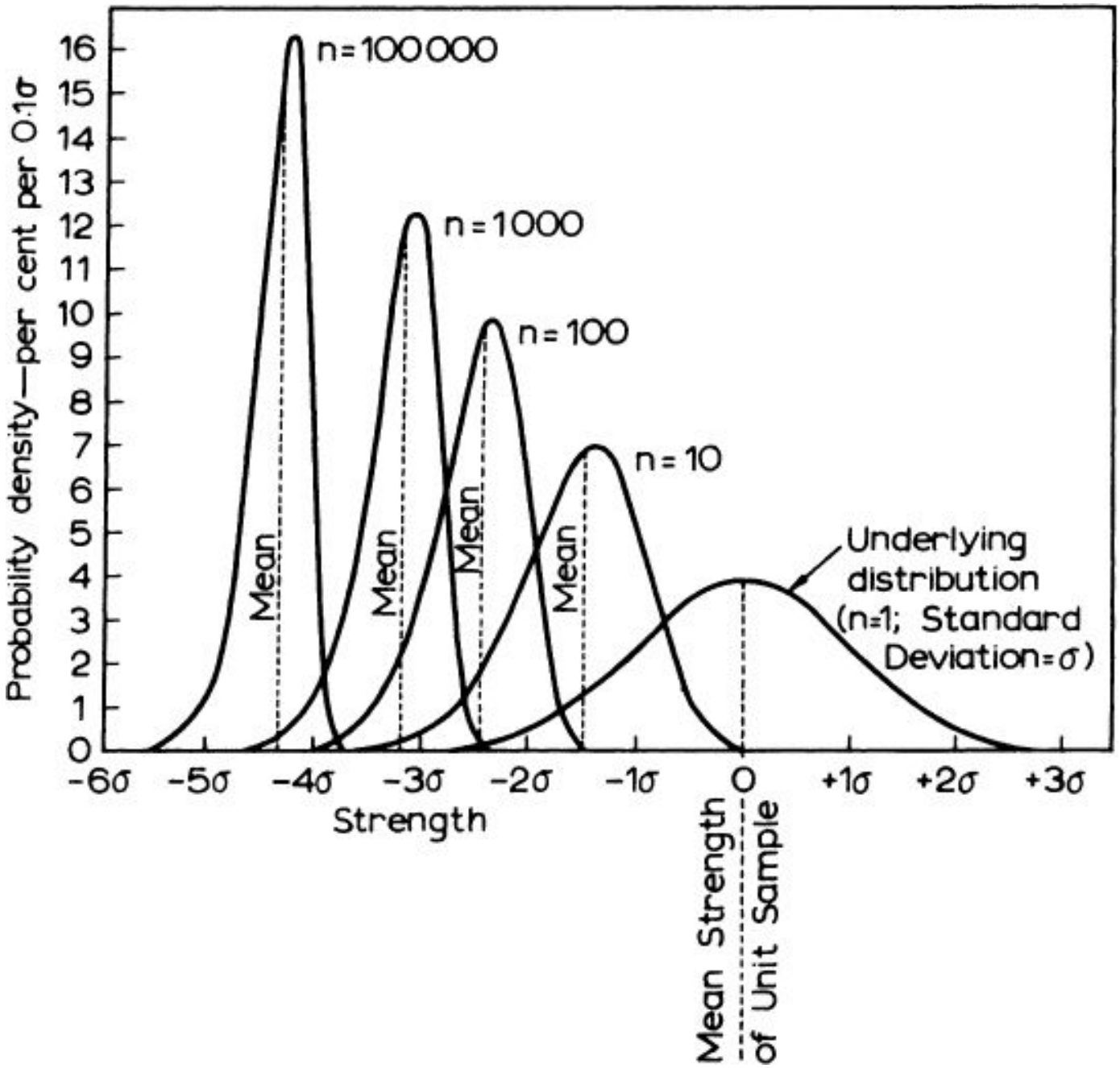


Fig. 12.12. Strength distribution in samples of size *n* for an underlying normal distribution[12.34]

On p. 292, the concept of the weakest link was discussed; to use this concept we require

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the knowledge of the distribution of extreme values in samples of a given size, drawn at random from a parent population with a given distribution of strength. This distribution is generally not known, and certain assumptions regarding its form have to be made. Here, it will suffice to give Tippett's[12.34] data on the variation in strength and standard deviation of samples of size n in terms of the strength and standard deviation of a sample of unit size, when the unit sample has a *normal distribution* of strength. Figure 12.12 shows this variation in strength for samples when n equals 10, 10^2, 10^3, and 10^5.

In the case of tests on the strength of concrete, we are interested in the averages of extremes as a function of the size of the specimen. Average values of samples chosen at random tend to have a normal distribution, so that the assumption of this type of distribution, when average values of samples are used, does not introduce serious error, and has the advantage of simplifying the computations. In some practical cases, a skew-

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ness of distribution has been observed; this may not be due to any 'natural' properties of concrete but to the rejection of poor quality concrete on the site so that such concrete never reaches the testing stage.[12.35] A full treatment of the statistical aspects of testing is outside the scope of this book.[*–]

[*–] See J. B. KENNEDY and A. M. NEVILLE, *Basic Statistical Methods for Engineers and Scientists*, 3rd Ed. (New York and London, Harper and Row, 1986).

Size effects in tensile strength tests

Figure 12.12 shows that both the mean strength and dispersion decrease with an increase in the size of the specimen. Experimental results have confirmed this pattern of behaviour in modulus of rupture tests[12.20,12.23] (see Figs 12.13 and 12.14) and also in direct tension[12.19] and in indirect tension.[12.64]

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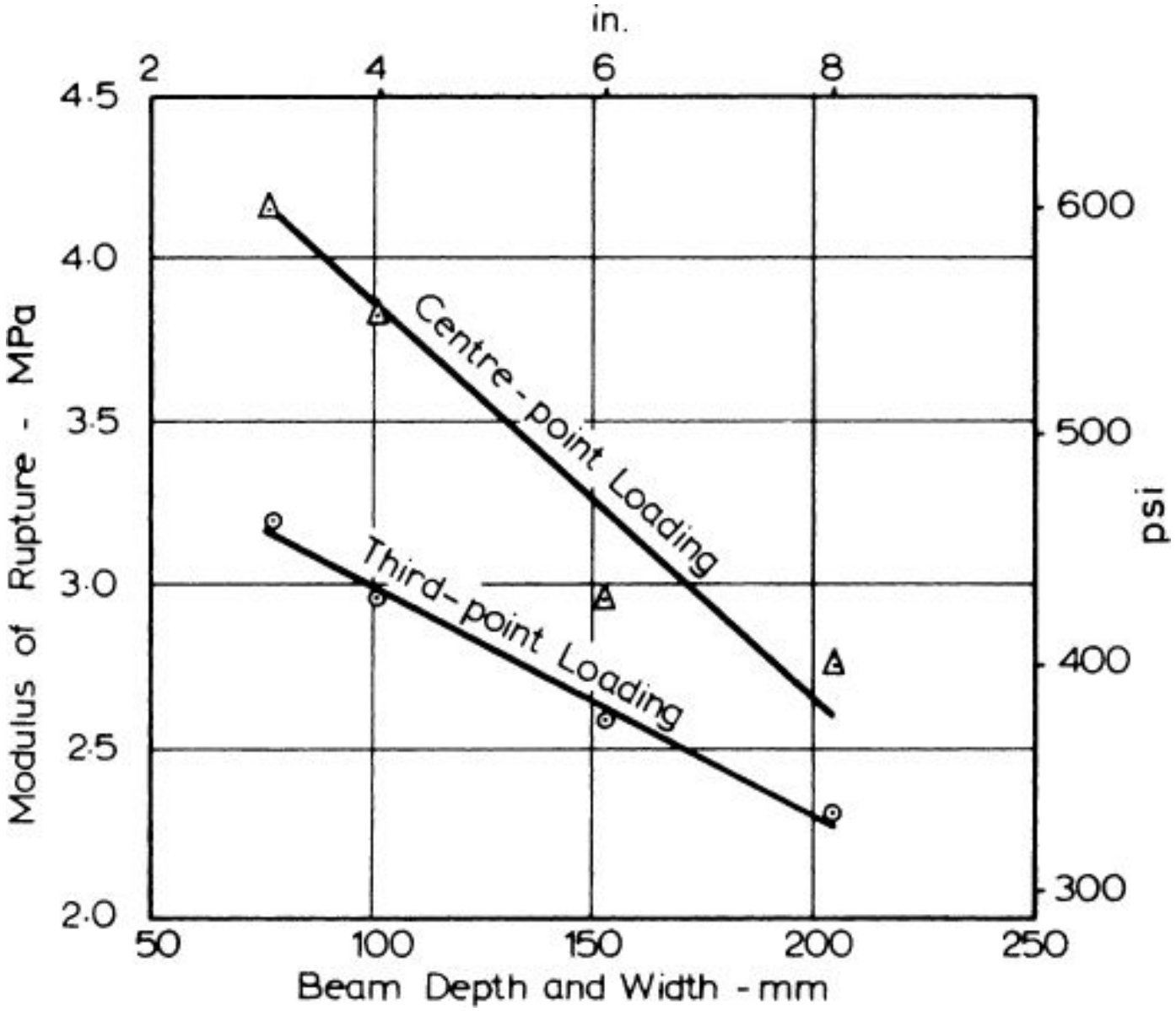


Fig. 12.13. Modulus of rupture of beams of different sizes subjected to centre-point and third-point loading[12.20] (Crown copyright)

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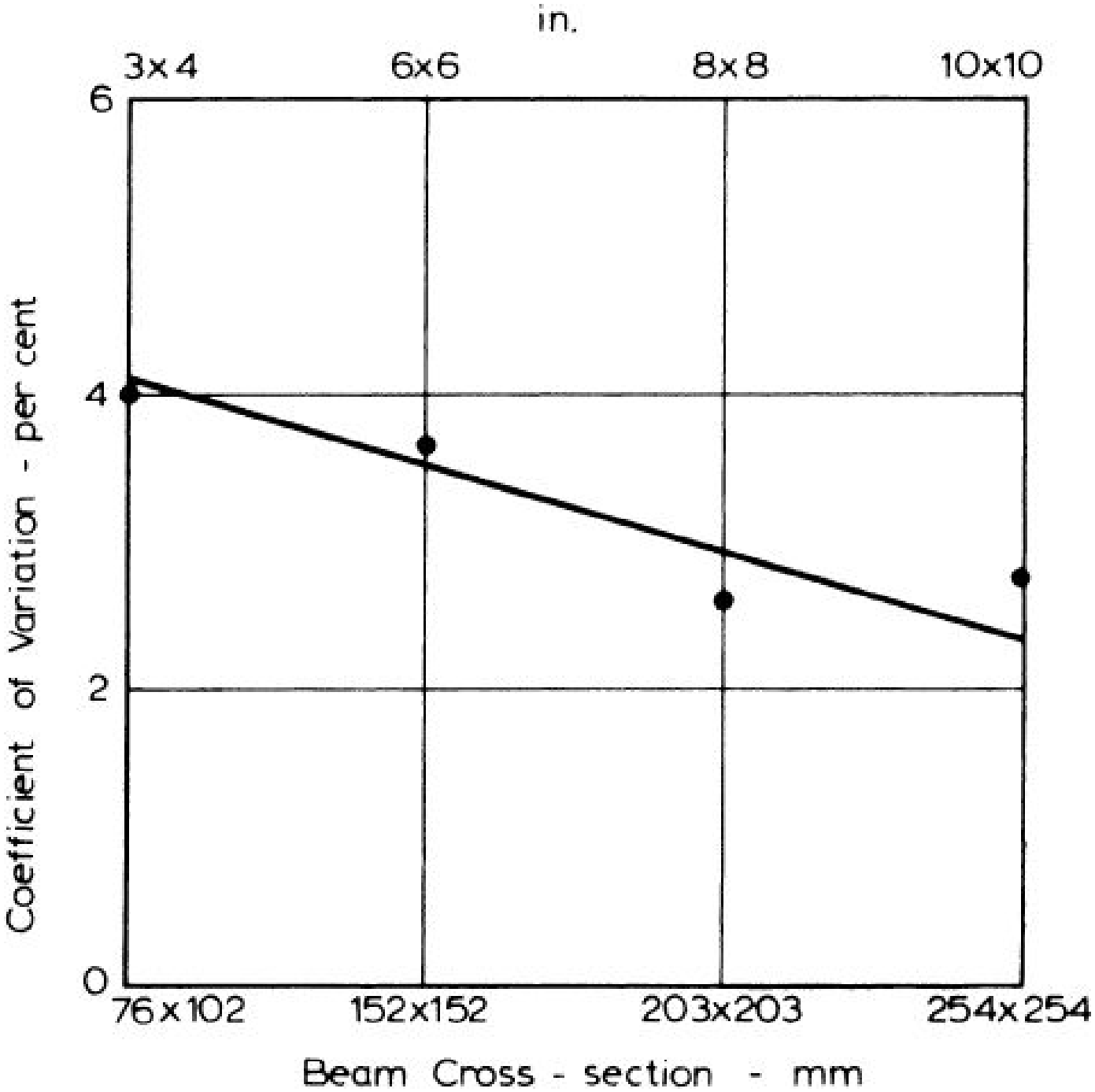


Fig. 12.14. Coefficient of variation of the modulus of rupture for beams of different sizes[12.23]

Direct tension tests on cylinders of concretes with compressive strengths between 35 and 128

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MPa (5000 and 18 500 psi) were performed by Rossi *et al.*[12.97] They confirmed the decrease in tensile strength and also in variability of test results with an increase in size: the decrease in strength is larger the lower the strength of concrete (see Fig. 12.15). The coefficient of variation also decreases with an increase in size of the specimen, as shown in Fig. 12.16, but there is no apparent effect of the strength of concrete on this relation. Rossi *et al.*[12.97] explain this influence of strength in terms of the heterogeneity of the mix components. Specifically, the size effect is a function of the ratio of the specimen size to the maximum size of aggregate and of the difference in strength between the aggregate particles and the surrounding mortar. This difference is small in very high strength concrete and also in lightweight aggregate concrete.[12.97]

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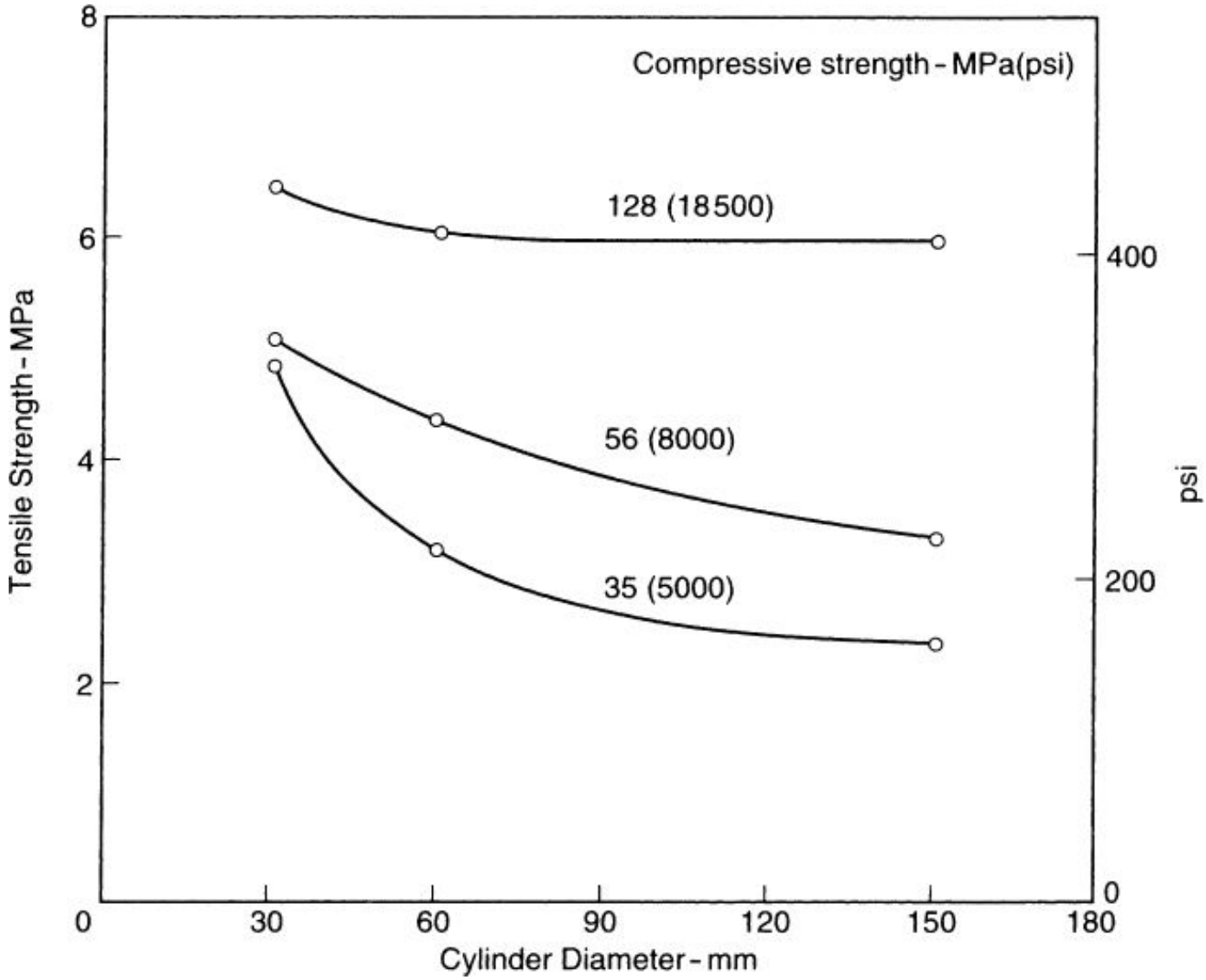


Fig. 12.15. Direct tensile strength of concrete cylinders tested by Rossi *et al.*[12.97] plotted as a function of cylinder diameter

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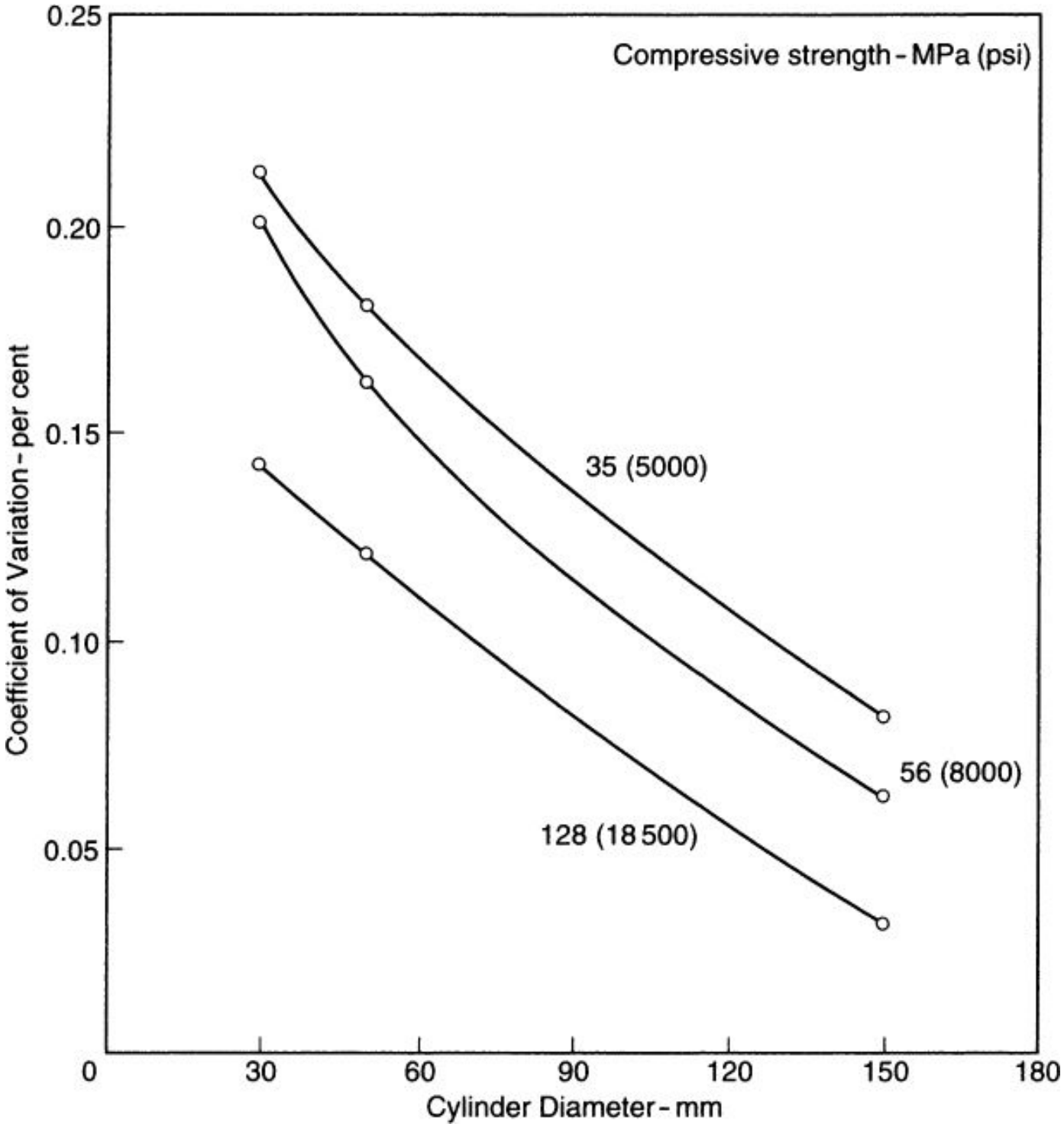


Fig. 12.16. Coefficient of variation of direct tensile strength of cylinders tested by Rossi *et al.*[12.97] plotted as a function of cylinder diameter

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Splitting tension tests on 150 mm diameter by 300 mm high (6 by 12 in.) cylinders and 100 mm diameter by 200 mm high (4 by 8 in.) cylinders have given an average ratio[12.131] of the strength of the former to the latter of 0.87; the average splitting tension strength of the larger cylinders was 2.9 MPa (415 psi). The standard deviation for the larger cylinders was 0.18 MPa (26 psi) and, for the smaller, 0.27 MPa (39 psi). The coefficients of variation were, respectively, 6.2 and 8.2 per cent. It is worth observing that the coefficient of variation of the splitting tension strength of 150 by 300 mm cylinders had nearly the same value as the coefficient of variation of the modulus of rupture determined on beams with a 150 by 150 mm (6 by 6 in.) cross-section made of the same concrete.[12.131]

The influence of the cylinder size on splitting tension strength was confirmed by Bažant *et al.*[12.94] on the basis both of their own tests on mortar discs and also on the basis of tests on concrete cylinders performed by Hasegawa *et al.* In

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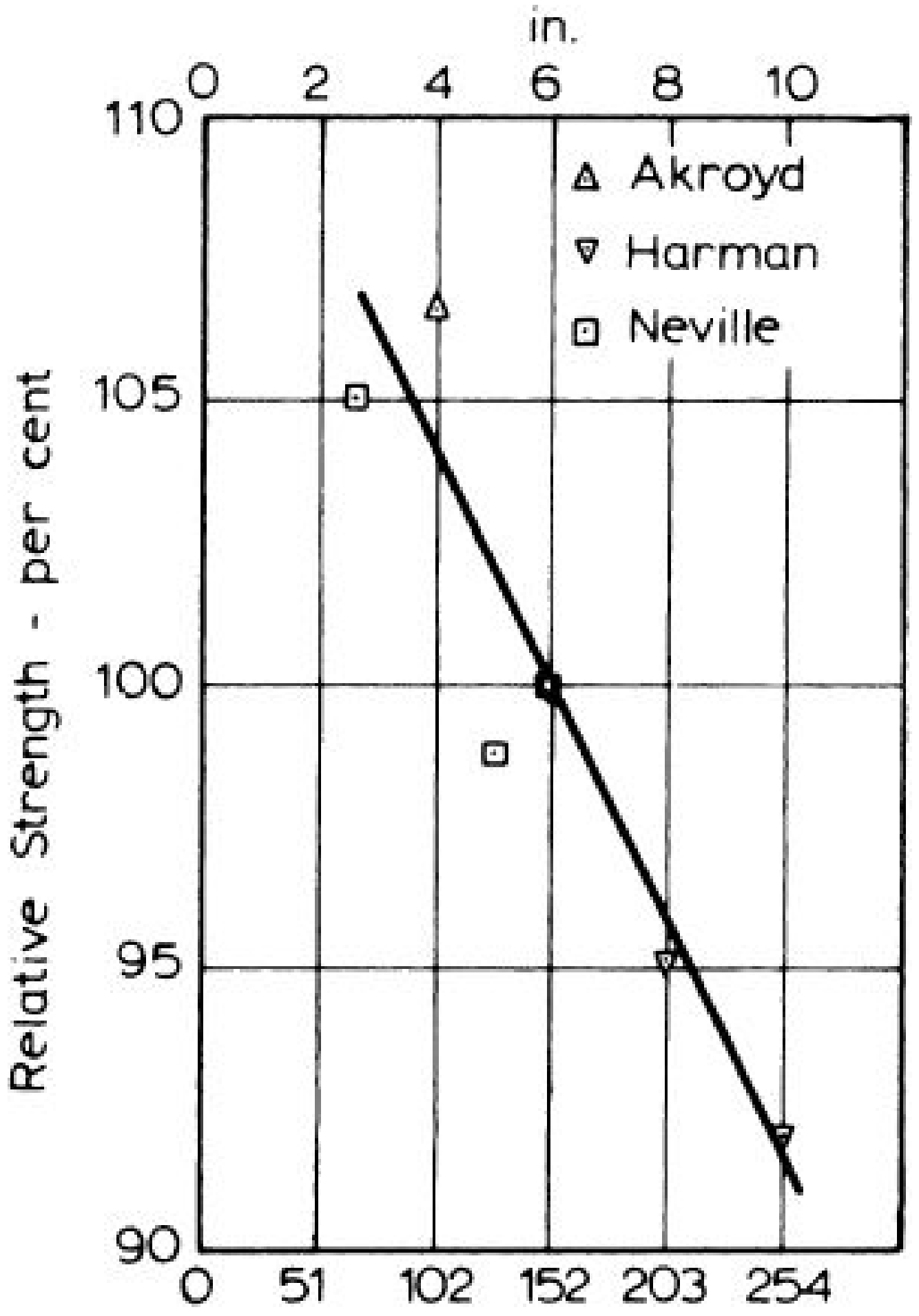
both these series of tests, the size effect disappears in large-size specimens; this topic is discussed in the next section.

Cement compacts have also been found to show the size effect when tested in splitting tension.[12.93] The same applies in the case of the ring test.[12.64]

Size effects in compressive strength tests

Let us now consider size effects in compressive strength specimens. Figure 12.17 shows the relation between mean strength and specimen size for cubes, and Table 12.2 gives the relevant values for standard deviation.[12.18] Prisms[12.36,12.37] and cylinders[12.38] exhibit a similar behaviour (Fig. 12.18). The size effects are, of course, not limited to concrete, and have been found also in anhydrite[12.39] and other materials.

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Fig. 12.17. Compressive strength of cubes of different sizes[12.35]

Table 12.2. Standard Deviation of Cubes of Different sizes[12.18]

Group	*Standard deviation for cubes of size:*					
	70.6 mm (2.78 in.)		*127 mm (5 in.)*		*152 mm (6 in.)*	
	MPa	*psi*	*MPa*	*psi*	*MPa*	*psi*
A	2.75	399	2.09	303	1.39	201
B	1.50	218	1.12	162	0.97	140
C	1.45	210	1.03	150	0.97	140
D	1.74	253	1.36	197	1.05	153

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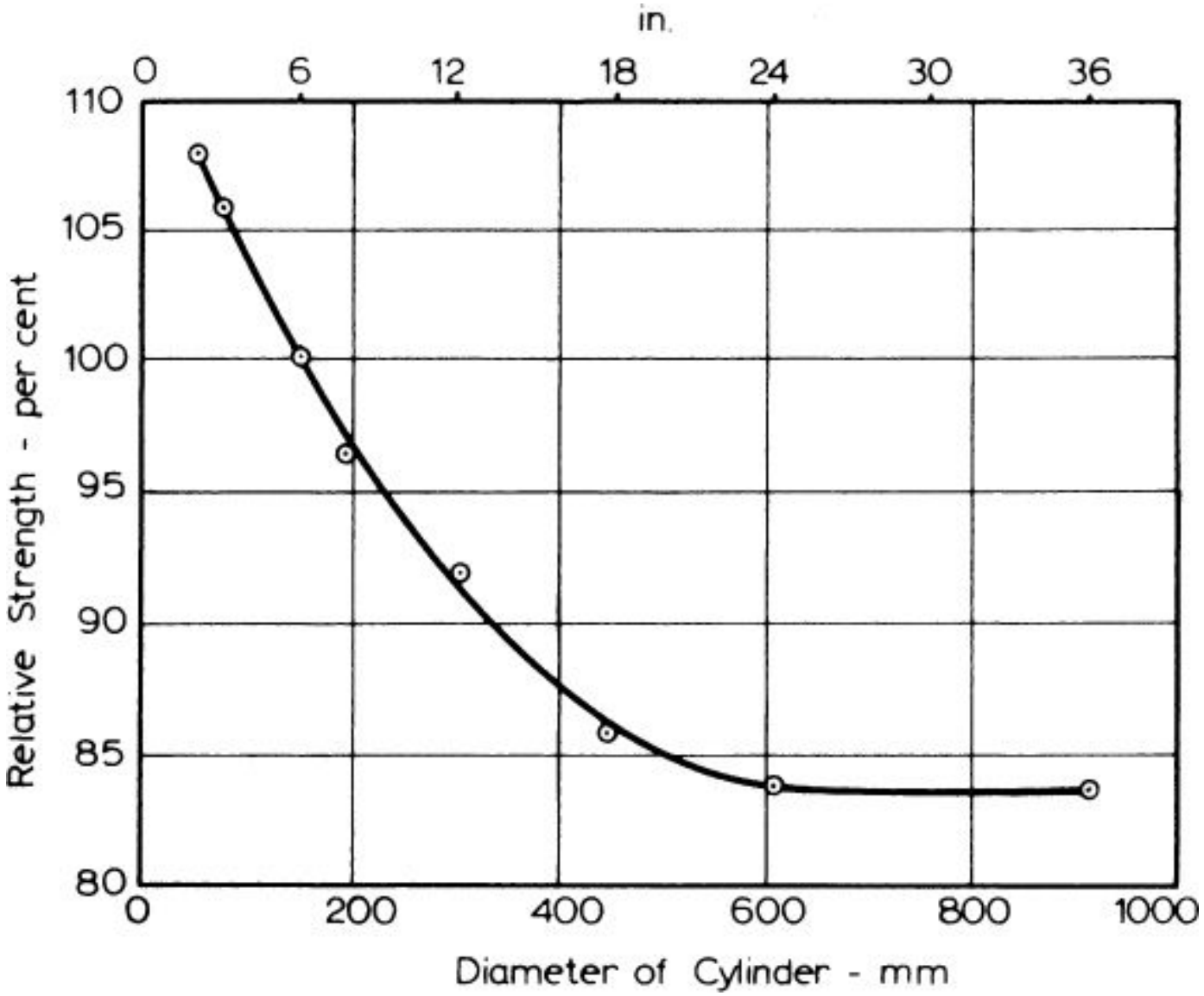


Fig. 12.18. Compressive strength of cylinders of different sizes[12.38]

It is interesting to note that the size effect disappears beyond a certain size so that a further increase in the size of a member does not lead to a decrease in strength, both in compression[12.38] and in splitting tension.[12.94] According to the U.S.

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Bureau of Reclamation,[12.77] the strength curve becomes parallel to the size axis at a diameter of 457 mm (18 in.), i.e. cylinders of 457 mm (18 in.), 610 mm (24 in.), and 914 mm (36 in.) diameter all have the same strength. The same investigation indicates that the decrease in strength with an increase in size of the specimen is less pronounced in lean mixes than in rich ones. For instance, the strength of 457 mm (18 in.) and 610 mm (24 in.) cylinders relative to 152 mm (6 in.) cylinders is 85 per cent for rich mixes but 93 per cent for lean (167 kg/m^3 (282 lb/yd^3)) mixes (cf. Fig. 12.18).

These experimental data are of importance in refuting a speculation that, if the size effect is extrapolated to very large structures, a dangerously low strength might be expected. Evidently this is not so because local failure is not tantamount to collapse.

The various test results on the size effect are of interest because size effects have been ascribed to a variety of causes: the wall effect; the ratio

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of the specimen size to the maximum aggregate size; the internal stresses caused by the difference in temperature and humidity between the surface and the interior of the specimen; the tangential stress at the contact surface between the platen of the testing machine and the specimen due to friction or bending of the platen; and the difference in the effectiveness of curing. The last suggestion, for instance, is disproved by Gonnerman's[12.40] results (Fig. 12.19) which show that specimens of different size and shape gain strength at the same rate. In this connection, Day and Haque[12.90] showed that the relation between the strength of 150 by 300 mm (6 by 12 in.) and 75 by 150 mm (3 by 6 in.) cylinders is not affected by the method of curing.

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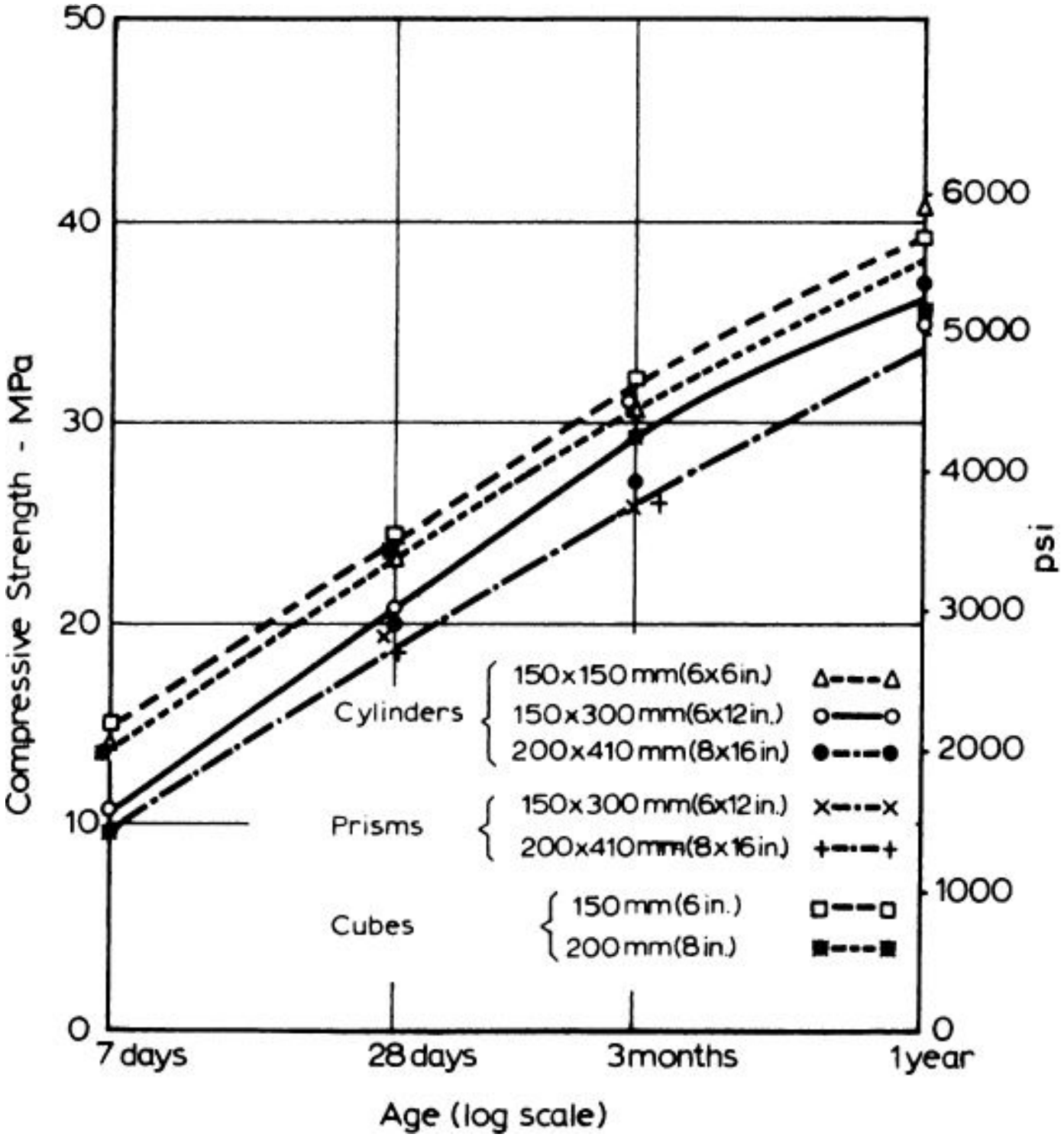


Fig. 12.19. Effect of age on the compressive strength of specimens of different shape and size[12.40] (mix 1:5 by volume)

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Within the range of sizes of specimens normally used, the effect of size on strength is not large, but it is significant and should not be ignored in work of high accuracy or in research. Analysis of numerous test data[12.65] has suggested a general relation between the compressive strength of concrete and the shape and size of the specimen in terms of $V/hd + h/d$, where V = volume of specimen, h = its height, and d = its least lateral dimension. Figure 12.20 indicates the fit of experimental data to the relation postulated. The validity of the form of the relation in high strength concrete has been confirmed.[12.148]

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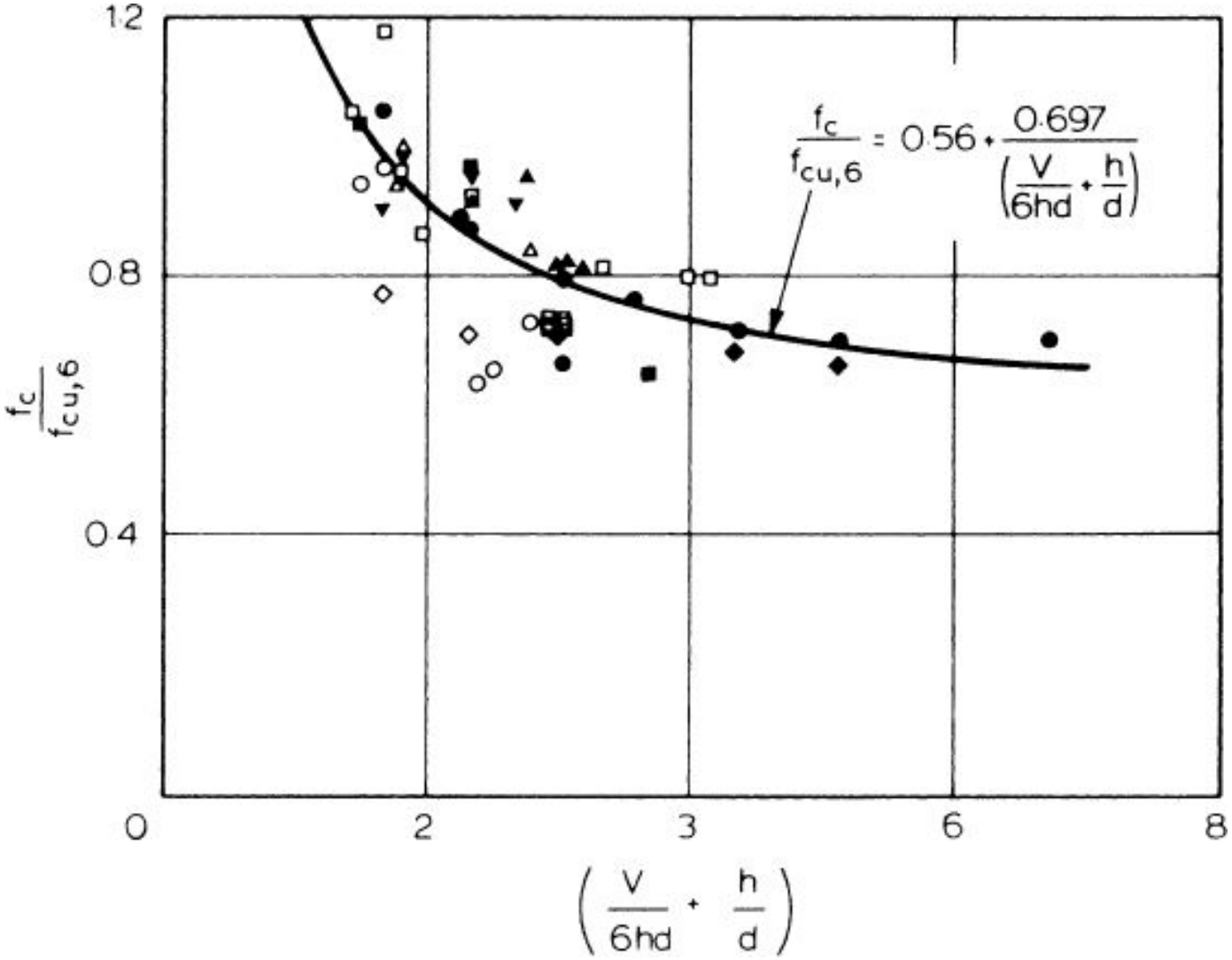


Fig. 12.20. General relation between ratio of strength of concrete specimens f_c to strength of 6 in. cube $f_{cu,6}$, and $V/6hd + h/d$, where V is volume of specimen, h its height and d is its least lateral dimension. (All dimensions in inches; in millimetre units, $f_{cu,6}$ would become $f_{cu,152}$ and on the right-hand side, 6 would be replaced by 152)

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In direct tension, strength was found to be proportional to V^n, where n varies between –0.02 and –0.04, depending on the type of aggregate.[12.91] Thus, if cylinders 150 mm (6 in.) in diameter have a strength of 1.0, 50 mm (2 in.) cylinders would have a strength of 1.05 to 1.08, and 200 mm (8 in.) cylinders 0.97 to 0.99. Prisms were found to have similar behaviour. It was also found that the coefficient of variation decreases as the specimen size increases.[12.91] Torrent[12.92] confirmed that the volume of 'highly stressed' concrete directly influences the strength of concrete in various tension tests; this description was used to denote concrete stressed to about 95 per cent of the maximum stress. Torrent's expression involved the term V^n, but, in his tests, n appeared to be independent of the type of aggregate or the water/cement ratio.

The discussion in this section shows that, within the range of usual specimen sizes, the influence of size on the average strength is not large for most practical purposes. However, because of

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the higher scatter of results obtained with smaller specimens, they have to be used in a greater number to give the same precision of the mean: five to six 100 mm (4 in.) concrete cubes would be required instead of three 150 mm (6 in.) cubes;[12.42] or five 13 mm ($\frac{1}{2}$ in.) mortar cubes instead of two 100 mm (4 in.) cubes.[12.43]

If the usual sets of three compressive strength cylinders are used, then, by changing from 150 by 300 mm (6 by 12 in.) cylinders to 75 by 150 mm (3 by 6 in.) cylinders, the coefficient of variation of the 28-day strength would typically rise from 3.7 to 8.5 per cent.[12.88] Such an increase in the variability is a serious drawback to the use of smaller test specimens.

Specimen size and aggregate size

If the maximum size of aggregate is large in relation to the size of the mould, the compaction of concrete and the uniformity of distribution of the large particles of aggregate are affected. This is known as the *wall effect* because the wall in-

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fluences the packing of the concrete: the quantity of mortar required to fill the space between the particles of the coarse aggregate and the wall is greater than that necessary in the interior of the mass, and therefore greater than the quantity of mortar available in a well-proportioned mix (Fig. 12.21). In tests on concrete made with 19.05 mm ($\frac{3}{4}$ in.) aggregate, 101.6 mm (4 in.) cubes have been found to require for full compaction an increase in sand content equal to 10 per cent of the total mass of aggregate, compared with a mix used in an infinitely large section.[12.44] To make up this deficiency of fine material during the actual making of specimens, mortar would need to be added from the remainder of the mix.

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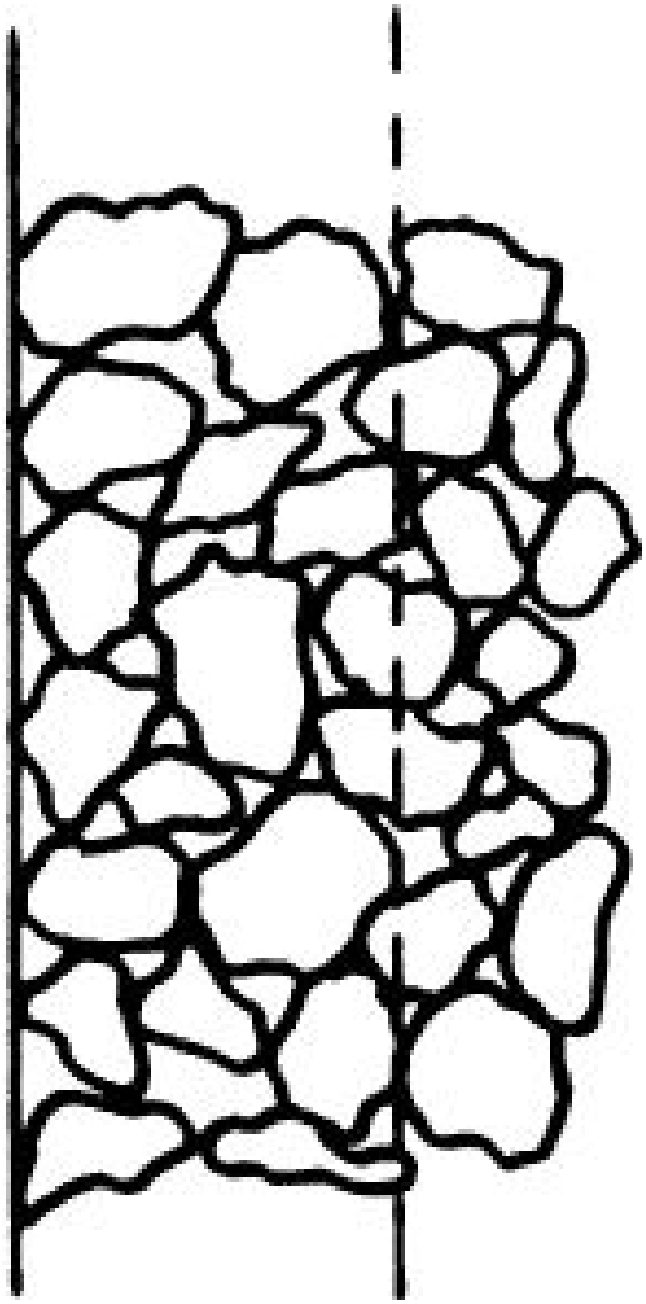


Fig. 12.21. 'Wall effect'

The wall effect is more pronounced the larger the surface/volume ratio of the specimen and is, therefore, smaller in flexure test specimens than in cubes or cylinders.

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To minimize the wall effect, various standards specify the minimum size of the test specimen in relation to the maximum size of aggregate. British Standards BS 1881-108 : 1983 and BS 1881-110 : 1983 (both withdrawn), respectively, allow the use of 100 mm cubes and 100 by 200 mm cylinders with aggregates whose maximum size is up to 20 mm; 150 mm cubes and 150 by 300 mm cylinders can be used with aggregate up to 40 mm in size. The requirement of ASTM C 192-07 is that the diameter of the test cylinder or the minimum dimension of a prism be at least 3 times the nominal maximum size of aggregate.

When the aggregate size exceeds the permissible value for the mould used, screening out of the large-size aggregate is sometimes resorted to. This operation is called *wet screening*. The screening must be done quickly in order to avoid drying out, and the screened material should be remixed by hand. Although the water/cement ratio of the screened concrete can be expected to remain unaltered, both the cement content and wa-

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ter content increase, and generally an increase in strength has been observed. For instance, screening out of particles greater than 19.05 mm ($\frac{3}{4}$ in.) from a mix with an original maximum size of 38.1 mm ($1\frac{1}{2}$ in.) has been found to increase the compressive strength by 7 per cent, and the flexural strength by 15 per cent.[12.45] On another project, screening of the 38.1 to 152.4 mm ($1\frac{1}{2}$ to 6 in.) fraction has resulted in an increase in compressive strength of 17 to 29 per cent.[12.7] With air-entrained concrete, wet screening produces some loss of air, and this causes an increase in strength.

These data reflect not only the effect of the change in the composition of the mix but also the influence of the maximum size of aggregate *per se* (see p. 174).

The limited data on the effect of wet screening on the strength of concrete in direct tension[12.87] do not allow a generalized conclusion.

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Test cores

The fundamental purpose of measuring the strength of concrete test specimens is to estimate the strength of concrete in the actual structure. The emphasis is on the word 'estimate', and indeed it is not possible to obtain more than an indication of the strength of concrete in a structure because this is dependent, *inter alia*, on the adequacy of compaction and on curing. As shown earlier in this chapter, the strength of a test specimen depends on its shape, proportions, and size, so that a test result does not give the value of the *intrinsic* strength of the concrete. Nevertheless if, of two sets of similar specimens made from two concretes, one set is stronger (at a statistically significant level), it is reasonable to conclude that the concrete represented by this specimen is stronger, too, There exist some methods of determining the strength of concrete in situ, but the limitations on the interpretation of test results must be remembered.

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If the strength of standard compression test specimens is found to be below the specified value, then either the concrete in the actual structure has too low a strength as well, or else the specimens are not truly representative of the concrete in the structure. This latter suggestion is often put forward in disputes on the acceptance, or otherwise, of a suspect part of the structure: the test specimens may have been disturbed while setting, they may have been exposed to frost before they hardened sufficiently or have otherwise been improperly cured, or simply the results of the compression test are doubted.

The argument is often resolved by testing a core of concrete taken from the suspect member. If it is intended to determine the *potential* strength of the concrete mix used, corrections for the actual conditions have to be applied. Cores can also be cut in order to determine the *actual* strength of concrete in the structure. The distinction between the two purposes must be clearly borne in mind when the test results are being

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evaluated. The selection of the location of cores also depends on the purpose of testing. This may be: to estimate the strength of a critical part of a structure, or of a part suspected of having been damaged, for example, by frost; or alternatively, to estimate a representative value for the entire structure, in which case a random selection of locations is appropriate.

Cores can also be used to detect segregation of honeycombing or to check the bond at construction joints or to verify the thickness of pavement.

Cores are cut by means of a rotary cutting tool with diamond bits. In this manner, a cylindrical specimen is obtained, sometimes containing embedded fragments of reinforcement, and usually with end surfaces far from plane and square. The core should be soaked in water, capped, and tested in compression in a moist condition according to BS EN 12504-1 : 2000 or ASTM C 42-04, but ACI 318-02[12.124] specifies a moisture condition corresponding to the service environment. Japanese tests[12.116] indicate that testing in

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a dry state yields strength values typically about 10 per cent higher than when the cores are tested wet.

The influence of the height/diameter ratio of the cylinder on the recorded strength was considered on p. 593. If the strength of cores is to be related to the strength of standard cylinders (height/diameter ratio of 2) then, in the core, this ratio should be near 2. When cubes are the standard test specimen, there is some advantage in using cores with a height/diameter ratio of 1 because cylinders with this ratio have very nearly the same strength as cubes. For values of the ratio between 1 and 2, a correction factor has to be applied. Meininger *et al.*[12.83] found the factor to be the same for wet- and dry-tested cores, but lower than specified by ASTM C 42-04 (see Table 12.1).

Cores with height/diameter ratios lower than 1 give unreliable results, and BS EN 12504-1 : 2009 prescribes a minimum value of 0.95 prior to capping but, according to BS 1881-120 : 1983,

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the cap thickness should not exceed 10 mm at any point. This limitation should be observed although in practice, the length of the core may be governed by the thickness of the concrete. Glueing cores which are too short is possible.[12.96]

Use of small cores

Both British and ASTM Standards specify a minimum core diameter of 94 mm (3.7 in.) with the proviso that the core diameter be at least 3 times the maximum size of aggregate; however, ASTM C 42-04 allows, as an absolute minimum, the ratio of the two sizes to be 2.

Nevertheless, there exist circumstances where only very small cores can be drilled, either because of the risk of structural damage or because of congestion of the reinforcement or for aesthetic reasons. In such cases, some standards allow the use of 50 mm (2 in.) diameter cores. These small cores may violate the requirement of a minimum ratio of core diameter to aggregate size, and the drilling operation can affect the bond

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between the aggregate and the surrounding hardened cement paste.[12.98] Tests[12.127] have shown that, when the maximum size of aggregate is 20 mm ($\frac{3}{4}$ in.), 50 mm (2 in.) cores have a strength about 10 per cent lower than 100 mm (4 in.) cores; other tests[12.110] on concretes with 28-day cube strengths between 20 and 60 MPa (or 3000 and 9000 psi) indicate that the difference is between 3 and 6 per cent. A good correlation between the strength of 28 mm ($1\frac{1}{8}$ in.) diameter cores and the cube strength was obtained in laboratory tests on concrete with a maximum size of aggregate of 30 and 25 mm ($1\frac{1}{8}$ and 1 in.)[12.78] (see Fig. 12.22).

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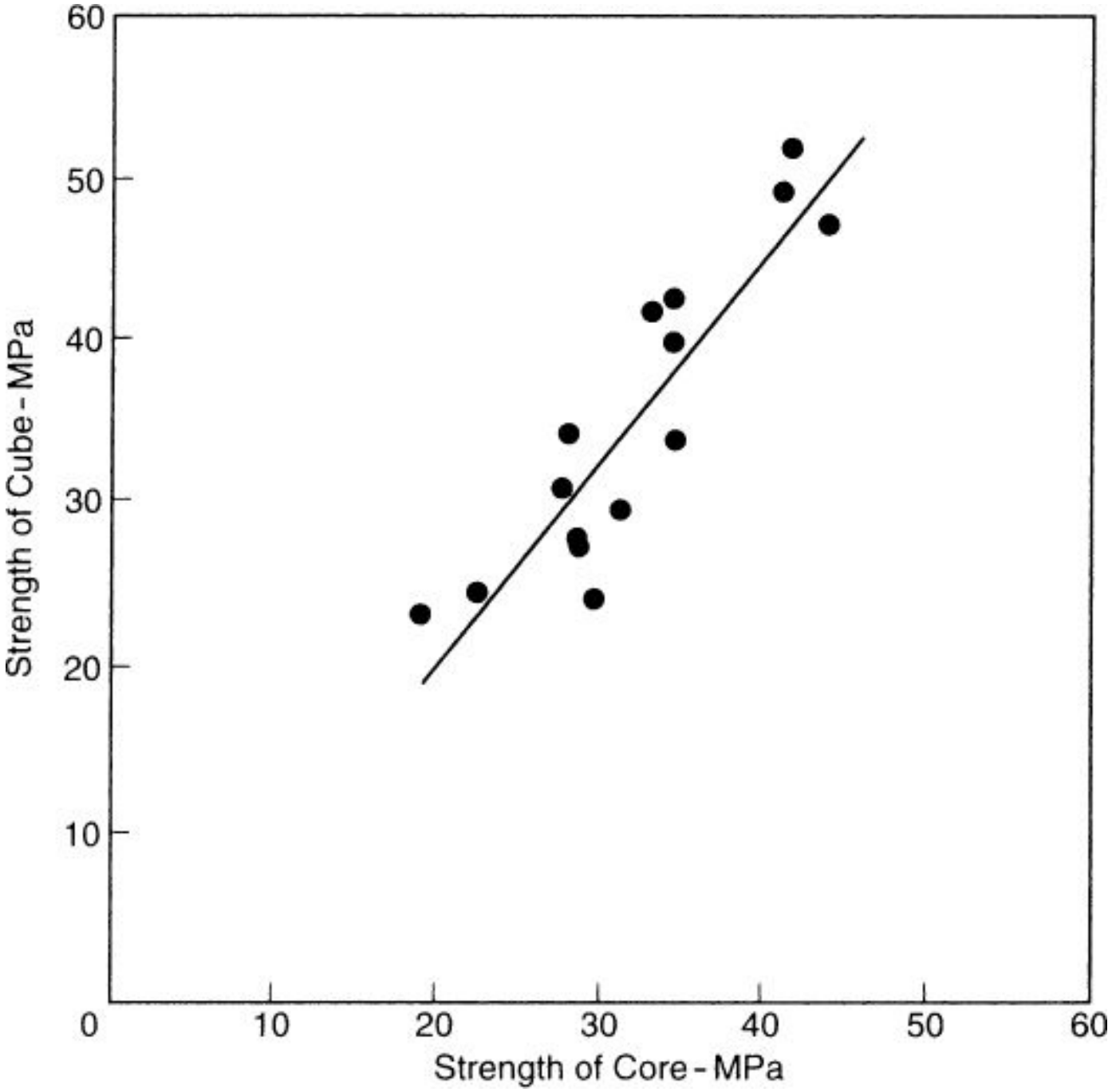


Fig. 12.22. Relation between the strength of 28 by 28 mm ($1\frac{1}{8}$ by $1\frac{1}{8}$ in.) cores and the strength of 150 mm (6 in.) cubes; maximum aggregate size 25 and 30 mm[12.78]

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Overall, in view of the numerous factors influencing the strength of cores, as compared with the relative uniformity of cast standard compression test specimens, the effect of core size can be considered to be unimportant. However, small cores have a higher variability than standard-size cores; typical values[12.100] of the coefficient of variation are 7 to 10 per cent for 50 mm cores, and 3 to 6 per cent for 150 mm cores. It follows that, for a given precision of the estimate of strength, the required number of 50 mm cores is probably 3 times larger than the number judged adequate for 100 mm (4 in.) or 150 mm (6 in.) cores. Likewise, when the core diameter is less than three times the maximum size of aggregate, an increased number of cores has to be tested.

Factors influencing strength of cores

The strength of cores is generally lower than that of standard cylinders, partly as a consequence of the drilling operation and partly because site curing is almost invariably inferior to curing pre-

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scribed for standard test specimens. However careful the drilling, there is a high risk of slight damage. The effect appears to be greater in stronger concrete, and Malhotra[12.99] suggests that the reduction in strength can be as high as 15 per cent for 40 MPa (6000 psi) concrete. A reduction of 5 to 7 per cent is considered reasonable by the Concrete Society.[12.100]

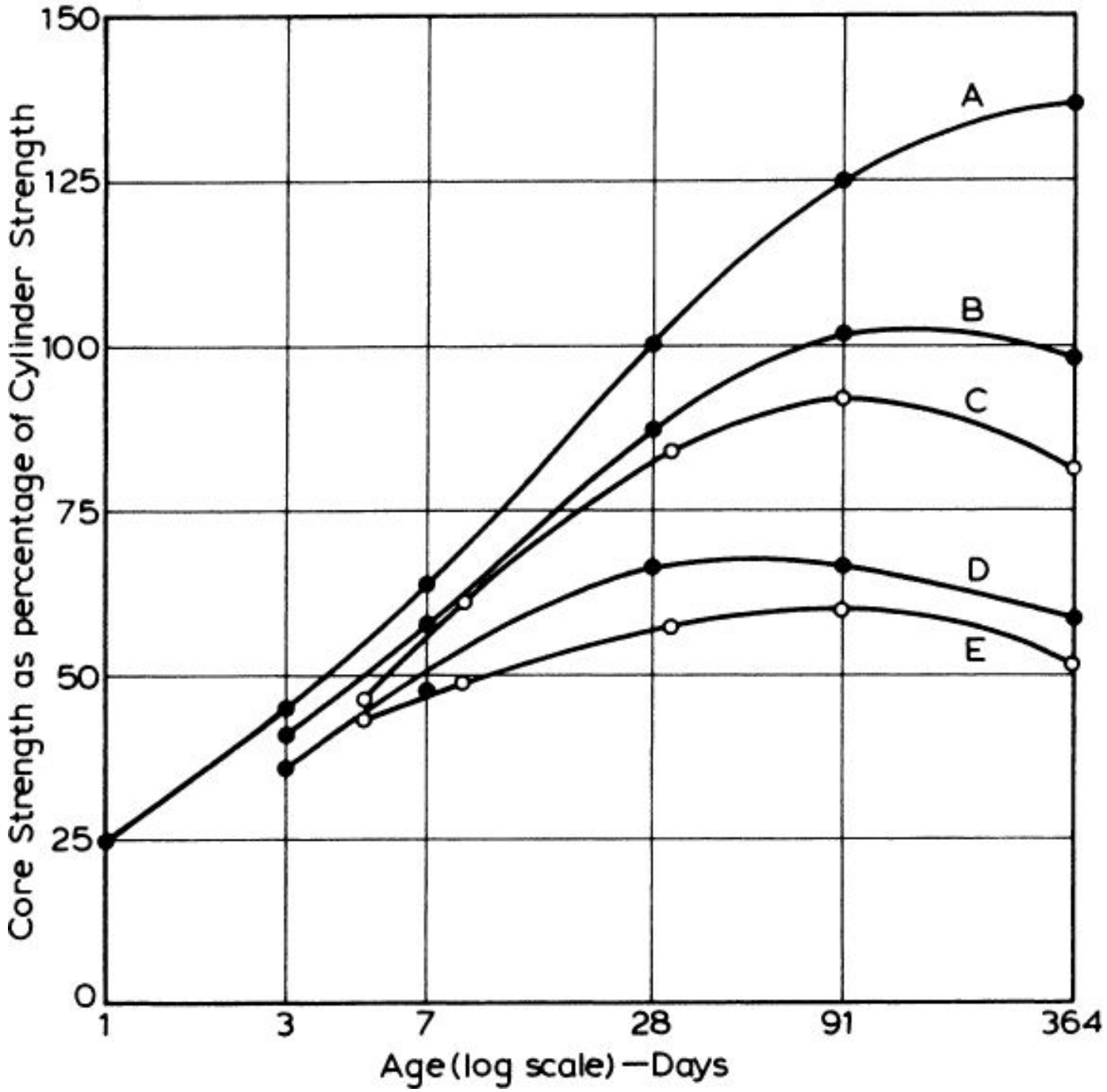
There is, however, a difficulty in separating out the effect of drilling because the curing history of cores is perforce different from the curing history of cast test specimens. The difficulty is exacerbated by the fact that the exact curing history of a structure is usually difficult to determine so that the effect of curing on the strength of cores is uncertain. For structures cured in accordance with the recommended practice, Petersons[12.67] found that the ratio of core strength to standard cylinder strength (at the same age) is always less than 1, and decreases with an increase in the concrete strength level. Approximate values of this ratio are: just under 1 when the cylin-

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der strength is 20 MPa (3000 psi) and 0.7 when it is 60 MPa (9000 psi).

Because cores are often taken after the 28-day test cylinders have been tested, cylinders of an age comparable to the age of the cores may not be available, but it is sometimes argued that cores taken from concrete many months old should have a higher strength than at 28 days. This appears not to be the case in practice (see Figs 12.23 and 12.24), and there is evidence that in situ concrete often gains little in strength after 28 days.[12.102,12.103] Tests on high strength concrete[12.112] show that, although the strength of cores increases with age, the core strength, even up to the age of 1 year, remains lower than the strength of standard 28-day cylinders; this is shown in Table 12.3.

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Fig. 12.23. Development with time of strength of concrete cores made with Type I cement expressed as a percentage of 28-day strength of standard cylinder (38 MPa (5500 psi)): (*A*) standard cylinder; (*B*) well-cured slab, core tested dry; (*C*) well-cured slab, core tested wet; (*D*) poorly cured slab, core tested dry; (*E*) poorly cured slab, core tested wet[12.101]

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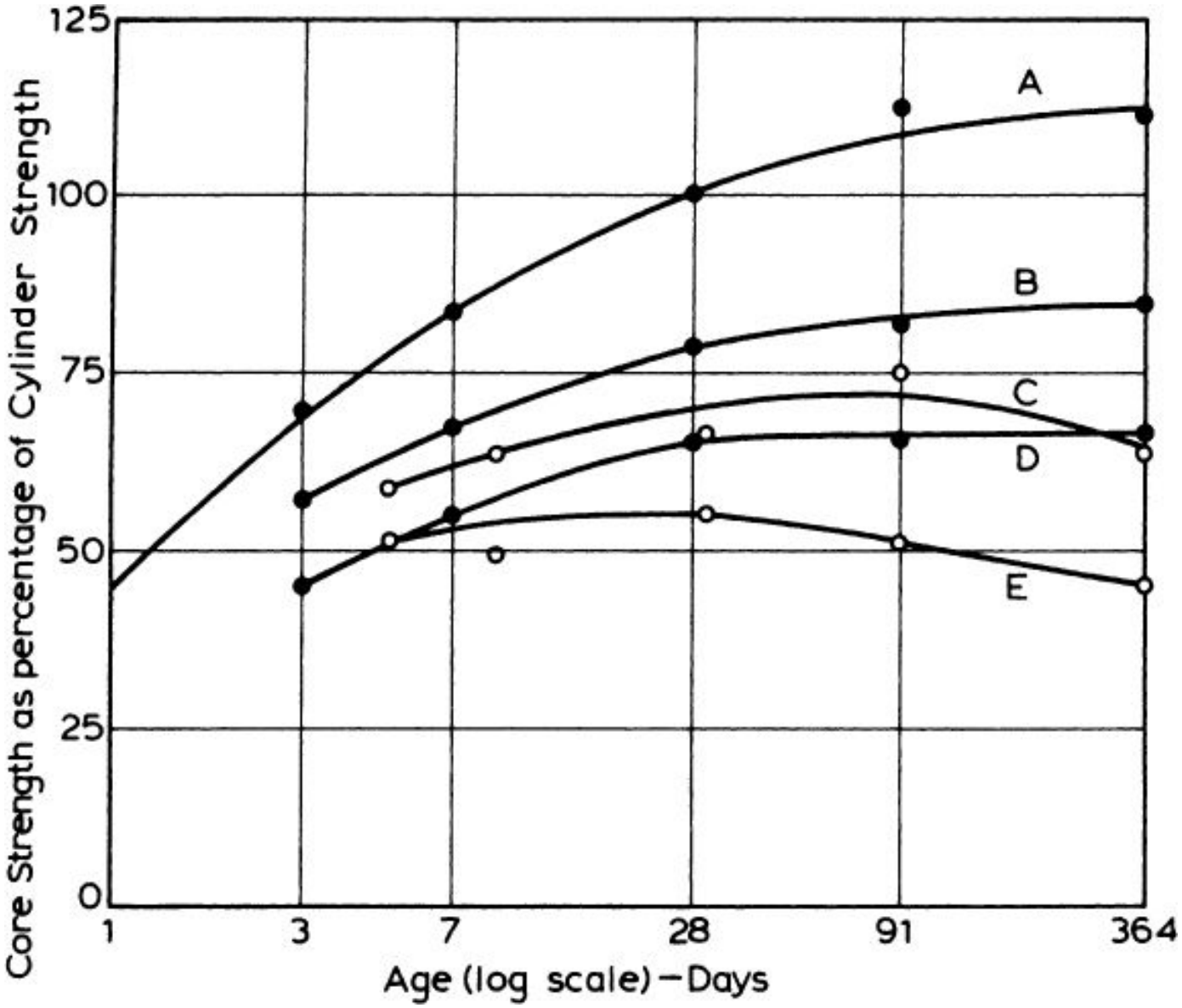


Fig. 12.24. Development with time of strength of concrete cores made with Type III cement expressed as a percentage of 28-day strength of standard cylinder (38 MPa (5500 psi)): (*A*) standard cylinder; (*B*) well-cured slab, core tested dry; (*C*) well-cured slab, core tested wet; (*D*) poorly cured slab, core tested dry; (*E*) poorly cured slab, core tested wet[12.101]

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Table 12.3. Development of the Strength of Cores* with Age (based on ref. 12.112)

Age, days	Strength, MPa		Core strength as a proportion of strength of 28-day standard cylinders
	Standard cylinders	Cores	
7	66.0	57.9	0.72
28	80.4	58.5	0.73
56	86.0	61.2	0.76
180	97.9	70.6	0.88
365	101.3	75.4	0.94

*Cores taken from columns cured using a sealing compound.

These results accord with Petersons' view[12.104] that, for average conditions, the increase in strength over that at 28 days is 10 per cent at three months, and 15 per cent at the age of six months. The effect of age is, therefore, not easy to deal with but, in the absence of definite moist curing, no increase in strength should be expected with age and no age correction should be used in the interpretation of the strength of cores.[12.100]

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The location in the structure from which the core has been taken may affect the strength of the core. If the core has been taken from concrete in tension, the core strength may be low because of the presence of cracks:[12.114] thus, a false picture of the strength of the concrete in the structure can be obtained.

The position of the core with respect to the height of the lift may also be of relevance. Cores usually have the lowest strength near the top surface of the structure, be it a column, a wall, a beam, or even a slab. With an increase in depth below the top surface, the strength of cores increases,[12.67] but at depths greater than about 300 mm there is no further increase. The difference can be as high as 10 or even 20 per cent. In the case of slabs, poor curing increases this difference. Compressive and tensile strengths are affected to the same degree.[12.105] This pattern of strength is, however, not universal, some tests indicating no significant variation in core strength with height.[12.112] It is likely that the variation

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in strength with height is the consequence of trapped bleed water, coupled with a variation in compaction: when these factors are absent, there is no variation in strength with height.

The presence of trapped bleed water may also be responsible, in part, for the reported influence of the orientation of the core (vertical or horizontal) on its strength. Cores drilled horizontally were found to have a strength lower by, typically, 8 per cent.[12.106] This effect is similar to the effect of bleed water on the strength of cubes (see p. 590).

The conversion expressions of BS EN 12504-1 : 2009, distinguish between cores drilled horizontally and those drilled vertically, the ratio of the strength of the former to the latter being 0.92. However, if there is no trapped bleed water in the concrete, the correction for horizontally-drilled cores may not be valid. It is also possible that difficulties in horizontal drilling contribute to the lower strength of such cores.

British Standard BS EN 12504-1 : 2009 gives also correction factors which allow for the weak-

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ening effect of transverse reinforcement in the core. Although some effect of embedded steel on strength could be expected, the information on this is contradictory. Reviews by Malhotra[12.99] and by Loo *et al.*[12.132] report some tests showing no reduction in strength, and other tests where the reduction ranged between 8 and 18 per cent; the reduction seems to be higher when the height/diameter ratio of the core is 2 than at lower values of this ratio.[12.132] The Concrete Society[12.100] also reports a reduction in strength as a function of the position of the steel: the effect is greater the further the steel is from the end of the core.

The tests of Loo *et al.*[12.132] confirmed that embedded transverse reinforcement reduces the strength of cores with a height/diameter ratio of 2, but the effect decreases at low values of the height/diameter ratio; at a height/diameter ratio of 1, embedded steel has no effect on the measured strength, regardless of the position of the steel in the core. This effect is linked to the stress distribution in cylinders with various values of

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the height/diameter ratio (see p. 595). When this ratio is 1, or in a cube, there is no lateral tensile stress in the specimen, and the steel is well able to resist vertical compression.

In view of the various factors involved and of the conflicting data, no reliable factor which allows for the presence of transverse steel can be accepted. The best solution, if possible, is to take cores from a location such that they contain no reinforcement, not only because it complicates the strength assessment, but more importantly, because cutting reinforcement may have highly undesirable structural consequences. In any case, the presence of steel parallel to the axis of the core is unacceptable.

Relation of core strength to strength in situ

It should be emphasized that the core strengths, when converted to the strength of cylinders of standard size or to cube strengths, represent, at best, the strength of in situ concrete. They are not to be *equated* with the strength of *standard*

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test specimens, which is the potential strength of the given concrete (see p. 584). Indeed, from the preceding review of the various factors influencing the strength of cores, it is apparent that it is not easy to *interpret* the strength of cores in relation to a specified 28-day strength. Various reports[12.99,12.103] suggest that, even under excellent conditions of placing and curing, the strength of cores is unlikely to exceed 70 to 85 per cent of the strength of standard test specimens. This view is supported by ACI 318-08[12.124] which considers that concrete in the part represented by a core test is adequate if the average strength of 3 cores is equal to at least 85 per cent of the specified strength and if no single core has a strength lower than 75 per cent of the specified value; no age allowance is made. It should be noted that, according to ACI 318-95, cores are tested in a dry state if the structure is dry in service, which should lead to a higher strength than when tested to ASTM or British Standards (see p. 602). Thus, the requirements given above are fairly liberal.

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It is useful to note that the '85 per cent allowance' is applied also to shotcrete according to ACI 506.2-90.[12.133] However, since shotcrete is accepted on the basis of core strength, and not of moulded specimens, there is no logical reason for this 'allowance'.[12.111]

In some cases, beam specimens can be sawn from road or airfield pavements, using a diamond or silicon carbide saw. Such specimens are tested in flexure in accordance with ASTM C 42-04 but, at least when siliceous aggregate is used, sawn specimens give appreciably lower strengths than comparable moulded beams.[12.23] Cutting of beams is not much used and the means of obviating their use was discussed on p. 600.

Cast-in-place cylinder test

It has been stressed repeatedly that standard compression test specimens give a measure of the potential strength of concrete, and not of the strength of the concrete in the structure. Knowledge of the latter cannot be directly obtained

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from tests on separately made specimens. And yet, it is sometimes necessary to assess the strength of concrete in the actual structure, for instance, for the purpose of deciding on the time of removal of formwork, the application of prestress, or subjecting the structure to loading. It may also be desired to assess the effectiveness of curing or of protection from freezing.

One means of obtaining the requisite information is by use of cast-in-place cylinder specimens which are made in push-out moulds. These special moulds are fastened in tubular supports within the formwork of the structure prior to placing the concrete, as shown in Fig. 12.25. This test method is limited to use in slabs with a depth of 125 to 300 mm (5 to 12 in.) and is prescribed by ASTM C 873-04. The mould is filled during placing of concrete in the slab formwork. Thus, the curing and temperature conditions of the specimen and the slab are similar.[12.122] Nevertheless, the compaction of the concrete in the mould is not identical with the compaction of the concrete

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in the actual structure. Consequently, the strength of cast-in-place cylinders is reported by ASTM C 873-04 to be about 10 per cent higher than the strength of cores drilled in the vicinity.

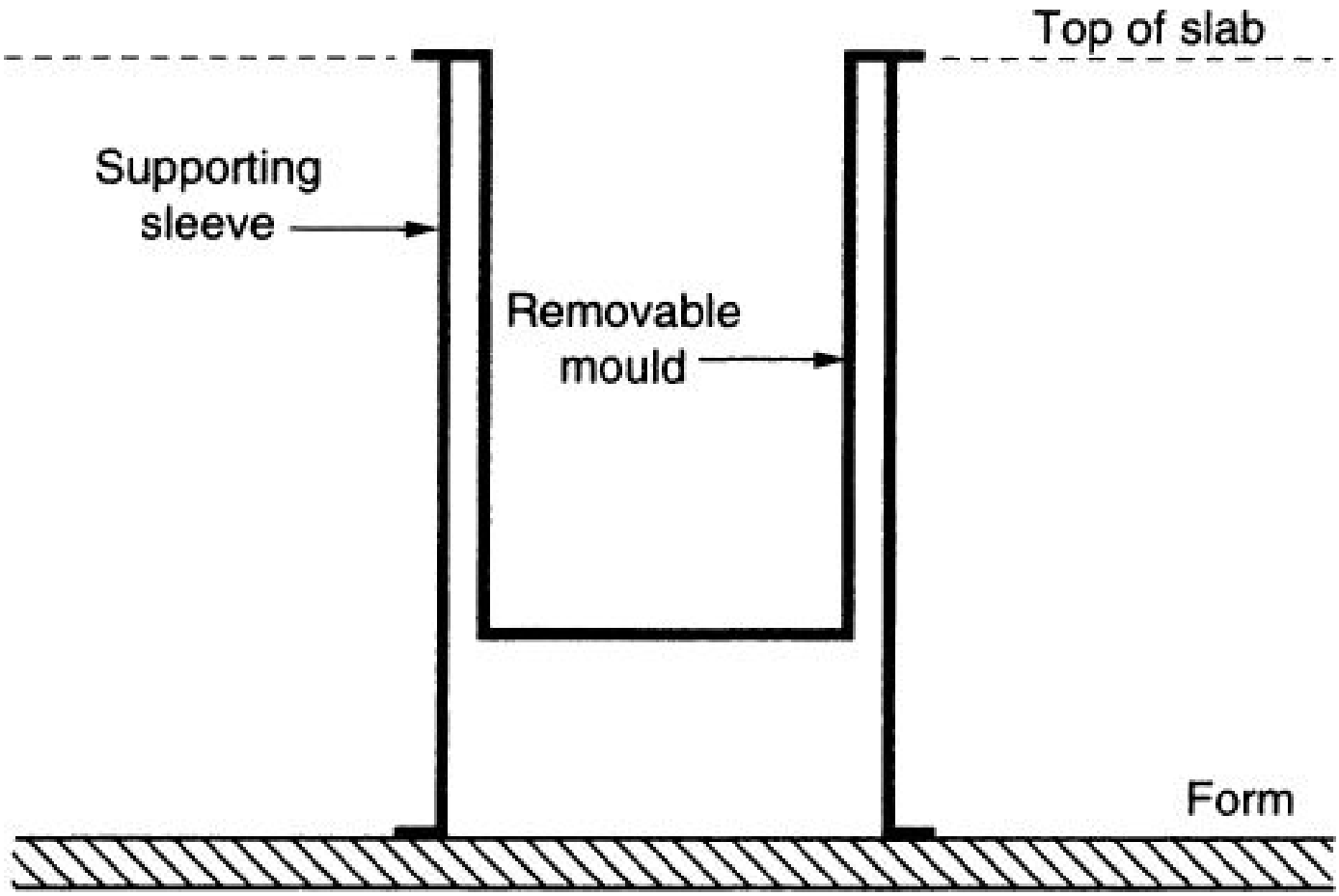


Fig. 12.25. Diagrammatic representation of a mould for a cast-in-place cylinder

The topic of the strength of concrete in structures is briefly considered on p. 625.

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Influence of rate of application of load on strength

In the range of speeds at which a load can be applied to concrete, the rate of application of load has a considerable effect on the apparent strength of concrete: the lower the rate at which stress increases the lower the recorded strength. This is probably caused by the increase in strain with time due to creep and, when limiting strain is reached, failure takes place. Loading in compression over a period of 30 to 240 minutes has been found to cause failure at 84 to 88 per cent of the ultimate strength obtained when the load is applied at the rate of approximately 0.2 MPa/s (30 psi/s).[12.27] Concrete can withstand indefinitely only stresses up to about 70 per cent of the strength determined under a load applied at the rate of 0.2 MPa/s (30 psi/s).[12.28]

Figure 12.26 shows that increasing the rate of application of compressive stress from 0.7 kPa/s to 70 GPa/s (0.1 to 10^7 psi/s) doubles the appar-

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ent strength of concrete. Raphael's study[12.52] of tests on concrete used in dams suggests that increasing the rate of application of compressive stress by 3 orders of magnitude (which may be the case in an earthquake) increases the strength by about 30 per cent. However, within the practical range of rates of loading of compression specimens, that is between 0.07 and 0.7 MPa/s (10 and 100 psi/s), the measured strength varies only between 97 and 103 per cent of the strength at 0.2 MPa/s (30 psi/s).

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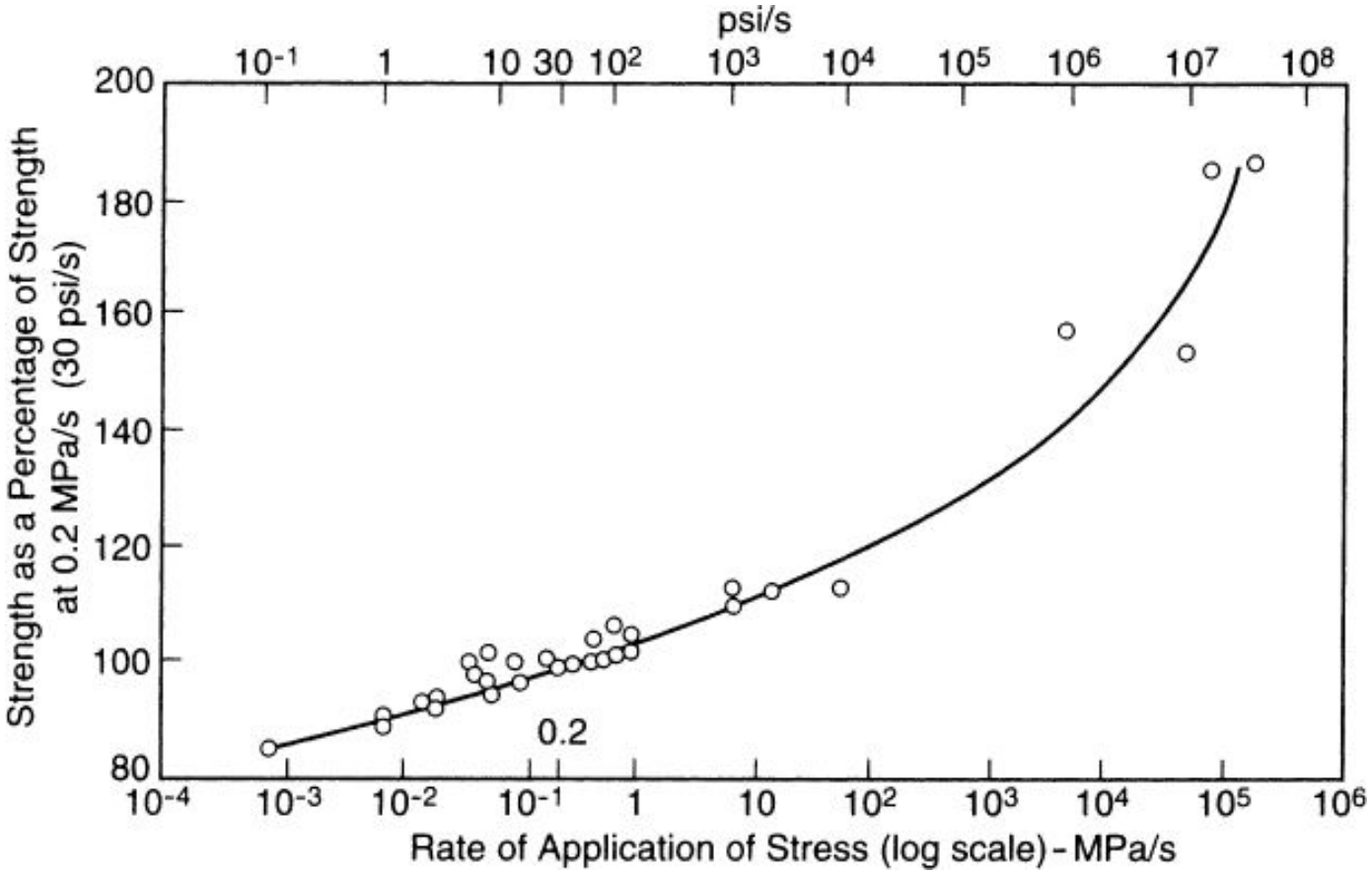


Fig. 12.26. Influence of the rate of application of load on the compressive strength of concrete[12.27]

Nevertheless, for test results to be comparable, the stress has to be applied at a standardized rate. The rate of loading of compression test specimens is prescribed by ASTM C 39-09a as 0.20 to 0.30 MPa/s (28 to 42 psi/s), although a higher rate may be applied during the first half of loading. British Standard BS EN 12390-3 : 2009 prescribes a rate of 0.2 to 1.0 MPa/s (30 to 145 psi/

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s) which has to be maintained throughout the application of the load?

The results of flexure tests are affected by the speed of loading in a way similar to compression tests. Increasing the rate of increase in stress in the extreme fibre of the test beam from 2 to 130 kPa/s (0.3 to 19 psi/s) was found to increase the modulus of rupture by about 15 per cent.[12.20] The modulus of rupture increases linearly with the logarithm of the rate of application of stress but, at very high rates of application of tensile stress, there seems to be a departure from linearity: the rate of increase in strength increases at an even greater rate. This is similar to the behaviour under compressive stress (Fig. 12.26). At 170 MPa/s (24 700 psi/s) the modulus of rupture was found to be 40 to 60 per cent larger than at 27 kPa/s (3.9 psi/s).[12.27] British Standard BS EN 12390-5 : 2009 prescribes a rate of increase in the extreme fibre in flexure of between 0.04 and 0.06 MPa/s (5.8 to 8.8 psi/s) and ASTM C 78-09 spe-

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cifies a rate between 0.86 and 1.21 MPa/min (2 to 3 psi/s).

It may be relevant to mention that the tensile strain capacity, which is of interest in the control of cracking in mass concrete, depends on the rate of increase in tensile stress. Liu and McDonald[12.89] found that at slow rates of loading (0.17 MPa (25 psi) per week) the strain capacity is 1.1 to 2.1 greater than when the rate of loading is 5 kPa/s (0.68 psi/s). The magnitude of this increase, which is probably due to creep, depends on the flexural strength and on the modulus of elasticity of the concrete: the increase is greater for higher strengths and for lower values of the modulus of elasticity.[12.89]

An increase in compressive strain capacity at lower rates of increase in strain was reported by Dilger *et al.*[12.68]

The influence of the strain rate on the recorded strength is largest for direct tension, intermediate for flexure, and least for compression[12.54] (Fig.

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12.27). Generally, stronger concrete exhibits lower sensitivity to the strain rate.

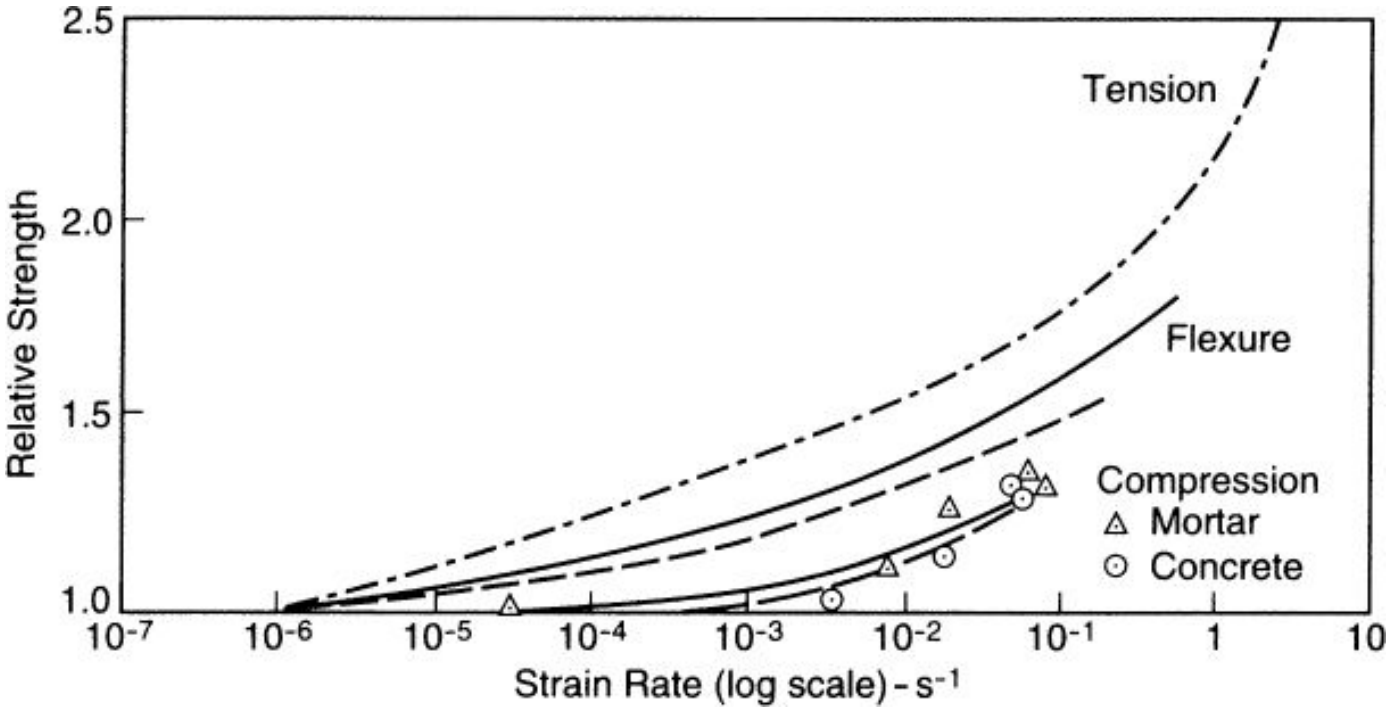


Fig. 12.27. Influence of strain rate on relative strength (expressed as a proportion of strength at the standard rate of strain) in tension, flexure, and compression (based on ref. 12.54 with the permission of the publisher (ASCE))

Accelerated-curing test

Concrete is usually placed in a structure in stages or lifts, one on top of another. Thus, by the time the results of the 28-day test, or even of the 7-day

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test, are available, a considerable amount of concrete may overlay that represented by the test specimens in question. It is then rather late for remedial measures if the concrete is too weak; if it is too strong, this indicates that the mix used was uneconomical. Indeed, production control with a 28-day delay is not sensible.

It is clear that it would be a tremendous advantage to be able to predict the 28-day strength within a few hours of placing of concrete. The strength of concrete at 24 hours is an unreliable guide in this respect, not only because different blended cements gain strength at varying rates, but also because even small variations in temperature during the first few hours after casting have a considerable effect on the early strength. It is, therefore, necessary for the concrete to have achieved a greater proportion of its potential strength before testing, and a successful test based on accelerated curing was developed by King[12.46] in the mid-1950s. Since that time, sev-

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eral accelerated-curing test methods have become standardized.

All these methods rely on accelerating the development of strength of standard compression test specimens by a rise in temperature of the concrete specimen, without permitting a loss of water from it. Details of the various tests are given in the respective standards but a common feature of the tests is that, as in the conventional strength tests, most of the test operations take place during usual working hours; this is beneficial on construction projects where the site laboratory does not function round the clock.

Four test methods using accelerated curing are prescribed by ASTM C 684-99 (03): their brief description is given in Table 12.4. In Method A, the temperature rise is due to the heat of hydration of cement, the primary function of the water bath being to conserve that heat. In Method B, there is an additional input of heat from the boiling-water bath. In Method C, curing takes place under adiabatic conditions, the sealed spe-

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cimen (so as to prevent moisture loss) being placed in an insulating container. In Method D, a container pressurized to 10.3 MPa (1500 psi) at a temperature of 149 °C (300 °F) is used. Thus, in Method D, specialized equipment is necessary;[12.130] also, the size of the test cylinder is limited so that, if the maximum size of aggregate is larger than 25 mm (1 in.), wet sieving has to be resorted to.

Table 12.4. Summary of the Procedures for Accelerated Curing Prescribed by ASTM C 684-99 (03)

Test method	*Curing medium*	*Curing temperature, °C (°F)*	*Age when accelerated curing begins*	*Duration of accelerated curing, hours*	*Age at test, hours*
A: warm water	'Insulating' water	35 (95)	Immediately after casting	$23\frac{1}{2}$	24
B: boiling water	Heating by water	100 (212)	23 hours	$3\frac{1}{2}$	$28\frac{1}{2}$
C: autogenous	Heat of hydration	Variable	Immediately after casting	48	49
D: high temperature and pressure	External heat and pressure	149 (300)	Immediately after casting	5	$5\frac{1}{4}$

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A word of caution about the use of boiling water in Methods B and D should be added: there is a danger of scalding and also of eye burns from a sudden escape of steam.

There are three British methods, given in BS 1881-112 : 1983, all of which use a water bath. One method is similar to Method A of ASTM C 684-99 (03), that is, it uses a water bath at 35 °C. The second and third methods use water baths at 55 and 82 °C, respectively. In all cases, strength is determined at the age of up to 24 hours. The British and American test methods differ with respect to the temperature of the specimen when its strength is determined.

It is interesting to examine the effects of the specific curing procedures on the products of hydration of cement. It is known that temperature influences the physical characteristics of these products (see p. 361) but there is also a chemical effect in the case of the boiling-water method: the crystallinity of ettringite is degraded.[12.118] This

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does not, however, affect the usefulness of the boiling-water method.

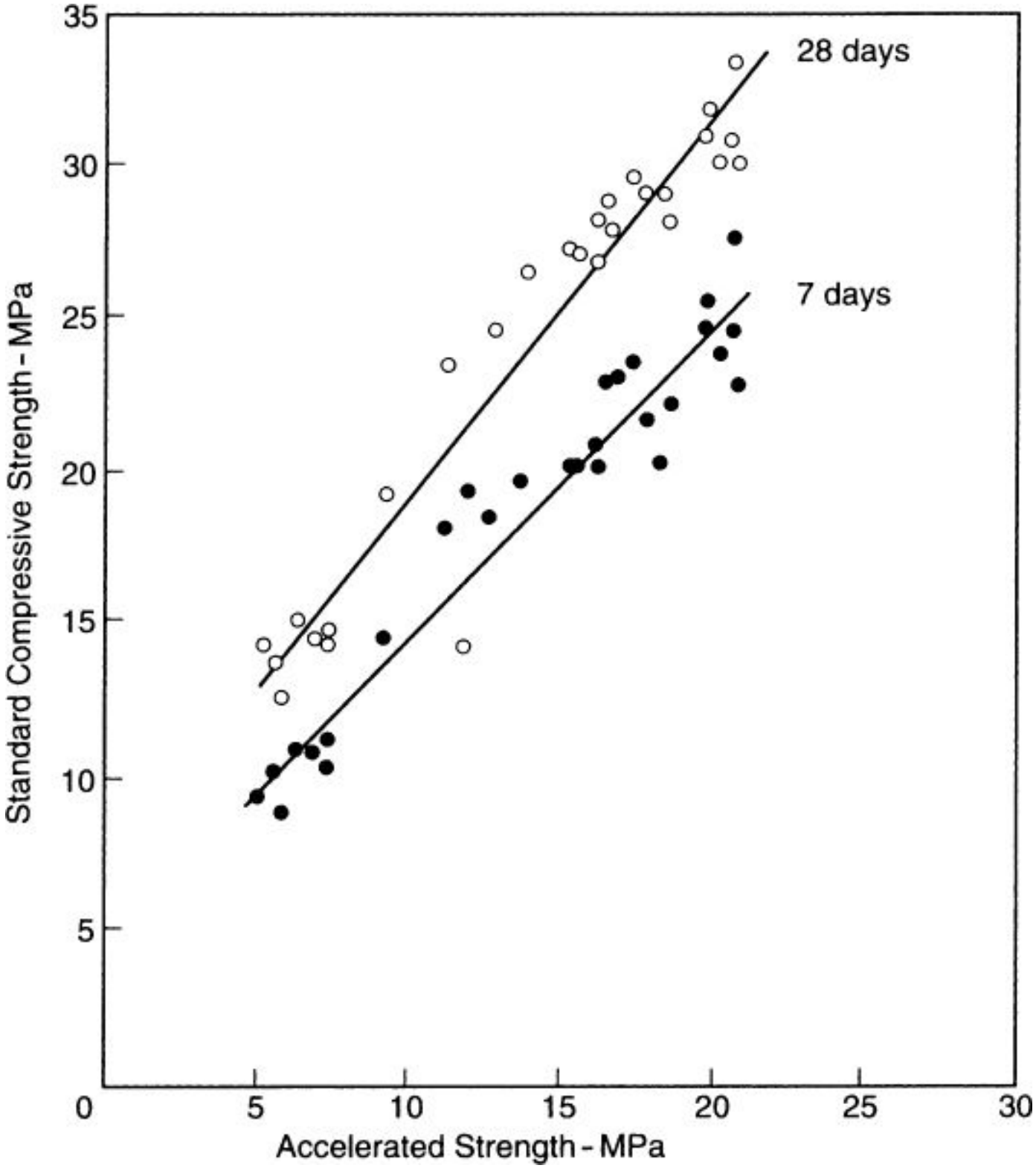
The autogenous-curing method (Method C of ASTM C 684-99 (03) does not result in a uniform acceleration of the development of strength because the nature of the cement used controls the temperature rise, and this influences the rate of further hydration. In addition, the strength is affected by the richness of the mix in a manner different from that under normal curing. Nevertheless, a reliable relation between the accelerated strength and the 28-day normal curing strength has been obtained. This is of the form: 28-day strength = accelerated strength plus a constant.[12.70]

In fact, all the accelerated-curing test methods give a linear relation between the accelerated strength and the strength of standard test specimens at 28 days, but each method gives a different relation. Figure 12.28 shows an example of this relation for Method B of ASTM C 684-99 (03), using a range of mixes containing fly ash

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from different sources but only a single Portland cement.[12.145] Generally speaking, the specific equation relating the 28-day strength of standard specimens to the accelerated-curing strength is different for cements having a different composition. Some tests[12.108] have shown that the maximum size of aggregate (but not its shape or texture) also affects the relation.

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Fig. 12.28. Relation between the accelerated-curing strength according to Method B of ASTM C 684-89 and the strength of standard test cylinders at 7 and 28 days[12.145]

According to BS 1881 : 112 : 1983, curing at 35 °C (95 °F) leads to a greater sensitivity of the accelerated-curing strength to the variation in mix proportions. On the other hand, tests on mortar indicate that curing at 35 °C (95 °F) has a high reproducibility.[12.118]

To establish the relation between the accelerated-curing strengths and the 28-day strength for the purpose of predicting the latter from the former, tests over a range of strength values are necessary; ACI 214.1R-81 (Reapproved 1986)[12.21] specifies the use of at least three water/cement ratios. The correlation coefficient of such an equation is generally very high so that the associated 95 per cent confidence interval is narrow: a value of less than 3 MPa (400 psi) has been reported.[12.120] This is so because the

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accelerated-curing test is no more variable than the standard 28-day test.[12.119]

Accelerated-curing test methods can also be used for the determination of flexural and splitting tension strengths.[12.107]

Direct use of accelerated-curing strength

The preceding reference to the variability of the results of the accelerated-curing tests suggests a particularly worthwhile *direct* use of accelerated-strength testing in quality control of the production of concrete: the early availability of test results makes it possible fairly rapidly to adjust the mix proportions or make other changes in the production process.

Furthermore, the fact that there is no unique relation between the accelerated-curing strength and the standard 28-day strength begs the question as to whether the purpose of determining the former strength should be to 'predict' the latter. Admittedly, this was the original impetus for the development of accelerated-curing test meth-

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ods, but there is nothing sacrosanct about the 28-day strength, especially when the specimens have been cured under ideal conditions, far removed from the usual curing conditions of the concrete in situ. Moreover, the actual strength of the concrete in the structure is influenced by the degree of compaction, bleeding, and segregation. Thus, the 28-day strength of standard test specimens is no more representative of the strength of the concrete in the actual structure than the strength of specimens subjected to accelerated curing.

It is, therefore, strongly arguable that the accelerated-curing strength can, in its own right, be used as an indication of the potential strength of the concrete which was delivered for placement in the structure or, indeed, as a measure of the potential strength. It is worth quoting Smith and Chojnacki[12.69] who expressed the opinion that "a suitable accelerated curing procedure can offer a more convenient and realistic way of ascertaining if the concrete will satisfy the purpose

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for which it was designed". This was written in 1963, and the replacement of routine use of the standard 28-day compression test by the accelerated-curing strength test is long overdue. The latter test is superior as a quality control test, as well as a compliance test, because the outcome of the test is available within a day or two of the placing of concrete.

The difficulty lies in the attachment of the engineers to the traditional test. For a change to take place, the design 'thinking' would have to be entirely in accelerated-strength values. As these are lower than the 28-day standard curing strength values, there is a certain amount of reluctance to accept the new 'numbers'. What should not be done is to accept concrete in the first instance by the accelerated test and to require it *also* to satisfy the 28-day cylinder test. This is too stringent a requirement because, for a given variability of concrete, the probability of passing two tests is smaller than that of passing either. Because the accelerated test and the 28-day test have approx-

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imately the same variability, either of them alone is adequate to establish that the concrete is from the desired population (see p. 639), and this is the aim of acceptance testing.

Non-destructive tests

The tests described so far in this chapter involve specially made specimens which, as such, do not necessarily give direct information about the concrete in the actual structure; yet, this is what matters. Field-cured test specimens, and also cores, are of some help in this respect. However, the former require pre-planning, and the latter cause damage, albeit local, to the structure.

To get around these problems, a wide range of *in-situ tests*, known also as *in-place tests*, have been developed. These tests are traditionally called non-destructive tests, it being understood that some minor damage to the structure may be involved, although its performance or appearance must not be impaired. An important feature of non-destructive tests is that they permit re-testing

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at the same, or nearly the same, location so that changes with time can be monitored.

The use of non-destructive tests leads to increased safety and allows better scheduling of construction, thus making it possible to progress faster and more economically. Broadly speaking, these tests can be categorized into those that assess the strength of the concrete in situ, and those that determine other characteristics of the concrete, such as voids, flaws, cracks, and deterioration.

With respect to strength, it should be noted that it can be only assessed, and not measured, because the non-destructive tests are, for the most part, comparative in nature. Thus, it is useful to establish an experimental relation between the property being measured by a given test and the strength of test specimens or cores from the actual concrete; thereafter, this relation can be used to 'convert' the non-destructive test result into a strength value. An understanding of the physical relation between the given non-destructive test

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result and strength is essential. This relation for the various tests will be discussed in what follows. As this book is concerned with the properties of concrete, and not with testing techniques, the actual details of the different tests have to be sought in the relevant standards or handbooks.

One more general comment about the interpretation of the results of non-destructive tests is necessary. The tests rarely give a 'number' which can be unequivocally interpreted: engineering judgement is necessary. Thus, if the testing arises from a dispute between the parties involved in the construction, the full test programme should be determined in advance and the interpretation of possible test results, bearing in mind their variability, should also be agreed. Otherwise, there is a risk that one party or another will seek additional tests, and the dispute about the concrete in the structure will be compounded by a dispute about testing. Helpful advice about planning non-destructive testing is given in BS 1881-201 : 1986, and BS 6089 : 2010 gives a

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guide to the assessment of concrete strength in existing structures.

Rebound hammer test

This is one of the oldest non-destructive tests and it is still widely used. It was devised in 1948 by Ernst Schmidt, and is therefore known also as *Schmidt hammer*, or *sclerometer*, test. The hardness measured by the rebound hammer is quite different from the hardness determined in tests on metals, which involve indentation.

The rebound hammer test is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass impinges. However, despite its apparent simplicity, the rebound hammer test involves complex problems of impact and the associated stress-wave propagation.[12.134] In the rebound hammer test (Fig. 12.29) a spring-loaded mass has a fixed amount of energy imparted to it by extending the spring to a fixed position; this is achieved by pressing the plunger against the sur-

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face of the concrete under test. Upon release, the mass rebounds from the plunger, still in contact with the concrete surface, and the distance travelled by the mass, expressed as a percentage of the initial extension of the spring, is called the rebound number. This number is indicated by a rider moving along a graduated scale. Some hammer models produce a print-out of test results. The rebound number is an arbitrary measure because it depends on the energy stored in the given spring and on the size of the mass. The hammer has to be used against a smooth surface, preferably a formed one. Open-textured concrete cannot, therefore, be tested. Trowelled surfaces should be rubbed smooth using a carborundum stone. If the concrete under test does not form part of a larger mass, it has to be supported in an unyielding manner, as jerking during the test would result in a lowering of the rebound number recorded.

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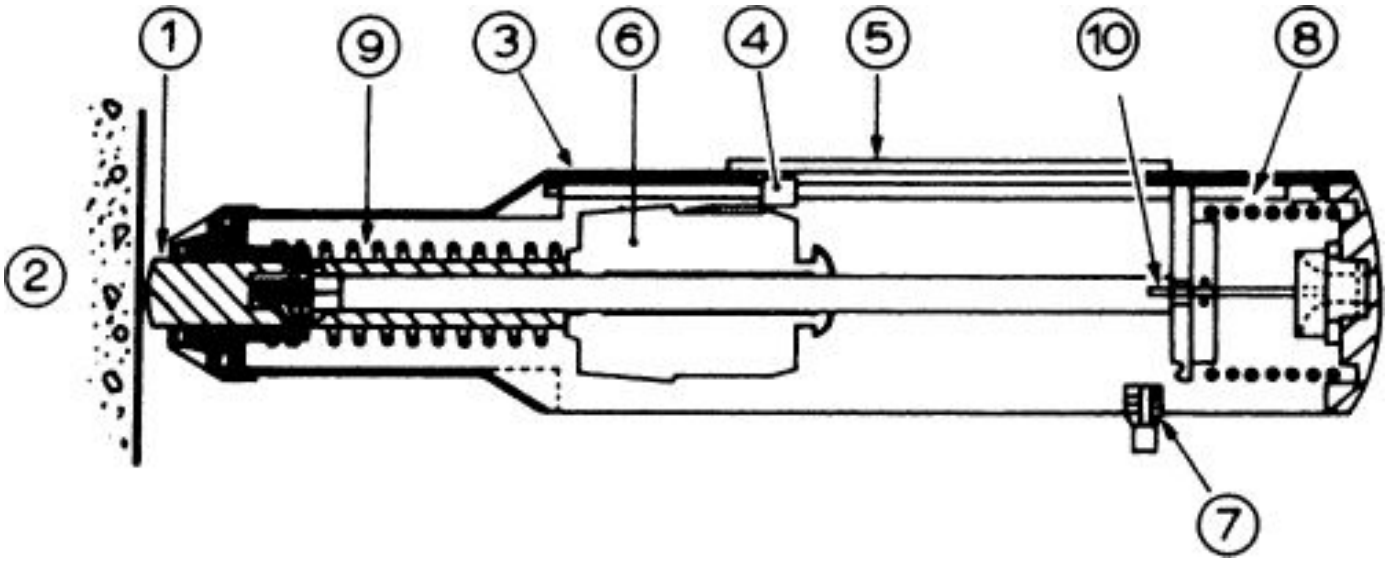


Fig. 12.29. Rebound hammer: (1) plunger; (2) concrete; (3) tubular housing; (4) rider; (5) scale; (6) mass; (7) release button; (8) spring; (9) spring; (10) catch

The test is sensitive to local variations in the concrete; for instance, the presence of a large piece of aggregate immediately underneath the plunger would result in an abnormally high rebound number; conversely, the presence of a void in a similar position would lead to a low result. Moreover, the energy absorbed by the concrete is related both to its strength and its stiffness, so that it is the combination of strength and stiffness that governs the rebound number.[12.122] Because

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the stiffness of concrete is influenced by the type of aggregate used (Fig. 12.30), the rebound number is not uniquely related to the strength of concrete.

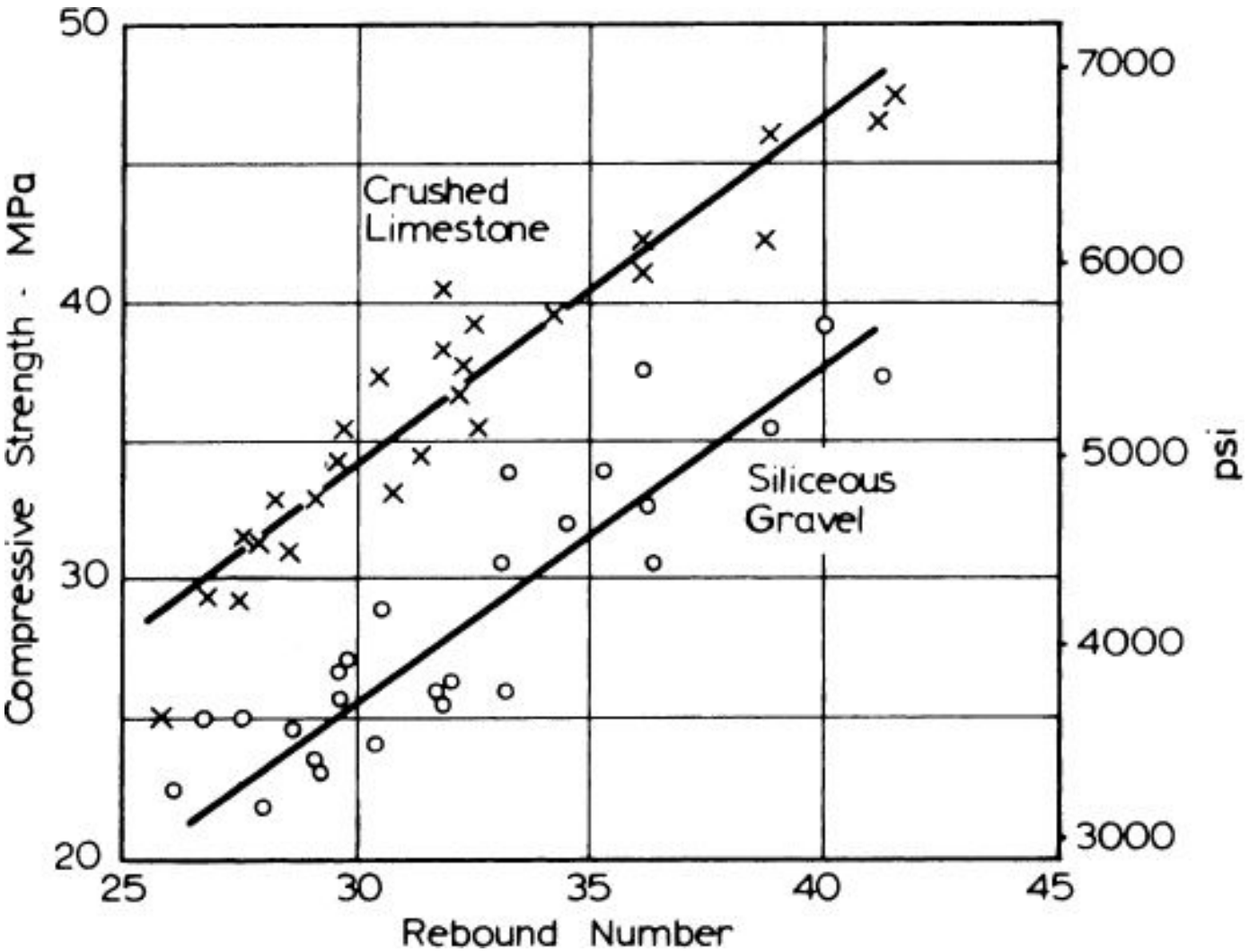


Fig. 12.30. Relation between compressive strength and rebound number for concrete cylinders made with different aggregates.[12.48] (Readings taken on the side of a cylinder with the hammer horizontal)

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The plunger must always be normal to the surface of the concrete under test, but the position of the hammer relative to the vertical will affect the rebound number. This is due to the action of gravity on the travel of the mass in the hammer. Thus, the rebound number of a floor is smaller than that of a soffit of the same concrete, and inclined and vertical surfaces yield intermediate values. For this reason, and also because of other factors which influence the rebound number, the use of 'global' diagrams relating the hardness number and strength is inadvisable. The correct procedure is to establish experimentally the relation between the rebound number measured on compression test specimens and their actual strength. If possible, the specimen mould material should be the same as the formwork material in the structure.

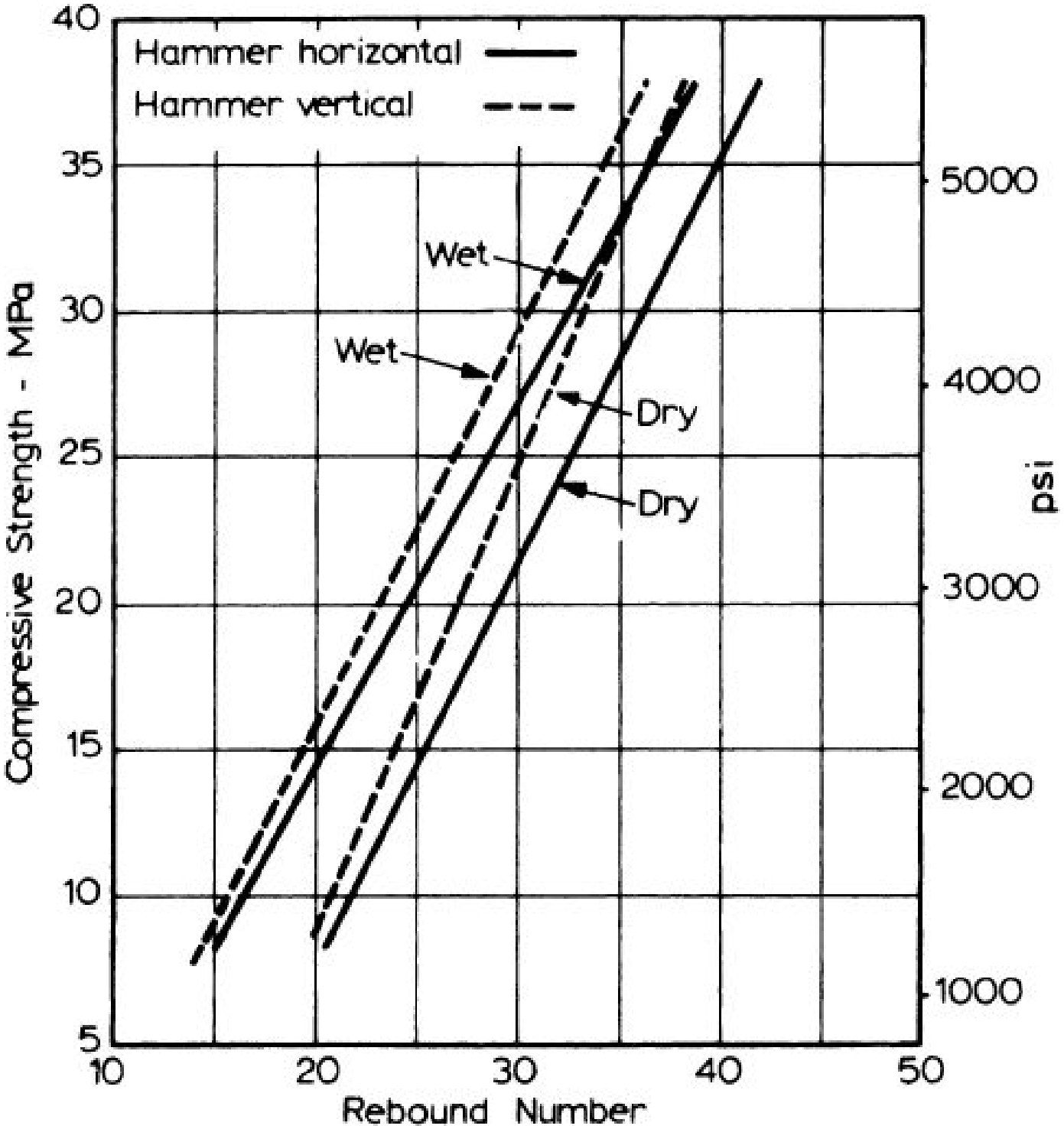
While the position of the curves relating the compressive strength to the rebound number varies, typically, for a change in strength of approximately 5 MPa (or 700 psi) there is a change of 4

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units in the rebound number. This relation is given only by way of illustration and cannot be relied upon to detect small differences in strength. It should be noted that different rebound hammers, even of the same design, cannot be assumed necessarily to give the same rebound number.

In any case, the rebound hammer test measures the properties of only the surface zone of concrete; according to BS 1881-202 : 1986 (withdrawn), the depth of this zone is about 30 mm (or $1\frac{1}{4}$ in.). Changes affecting only the surface of the concrete, such as the degree of saturation at the surface (which lowers the rebound number, see Fig. 12.31[12.47]) or carbonation (which raises the number[12.125]), have little influence on the properties of the concrete at depth.

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Fig. 12.31. Relation between compressive strength of cylinders and rebound number for readings with the hammer horizontal and vertical on a dry and a wet surface of concrete[12.47]

Because of local variability in the hardness of concrete over a small area, the rebound number should be determined at a number of locations in close proximity but, according to ASTM C 805-08 not closer than 25 mm (1 in.) apart. British Standard BS 1881-202 : 1986 (withdrawn) recommends testing on a grid pattern with a spacing of 20 to 50 mm within an area not larger than 300 by 300 mm (12 by 12 in.); this reduces the operator bias.

The rebound hammer test is largely comparative in nature and, as such, is useful in the assessment of uniformity of concrete within a structure or in the manufacture of a number of similar products such as precast elements. The test can also be used to establish whether the rebound number has reached a value known to correspond

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to the desired strength. This is of help in deciding when to remove falsework or to put the structure into service. Another use of the hammer is to check whether the strength development of a given concrete has been affected by frost at an early age but, according to ASTM C 805-08, still frozen concrete may give very high rebound numbers.

A particular application of the rebound hammer test is in assessing the abrasion resistance of concrete floors, which largely depends on surface hardness.

Overall, while the rebound hammer test is useful within a limited scope, the test is not a strength test and exaggerated claims of its use as a replacement for the compression test should not be accepted.

Penetration resistance test

The determination of the resistance of concrete to penetration by a steel rod, or probe, driven by a fixed amount of energy can be used to assess

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the compressive strength of concrete. The underlying principle is that, for standard test conditions, the depth of penetration is inversely proportional to the compressive strength of concrete, but no theoretical basis for this has been established. Moreover, the relation between strength and the depth of penetration greatly depends on the hardness of the aggregate because the coarse aggregate particles become fractured in the penetration tests, unlike in the compression test. Specifically, softer aggregate allows greater penetration than hard aggregate while the compressive strength may not be affected.[12.122]

The test equipment manufacturers supply 'standard' curves relating strength to the depth of penetration for concretes containing coarse aggregates with various values of hardness on Mohs' scale. However, different investigators found significantly different relations,[12.126] possible contributing factors being the shape and surface characteristics of the coarse aggregate.[12.135] Thus, the relation between strength and depth of

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penetration needs to be established by experiments for any given concrete. There is, however, some difficulty even in this because the same cylinder or cube cannot be used both for the penetration resistance test and for the compressive strength test as the former test weakens the specimen. Moreover, if the penetration resistance test is performed too close to the edge of the concrete, say less than 100 to 125 mm (4 to 5 in.), splitting can take place.

The test method for penetration resistance is prescribed by ASTM C 803-03 and by BS 1881 : 207 : 1992. For convenience, it is not the actual depth of penetration, but the complementary exposed length of a standard-length probe, that is measured. The penetration probes are driven in sets of three, the average value being used as a test result.

A typical relation between strength and the depth of penetration is shown in Fig. 12.32.

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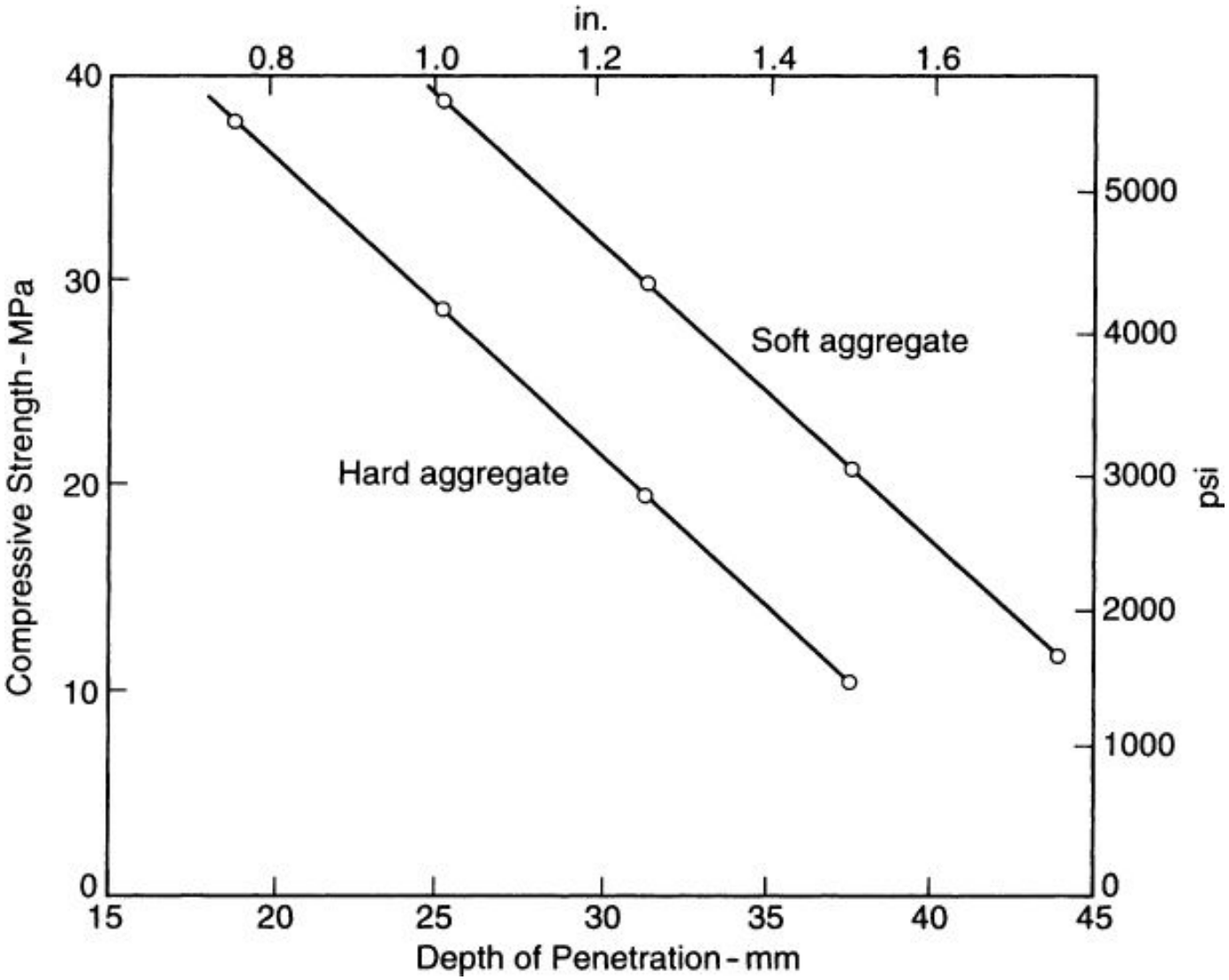


Fig. 12.32. Influence of the hardness of aggregate on the relation between depth of penetration and compressive strength (based on ref. 12.122)

The penetration resistance test is useful in determining whether formwork can be removed. The test has some advantages over the rebound hammer test because a greater depth of concrete

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is tested. Also, the number of tests required to detect, with adequate confidence, a given difference in strength is reported[12.140] to be smaller than when the rebound hammer test is used. However, the cost of the penetration resistance test is much higher. The penetration resistance test is likely to be preferable to drilling small-diameter cores.

Pull-out test

This is a test which measures, by means of a special tension jack, the force required to pull out a previously cast-in metal insert with an enlarged end (see Fig. 12.33). The insert is pulled out with a lump of concrete, approximately in the shape of a frustum of a cone. This shape is the consequence of the geometry of the insert together with the bearing ring. For a given geometry, the pull-out force is related to the compressive strength of concrete.

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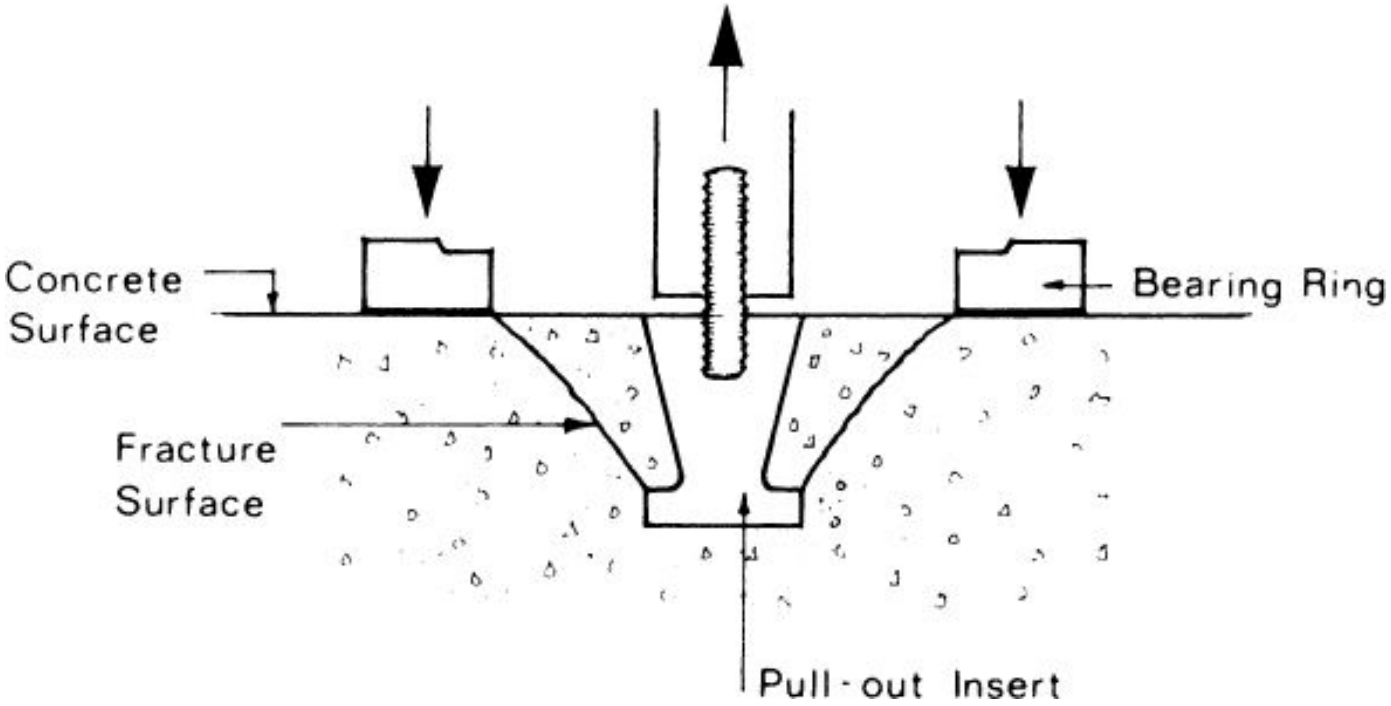


Fig. 12.33. Diagrammatic representation of the pull-out test

This relation is purely experimental, and not based on consideration of stresses involved, because the stress system at the fracture surface is three-dimensional: there are radial and circumferential tensile stresses, and a compressive stress along the surface of the cone.[12.136] Consequently, the pull-out force should be reported as such (in kN or lb), and calculations of a 'pull-out strength' lack reliable physical meaning. An example of the relation between the pull-out force and the

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strength of cores for a wide range of curing conditions is shown in Fig. 12.34.[12.105]

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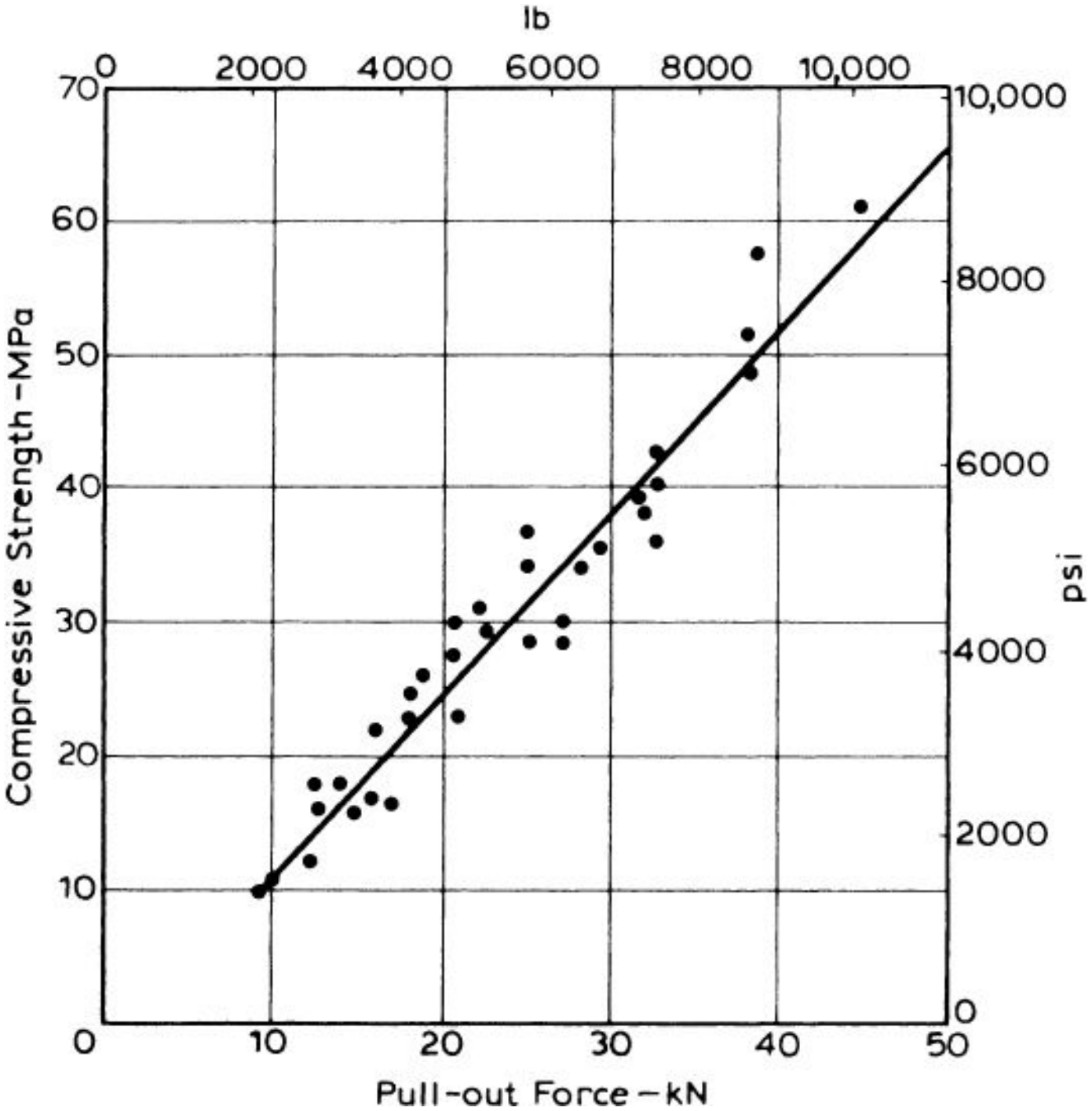


Fig. 12.34. Relation between compressive strength of cores and pull-out force for actual structures[12.105]

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The test method for the pull-out test is prescribed by ASTM C 900-06 and by BS 1881-207 : 1992. The ASTM Standard requires the depth of concrete above the enlarged end of the insert to be equal to the diameter of the enlarged end; the Standard also puts limits on the diameter of the bearing ring in relation to the diameter of the enlarged end of the insert. These limits ensure that the apex angle of the frustum of the cone lies between 54 and 70 degrees.[12.122]

According to Malhotra,[12.113] the pull-out test is superior to the rebound hammer test and to the penetration resistance test because a larger volume and a greater depth of concrete are involved in the pull-out test. On the negative side, repair of concrete is required. However, if the purpose of testing is to verify whether the concrete has reached a desired strength, the pull-out test need not be carried out to completion: it may be sufficient to apply a predetermined force to the embedded insert and, if it does not pull out, the desired strength is assumed to exist.

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Post-installed tests

A disadvantage of the pull-out test is that it requires pre-planned placing of inserts prior to concreting. To make it possible to perform a pull-out test without prior installation, several methods have been developed. These involve cutting a hole in the hardened concrete, undercutting the hole with a special tool, and inserting an expandable ring with a bolt attached to it. The pull-out test can then be performed in the usual manner.[12.139]

Other post-installed tests include the *internal fracture test*, which has proved useful in investigations of suspect concrete made with high-alumina cement.[12.129] In this test, a wedge-anchor bolt is tapped into a hole drilled into the concrete. The bolt is then pulled by turning a nut resting on a thrust pad on a spherical seating. The torque required to pull the bolt gives an assessment of the compressive strength of concrete, although the bolt, when pulled, applies both vertical and hori-

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zontal forces to the concrete.[12.140] As in the case of the pull-out test, the pulling can be stopped at a predetermined value of the torque, previously calibrated to correspond to a desired strength. The internal facture test is described in BS 1881-207 : 1992.

In the *break-off test*, it is possible to assess the flexural strength of concrete in a circular cross-section parallel to the surface of concrete. The section is formed by a tube inserted into fresh concrete or by drilling a sleeve. A jack is used to apply a transverse force in the element being broken off.[12.138] The break-off test is standardized in ASTM C 1150-90 (withdrawn) and BS 1881-207 : 1992.

There have also been developed *pull-off tests* involving measurement of the force required to pull off a part of the concrete, using a glued metal disc.[12.137] Thus, direct tension is applied but the area upon which it acts is uncertain. The pull-off test is prescribed in BS 1881 : 207 : 1992.

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Tests involving removal of a fragment of concrete are proliferating. Good reviews are offered by Bungey[12.135] and by Carino.[12.140]

Ultrasonic pulse velocity test

This is a long-established, non-destructive test method which determines the velocity of longitudinal (compressional) waves. This determination consists of measurement of the time taken by a pulse – hence the name of the method – to travel a measured distance. The apparatus includes transducers which are placed in contact with the concrete, a pulse generator with a frequency of between 10 and 150 Hz, an amplifier, a time measuring circuit, and a digital display of the time taken by the pulse of longitudinal waves to travel between the transducers. The test method is prescribed by ASTM C 597-09 and by BS EN 12504-4 : 2004.

The wave velocity, V, in a homogeneous, isotropic and elastic medium is related to the dynamic modulus of elasticity, E_d, by the expression:

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$$V^2 = \frac{E_d(1-\mu)}{\rho(1+\mu)(1-2\mu)}$$

where ρ is density, and μ is Poisson's ratio.

Concrete does not fulfil the physical requirements for the validity of the above expression, and the determination of the modulus of elasticity of concrete from the pulse velocity is not normally recommended.[12.63] Nevertheless, Nilsen and Aïtcin[12.117] have found it useful to do so in monitoring the modulus of elasticity of high strength concrete in service. It can be added that the value of Poisson's ratio (see p. 422) is generally not accurately known. However, a change in Poisson's ratio over the full range of possible values, that is from, say, 0.16 to 0.25, reduces the computed value of the modulus by only about 11 per cent.

With respect to the use of the value of the ultrasonic pulse velocity to determine the strength of concrete, it has to be stated that there is no physical relation between the two. It may be re-

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called that the modulus of elasticity is related to strength (see p. 419), but this relation, too, has no physical basis. However, the ultrasonic wave velocity is related to the density of concrete, as shown in the expression above. This last-mentioned relation offers the rationale for the use of the ultrasonic wave velocity measurements for the purpose of assessing the strength of concrete, but only subject to strict limitations, discussed below.

The velocity of the ultrasonic pulse through concrete is the outcome of the time taken by the pulse to travel through the hardened cement paste and through the aggregate. The modulus of elasticity of aggregates varies considerably, so that the pulse velocity of the concrete depends on the modulus of elasticity of the actual aggregate and on the aggregate content of the mix. On the other hand, the strength of concrete need not be significantly affected either by the content, or the modulus of elasticity, of the aggregate. Consequently, no unique relation between ultrasonic

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pulse velocity and compressive strength exists.[12.62] Figure 12.35 shows that there is a different relation for hardened cement paste, mortar, and concrete.

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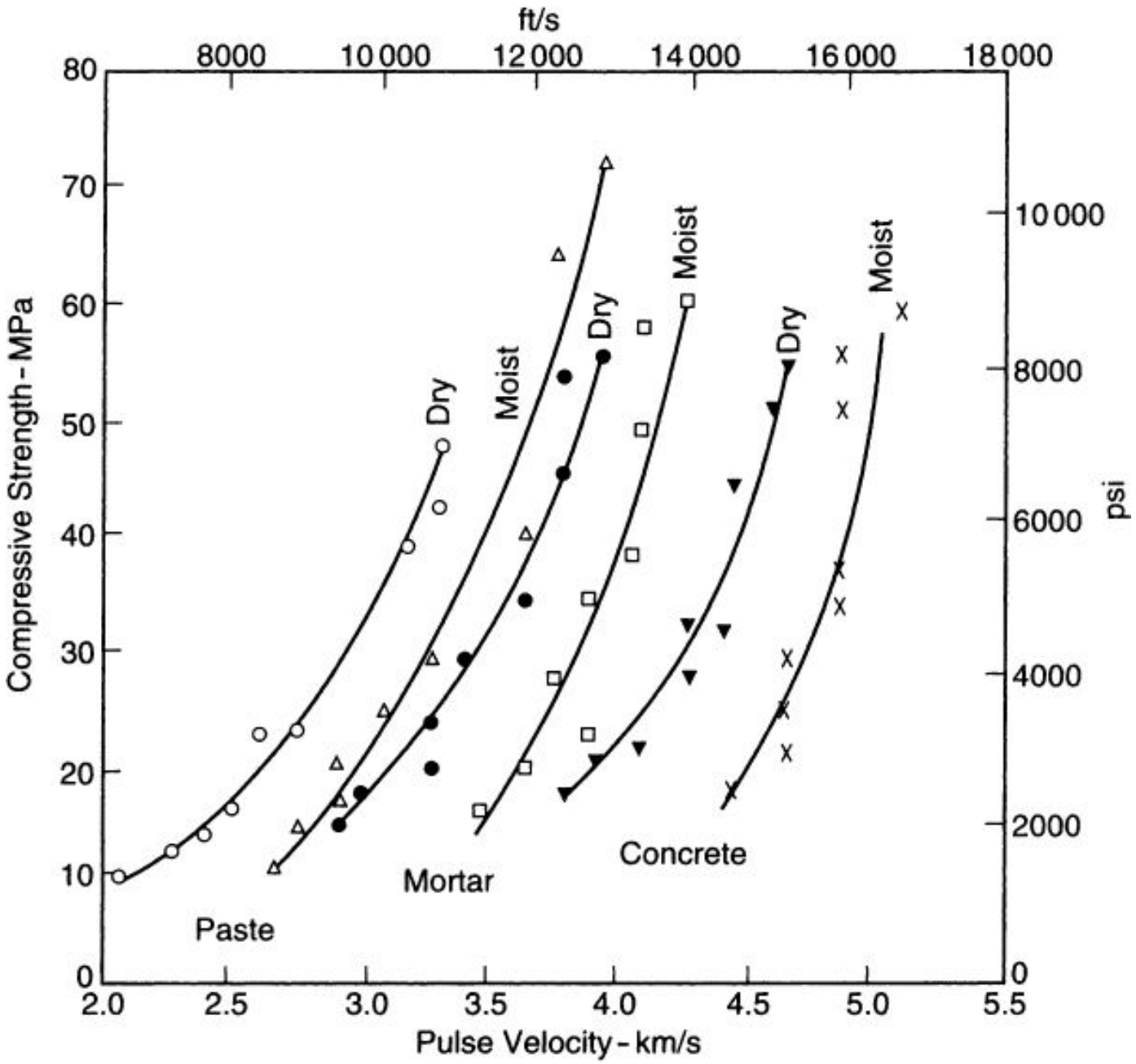


Fig. 12.35. Relation between compressive strength and ultrasonic pulse velocity for hardened cement paste, mortar, and concrete, in a dry and a moist condition (based on ref. 12.62)

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However, for a given aggregate and a given richness of the mix, the ultrasonic pulse velocity of the concrete is affected by changes in the hardened cement paste, such as a change in the water/cement ratio, which affects the modulus of elasticity of the hardened cement paste. It is only within these limitations that the ultrasonic pulse velocity test can be used to assess the strength of concrete. There is a further limitation arising from the fact that the pulse travels faster through a water-filled void than through an air-filled one. In consequence, the moisture condition of the concrete affects the pulse velocity while the strength in situ is not affected (see Fig. 12.35).

Avoiding other spurious influences is also essential: for instance, steel reinforcement, especially of large diameter, which lies along the path of the pulse leads to an increased ultrasonic pulse velocity, but does not affect the compressive strength of the concrete.[12.135]

Indeed, this is a particular case of the fundamental shortcoming of all non-destructive tests in

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which the property of concrete being measured is affected by various factors in a manner different from the influence of those factors on the strength of concrete.

Despite the limitations listed above, the ultrasonic pulse velocity test has the considerable merit of giving information about the interior of a concrete element. The test is, therefore, useful to detect cracking (but not parallel to the direction of the pulse), voids, deterioration due to frost or fire,[12.61] and the uniformity of concrete in similar elements. The test can be used for the purpose of following changes in a given concrete element, for instance, due to repeated cycles of freezing and thawing. It is useful to note that stress in concrete does not affect the value of ultrasonic pulse velocity.[12.142]

The ultrasonic pulse velocity test can also be used to assess the strength of concrete at very early ages, from about 3 hours onwards.[12.146] This is of interest in precasting or as an aid in de-

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ciding on removal of formwork, including steam-cured concrete.[12.143]

An echo type of the ultrasonic pulse technique makes it possible to measure the thickness of concrete roads and similar slabs.[12.79]

Further possibilities in non-destructive testing

We have discussed the various non-destructive test methods individually, but it is possible to use more than one method at a time. This is advantageous when a variation in properties of concrete affects the test results in opposite directions. Such is the case, for instance, with the presence of moisture in concrete: an increase in the moisture content increases the ultrasonic pulse velocity but decreases the rebound number recorded by the rebound hammer.[12.123] An example of the use of the combined results of these two test methods is given in Fig. 12.36. Recommendations on the use

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of combined non-destructive tests have been prepared by RILEM.[12.141]

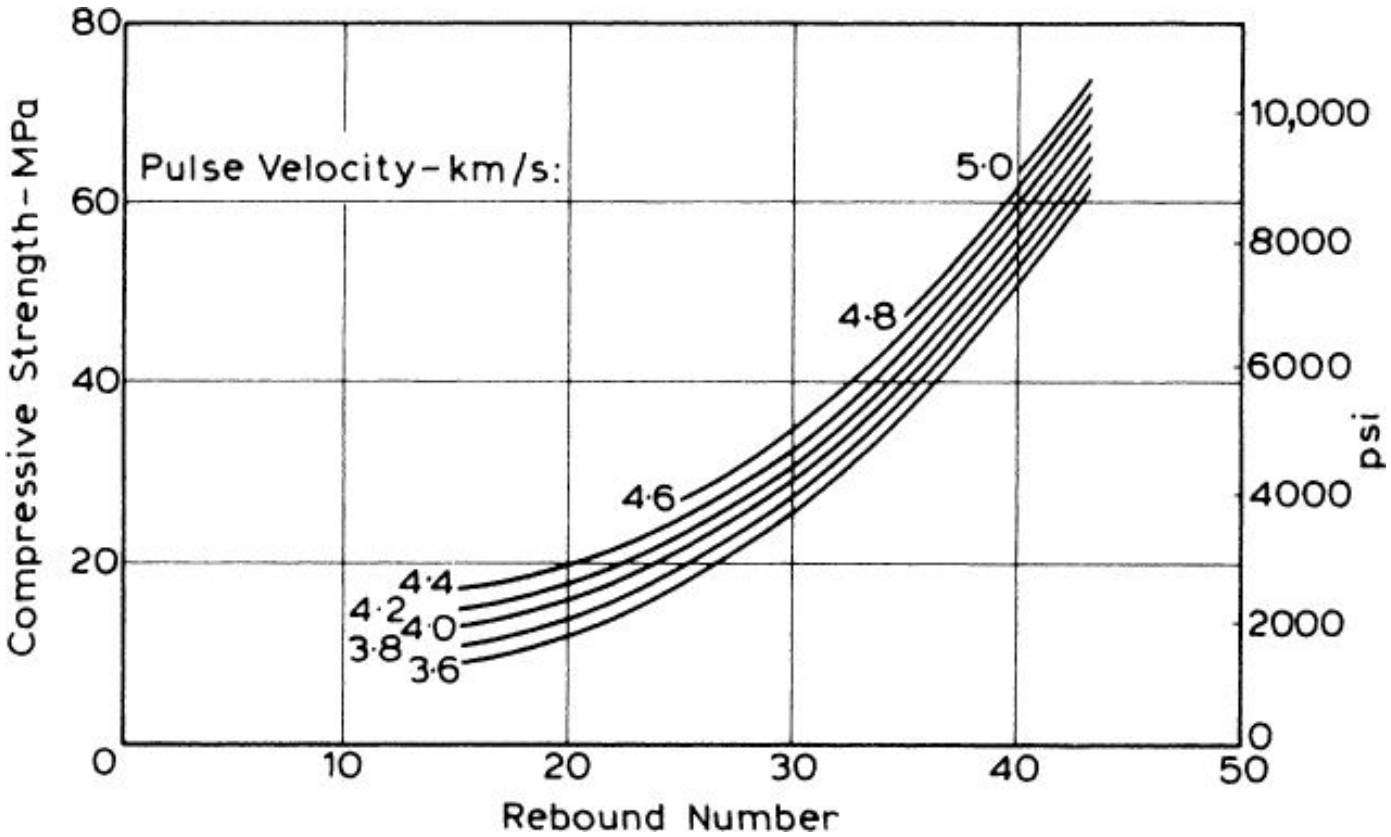


Fig. 12.36. Curves for the assessment of the in-situ compressive strength of concrete using the combined ultrasonic pulse velocity and rebound hammer tests[12.123]

There exist numerous other non-destructive tests on concrete in situ, some of which are still in the developmental stage. These tests include *radiography* using gamma rays or high-energy X-rays (to detect voids), *radiometry* (to measure

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density), *neutron transmission* or *reflection* (to estimate the moisture content of concrete), and *surface-penetrating radar* (to detect voids, cracks, or delamination). In the *impact–echo* technique, transient stress waves induced by impact become reflected by voids and cracks in concrete, and the resulting surface displacement is monitored near the point of impact. Thus, flaws in the interior of concrete can be detected.

The determination of acoustic emissions, which are transient elastic waves induced by stresses representing a high proportion of the ultimate strength, can be used to detect the development of cracking. The technique may be of value in assessing the remaining integrity of a structure that has undergone extreme loading.[12.66]

The various tests referred to above are not discussed in this book as its scope is limited to properties of concrete. However, one general comment should be made: all test results are variable and should, therefore, be interpreted in the light of their variability.

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Resonant frequency method

In some cases, it is desirable to determine the progressive changes in the state of a concrete specimen, for example, in consequence of repeated cycles of freezing and thawing or of chemical attack. This can be done by determining the fundamental resonant frequency of the specimen at appropriate stages of the investigation. From this frequency, the dynamic modulus of elasticity of the concrete can be calculated.

The vibration can be applied in a longitudinal, transverse (flexural) or torsional mode. The test method is prescribed in ASTM C 215-08 and BS 1881-209 : 1990; the latter standard covers the longitudinal mode only. In this mode, a specimen of specified dimensions (preferably similar to those used to determine the modulus of rupture) is clamped at its centre (Fig. 12.37) with a driving unit placed against one end face of the specimen and a pick-up against the other. The exciter is driven by a variable frequency oscillator with a range of 100 to 10 000 Hz. The vi-

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brations propagated within the specimen are received by the pick-up, amplified, and their amplitude is measured by an appropriate indicator. The frequency of excitation is varied until resonance is obtained at the fundamentel (i.e. lowest) frequency of the specimen; this is indicated by the maximum deflection of the indicator.

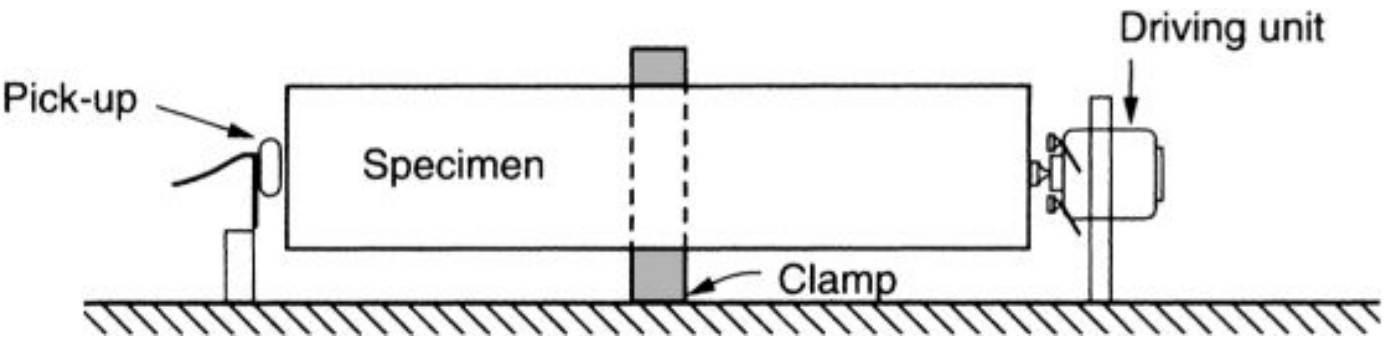


Fig. 12.37. Test arrangement for the determination of the dynamic modulus of elasticity in longitudinal vibration

If this frequency is n Hz, L is the length of the specimen, and ρ its density, then the dynamic modulus of elasticity is given by:

$$E_d = Kn^2L^2\rho,$$

where K is a constant.

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The length of the beam and its density have to be determined very accurately. If L of a specimen of square cross-section is measured in millimetres and ρ in kg/m^3, then E_d in GPa is given by:

$$E_d = 4 \times 10^{-15} n^2 L^2 \rho.$$

If L is measured in inches, and ρ is in lb/ft^3, then E_d in psi is:

$$E_d = 6 \times 10^{-6} n^2 L^2 \rho.$$

It should be emphasized that the dynamic modulus of elasticity calculated from the resonance frequency cannot be interpreted to represent the strength of concrete; the reasons for this were given in the section on the ultrasonic pulse velocity. It is only under strictly limited circumstances of a single concrete mix that changes in strength can be inferred from changes in the value of the modulus.

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Tests on the composition of hardened concrete

In some disputes about the quality of hardened concrete, the question is raised whether the composition of concrete was as specified and, to answer this, chemical and physical tests are made on a sample of hardened concrete. The primary interest is usually in the cement content and in the water/cement ratio, but the latter has to be derived from determinations of the cement content and original water content.

There are no universally applicable methods of chemical analysis because of the wide range of materials used to make concrete. If the original mix ingredients are available for testing, then the outcome of tests on a sample of hardened concrete is fairly reliable but, even then, the interpretation of the results of analysis requires engineering judgement based on practical experience.

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Cement content

There exists no direct method of determining the content of cement, even Portland cement alone, in a sample of concrete. The approach is to determine the contents of soluble silica and calcium oxide, and hence to calculate the cement content. The lower of the two values is used. The underlying basis is the fact that the silicates in Portland cement are much more readily decomposed and made soluble than are the silica compounds normally contained in aggregate. The same applies to the relative solubilities of the lime compounds in the cement and in aggregate (excepting, however, limestone aggregates), so that there exists also a soluble calcium oxide method.

Standard methods for the determination of the content of Portland cement are prescribed by ASTM C 1084-10 and BS 1881-124 : 1988, but the precision of the results is generally too low to prove, or otherwise, compliance with the original specification for the cement content; this is especially so in the case of mixes with low cement

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contents, and it is often in this type of mix that the exact value of cement content is required. Furthermore, the interpretation of the tests depends on the knowledge of the chemical composition of aggregate. When large amounts of both soluble silica and calcium oxide are liberated from the aggregate, the methods are even less reliable.

A guide to tests when various cementitious materials are present is given in the Concrete Society Report No. 32.[12.25] That report suggests that it is possible to calculate the slag content from a determination of the sulfide content in a sample of concrete, provided the composition of the slag used is known, but reliable results are difficult to obtain. No standard method for the determination of the fly ash content is available. Likewise, the determination of the presence and dosage of admixtures is not routinely possible because of the wide range of admixtures available and the low dosages used.[12.29]

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Determination of the original water/cement ratio

The water/cement ratio that existed at the time of placing of a concrete mix, now hardened, can be calculated from the cement content (determined as described in the preceding section) and from an estimate of the original water content. The original water is the sum of the mass of combined water in the cement and of the volume of capillary pores, which represents the remainder of the original water. The combined water can be taken as being equal to 23 per cent of the mass of the cement (see p. 26) or can be determined by igniting the sample at 1000 °C and measuring the water driven off. The test method is prescribed by BS 1881: Part 124: 1989. According to the Concrete Society Report No. 32,[12.25] there is no evidence to show that this method can be used for concretes made with blended cements. Even for Portland cement concrete, the calculated water/cement ratio is likely to be within 0.1 of the actual water/cement ratio.[12.25] An estimate

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of this accuracy is of little practical value. Other methods have been tried.[12.147] The precision of the determination of the water/cement ratio of hardened concrete is discussed in a recent paper by Neville.[12.149]

Physical methods

Guidance on petrographic examination of hardened concrete is given in ASTM C 856-04; ASTM C 457-10a covers other microscopical techniques which can be used to determine the volumetric composition of a sample in the form of a polished slice. These include the *linear traverse method* (see p. 558), the basis of which is the fact that the relative volumes of the constituents of a heterogeneous solid are directly proportional to their relative areas in a plane section, and also to intercepts of these areas along a random line. The aggregate and the voids (containing air or evaporable water) can be identified, the remainder being assumed to be hydrated cement. In order to convert the quantity of the latter to the

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volume of unhydrated cement, we have to know the specific gravity of dry cement and the non-evaporable water content of hydrated cement (see p. 37). The test determines the cement content of the concrete within 10 per cent, but the original water content or voids ratio cannot be estimated since no distinction is made in the test between air and water voids.

The *point-count method* is based on the fact that the frequency with which a constituent occurs at a given number of equally spaced points along a random line is a direct measure of the relative volume of that constituent in the solid. Thus, a point count by means of a stereomicroscope can rapidly give the volumetric proportions of a hardened concrete specimen.

Variability of test results

The variation in strength of nominally similar test specimens has been mentioned, and it follows that, whatever the test, the test results have to be interpreted in statistical terms. The mere fact that

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some test result is, for example, larger than some other result does not necessarily mean that the difference is *significant* and not a chance consequence of the natural variability of values from the same source. While all test results are variable, those derived from non-destructive tests generally have a larger variability than is the case with standard compressive strength specimens. Some of the simpler statistical terms will now be introduced.

Distribution of strength

Let us suppose that we have measured the compressive strength of 100 test specimens, all made from similar concrete. This concrete can be imagined to be a collection of units all of which could be tested; such a collection is referred to as the *population*, and the portion of concrete in the actual test specimens is called the *sample*. It is the purposes of the tests on the sample to supply information on the properties of the parent population.

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From the nature of the strength of concrete (p. 291) it would be expected that the recorded strengths will be different for different specimens, i.e. the results will show a scatter. To illustrate this let us consider the results on specimens tested in the construction of an offshore platform,[12.95] shown in Table 12.5. A good picture of the distribution of these strengths can be obtained by grouping the actual strengths in intervals of 1 MPa, so that we now have a certain number of specimens whose strength falls within each interval, as in Table 12.5.

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Table 12.5. Example of Distribution of Strength Test Results[12.95]

Strength interval, MPa	*Number of specimens in interval*	*Strength interval, MPa*	*Number of specimens in interval*
42–43	1	55–56	51
43–44	1	56–57	59
44–45	0	57–58	54
45–46	0	58–59	32
46–47	3	59–60	23
47–48	3	60–61	7
48–49	8	61–62	10
49–50	11	62–63	3
50–51	31	63–64	1
51–52	31	64–65	2
52–53	37	65–66	0
53–54	55	66–67	1
54–55	69		—
			Total = 493

If we now plot the (constant) strength interval as abscissae and the number of specimens in each interval (known as frequency) as ordinates we obtain a *histogram*. The area of the histogram represents the total number of specimens to an appropriate scale. Sometimes, it is more convenient to express the frequency as a percentage of

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the total number of specimens, i.e. to use relative frequency.

The histogram for the above-mentioned data is plotted in Fig. 12.38, and it can be seen that it gives a clear picture of the scatter of results or, more accurately, of the distribution of strength within the sample tested.

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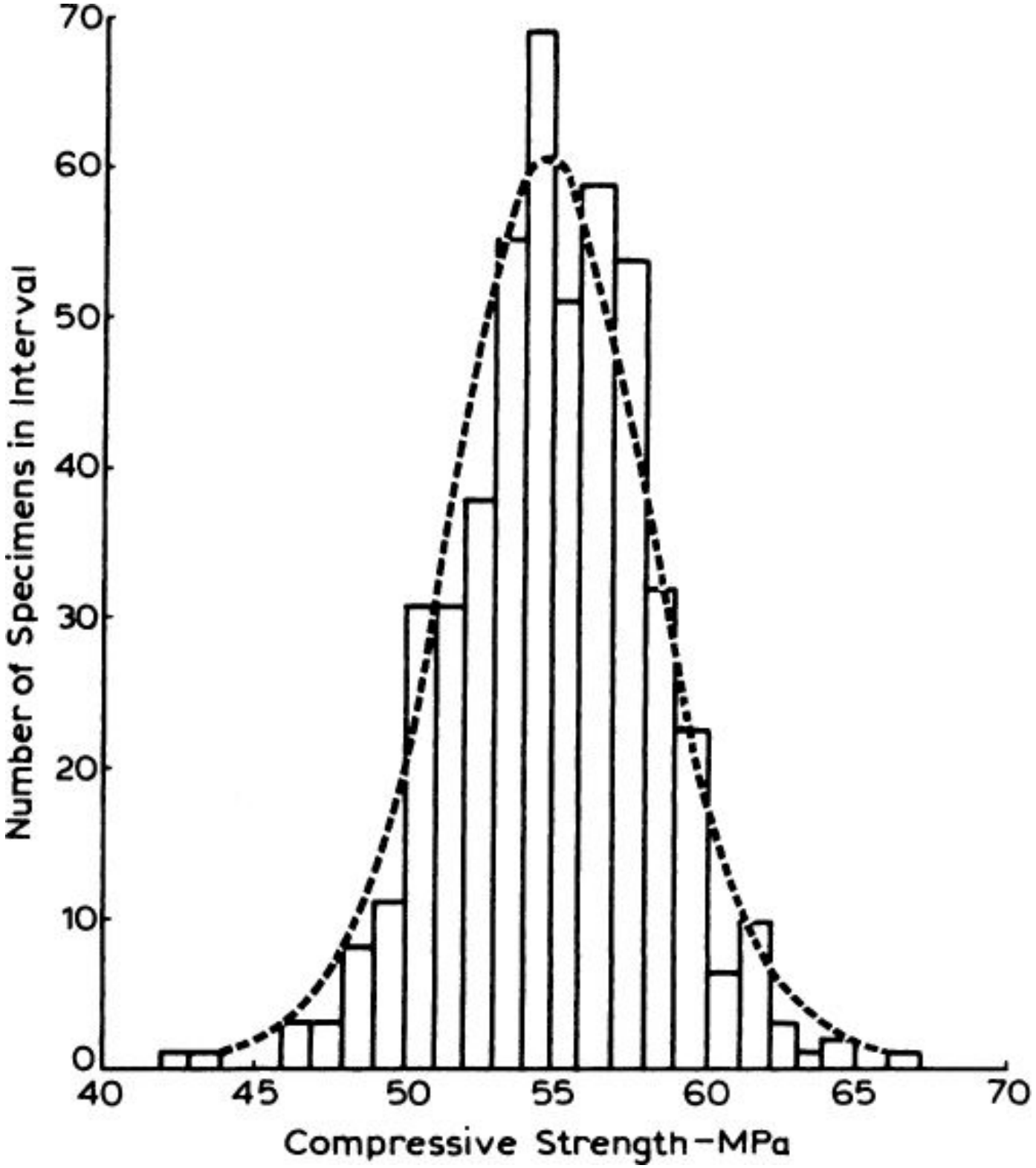


Fig. 12.38. Histogram of strength values of p. 637[12.95]

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Another simple measure of dispersion is given by the *range* of values, i.e. the difference between the highest and the lowest strengths: 25 MPa in the above case. The range is, of course, calculated extremely rapidly, but it is a rather crude measure: it depends on two values only, and furthermore, in a large sample, these values are of low frequency; thus range increases with sample size for the same underlying distribution. The theoretical relation between range and standard deviation is shown in Fig. 12.39, together with data obtained in practice.

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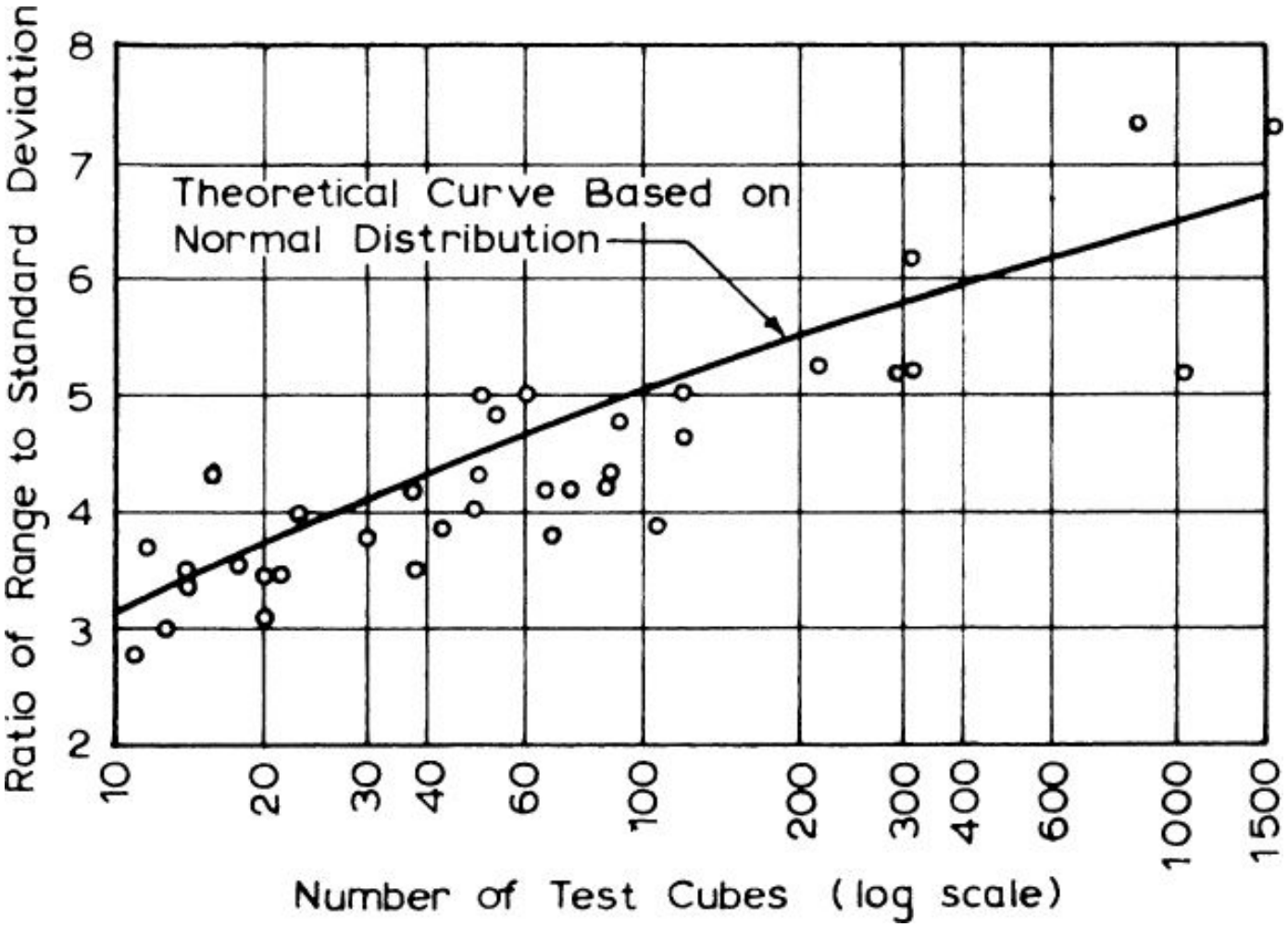


Fig. 12.39. Ratio of range to standard deviation for samples of different size[12.26] (Crown copyright)

If the number of specimens is increased indefinitely and, at the same time, the size of the interval is decreased to a limiting value of zero, the histogram would become a continuous curve, known as the *distribution curve*. For the strength of a certain type of material, this curve would

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have a characteristic shape, and there are, in fact, several 'type' curves whose properties have been calculated in detail and are listed in standard statistical tables.

One such type of distribution is the so-called *normal* or Gaussian distribution. The applicability of this type of distribution to the strength of concrete was mentioned on p. 605; the assumption of normal distribution is sufficiently close to reality to be an extremely useful tool in computations (see Fig. 12.38).

The equation to the normal curve, which depends only on the values of the mean, μ, and standard deviation, σ, is:

$$y = \frac{1}{\sigma\sqrt{2\pi}} e^{\frac{-(x-\mu)^2}{2\sigma}}$$

The standard deviation is defined in the next section. This equation is represented graphically in Fig. 12.40, and it can be seen that the curve is symmetrical about the mean value and extends to

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plus and minus infinity. This is sometimes mentioned as a criticism of the use of normal distribution for strength, but the extremely low probability of the occurrence of the very high or very low values is of little practical significance.

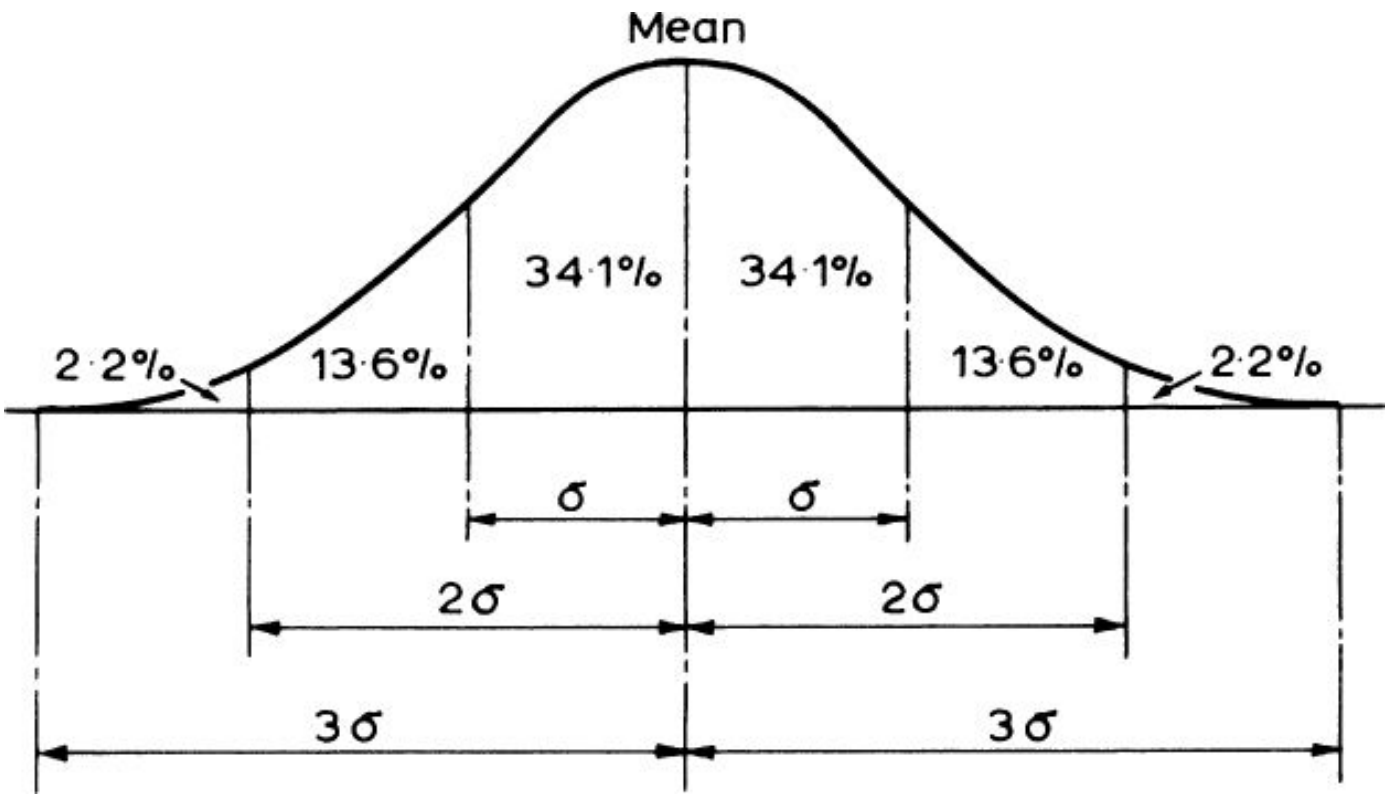


Fig. 12.40. Normal distribution curve; percentage of specimens in intervals of one standard deviation shown

The area under the curve between certain values of strength (measured in terms of standard deviation) represents, in a manner similar to the histogram, the proportion of specimens between

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the given limits of strength. Since, however, the curve refers to an infinite population of specimens, and we deal with a limited number of them, the area under the curve between given ordinates, expressed as a fraction of the total area under the curve (and known therefore as proportional area), measures the chance that the strength of an individual drawn at random, x, will lie between the given limits. This chance multiplied by 100 gives the percentage of specimens that may be expected in the long run to have a strength between the two limits considered. Statistical tables give the values of proportional areas for different values of $(x - \mu)/\sigma$.

Standard deviation

It can be seen from the foregoing discussion of probability that the dispersion of strength about the mean is a fixed function of the standard deviation. This is defined as the root-mean-square deviation, i.e.

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$$\sigma = \left(\frac{\sum (x - \mu)^2}{n} \right)^{1/2}$$

where x represents the values of strength of all n specimens, and μ is the arithmetic mean of these strengths, i.e. $\mu = \sum(x/n)$.

In practice, we deal with a limited number of specimens, and *their* mean, $\bar{x}$, is our estimate of the true (population) mean μ. We calculate the deviations from $\bar{x}$ and not from μ, and therefore put $(n - 1)$, instead of n, in the denominator of the expression for the estimate of σ. The reason for this correction of $n/(n - 1)$, known as Bessel's correction, is that the sum of squares of deviations has a minimum value when taken about the sample mean, $\bar{x}$, and is therefore smaller than it would be if taken about the population mean μ. (Bessel's correction need not be applied when n is large.) Thus, the estimate of σ is

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$$s = \left(\frac{\Sigma (x - \bar{x})^2}{n - 1} \right)^{1/2}.$$

An important practical point is that one value (e.g. the result of one specimen test) yields no information whatever about the standard deviation and, therefore, about the reliability or possible 'error' of the value obtained. Many calculators are programmed to calculate the standard deviation direct but, for hand calculation, a more convenient form of the expression for the standard deviation is

$$\sigma = \left(\frac{(\Sigma x^2)}{n} - \bar{x}^2 \right)^{1/2} = \frac{1}{n} (n\Sigma(x^2) - (\Sigma x)^2)^{1/2}.$$

Thus, the sum of x^2 is obtained without first finding the differences $(x - \bar{x})$. Other simplifications, such as subtracting a fixed quantity from all values, further aid computation. To find s, Bessel's correction is applied:

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$$s = \sigma\left(\frac{n}{n-1}\right)^{1/2}.$$

The standard deviation is expressed in the same units as the original variate, x, but for many purposes it is convenient to express the scatter of results on a percentage basis. We take then the ratio $(\sigma/\bar{x}) \times 100$, and this is called the *coefficient of variation*. It is a dimensionless quantity.

The graphical representation of the standard deviation (see Fig. 12.40) is the horizontal distance from the mean to the point of contraflexure of the normal distribution curve. Since the curve is symmetrical, the area under the curve contained between abscissae $\mu - \sigma$ and $\mu + \sigma$ is 68 per cent of the total area under the curve. In other words, the probability that the strength of a test specimen chosen at random lies within the range $\mu \pm \sigma$ is 0.68. The probabilities for other deviations from the mean are indicated in Fig. 12.40.

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For a given mean strength, the standard deviation characterizes fully the distribution, assumed to be of the normal type; the variation in the value of the standard deviation determines the spread of strengths in MPa or psi. It is useful to add that the precision with which $\bar{x}$ estimates the value of the population mean μ is governed by the standard deviation of the mean, known as the standard error σ_n where $\sigma_n = \sigma/\sqrt{n}$. Thus, there is a probability of 0.68 that $\bar{x}$ is within the interval $\mu \pm \sigma_n$.

The distribution curves for values of standard deviation of 2.5, 3.8, and 6.2 MPa (350, 560, and 900 psi) are shown in Fig. 14.3. The value of the standard deviation affects the (mean) strength that has to be aimed at in mix design for a given 'minimum' or characteristic strength specified by the designer of the concrete structure. This problem is discussed fully in Chapter 14. Details of statistical methods applicable to testing, particularly data on the choice of sample size, have to be sought in specialized books.* The terms preci-

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sion, repeatability and reproducibility are defind in BS ISO 5725-1 : 1994.

[*]E.g. J. B. KENNEDY and A. M. NEVILLE, *Basic Statistical Methods for Engineers and Scientists*, 3rd Ed. (New York and London, Harper and Row, 1986).

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Chapter 13. Concretes with particular properties

This chapter deals with several types of concretes which can be used when particular properties are required. The term ‘particular’ does not imply anything unusual or rarely required; rather, it refers to specific properties which are desirable under the given circumstances. Several types of concretes will be considered. First, concretes containing the different cementitious materials which are frequently used nowadays (discussed in Chapter 2). These are: fly ash, ground granulated blastfurnace slag, and silica fume.[13.90]

The second type of concrete to be considered is the so-called high performance concrete. This concrete invariably contains one or more of the cementitious materials mentioned above, and usually a superplasticizer as well. The term ‘high performance’ is somewhat pretentious because

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the essential feature of this concrete is that its ingredients and proportions are specifically chosen so as to have particularly appropriate properties for the expected use of the structure; these properties are usually a high strength or a low permeability.

The third, and last, type of concrete discussed in this chapter is lightweight concrete, that is, concrete with a density significantly lower than the density of concretes made with normal aggregates, which is in the range of 2200 to 2600 kg/m^3 (140 to 160 lb/ft^3).

One more type of concrete should be mentioned: high-density concrete, which is used for the purpose of attenuation of high-energy X-rays, gamma rays, and neutrons. Because of this specialized use of high-density concrete, it will not be considered in the present book.

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Concretes with different cementitious materials

The preceding chapters have dealt with concretes which may contain a range of cementitious materials, but mainly with concretes containing Portland cement only. The reason for this approach is that, until fairly recently, Portland cement was considered as the 'best', if not the sole, cementitious material in concrete. When other materials, primarily fly ash and ground granulated blastfurnace slag, were introduced, they were viewed as replacements or substitutes for cement, and their influence and performance were judged against the standard of concrete containing only Portland cement.

The situation has changed dramatically: as pointed out on p. 90, several cementitious materials are today concrete ingredients in their own right. These materials, fly ash, ground granulated blastfurnace slag (for brevity, referred to as *ggbs*), and silica fume, were discussed in Chapter 2 in so far as their physical and chemical prop-

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erties are concerned. When various properties of concrete were considered in subsequent chapters, the influence of these materials was often mentioned. This, however, was unavoidably fragmentary, and it is now proposed to review the properties of concretes containing the various cementitious materials.

It can be argued that the influences of the individual cementitious materials should be discussed first. On the other hand, a brief review of these materials considered together is useful in painting a general picture of their role in the behaviour of concrete. Therefore, the common features of two, or all three, of these materials and the use of more than one of them at a time will be discussed. This will be followed by their individual consideration.

General features of use of fly ash, ggbs, and silica fume

An argument which is sometimes advanced in favour of the use of these various cementitious ma-

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terials is that, compared with Portland cement, they save energy and conserve resources. This is factually correct, but it is the actual technical benefits of the inclusion of these materials in the concrete that are the strongest argument in favour of their use. Indeed, in many cases, they should be used in preference to a Portland-cement-only mix regardless of economic or environmental considerations.

There is some difficulty in presenting the available information on the influence and use of the three cementitious materials – fly ash, ggbs, silica fume – in an objective and generally valid way. An extremely large number of research papers have been published but, in many of them, an enthusiastic researcher describes a single set of tests on one of these materials and points out the benefits of the use of that particular material, which often is a specific local product. This description may well be a true and factual account, but the conclusions are usually couched in terms of a comparison with a 'reference' mix contain-

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ing Portland cement only. The differences between the mix with the given cementitious material and the 'reference' mix may include workability, strength at some age or another, total content of cementitious material or water/cement ratio; any one of these may be of importance in construction. A worthwhile generalization from such a comparison is not possible. What is of use is a general review of the pattern of properties of the mixes containing the different cementitious materials. This should make it possible to assess the properties of concretes with different ingredients, possibly in different proportions. Specific properties of any given mix have to be ascertained by experiment.

The various cementitious materials affect the progress of hydration in consequence of their chemical composition, reactivity, particle size distribution, and particle shape.[13.9] The actual reactivity of ggbs depends on its composition, glass content, and particle size.[13.9] High-calcium fly ash (ASTM Class C, BS EN class W) is much

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more reactive than Class F (BS EN Class V) fly ash and, therefore, exhibits some similarity to the behaviour of ggbs.[13.9] The reaction of Class F fly ash requires a high alkalinity of the pore water. This alkalinity is reduced when silica fume or ggbs are present in the mix. In consequence, the reactivity of fly ash in such mixes is reduced.[13.15]

At a given total content of cementitious material the inclusion of fly ash or ggbs generally reduces the water demand and improves workability. In the case of ggbs, the improvement may not be measurable in terms of slump but, once vibration has started, concrete containing ggbs becomes 'mobile' and compacts well. Silica fume greatly reduces, or even eliminates, bleeding. The improvement of workability by fly ash is ascribed to the spherical shape of its particles. However, the inclusion of fly ash and, to a lesser extent, of ggbs in the mix has the physical effect of modifying the flocculation of cement, with a resulting reduction in the water demand.[13.9] The changed dispersion of cement particles is reflected in the

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microstructure of the hydrated cement paste, mainly its pore size distribution, the median pore size being smaller and, consequently, the permeability being lower.[13.9] This effect is present at a constant total porosity (which is controlled by the overall water/cement ratio).

The improvement of the strength of concrete by fly ash is not only the consequence of its pozzolanicity but also of the ability of the very small fly ash particles to 'fit in' between cement particles. Proof of this is provided by the beneficial effect of fly ash used with Portland blast-furnace cement when pozzolanic reaction is unlikely[13.12]

Durability aspects

Although an early reason for the use of the various cementitious materials in concrete was their influence on the rate of development of heat and of strength, even more important is their influence on the resistance of concrete to chemical attack, which is the consequence not only of the

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chemical nature of the hydrated cement paste but also of its microstructure. This topic was considered in Chapters 10 and 11. It is no exaggeration to say that the cementitious materials have a major influence on all aspects of durability related to the transport of attacking agents through concrete. A reason for this is that, generally, the cementitious materials considered in this chapter are finer than Portland cement and, therefore, improve particle packing, so that, provided adequate wet curing is applied, their presence reduces permeability.[13.92]

Even though the use of fly ash or ggbs reduces permeability, it allows faster carbonation.[13.113] The increase in the rate of carbonation is greater when fly ash is used with Portland blastfurnace cement.[13.12] When the ggbs plus fly ash content is more than 60 per cent, the increase in carbonation is greater the greater the fly ash content.[13.13] The enhanced carbonation need not necessarily be large in practice when mixes with proper mix proportions are used. Also, carbonation may re-

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duce the permeability, but not when both fly ash and ggbs are present in the mix.[13.12] Good resistance to freezing and thawing without air entrainment was found in concretes (with a water/cement ratio of 0.27 and a superplasticizer) containing Class C fly ash representing 20 to 35 per cent of the mass of the total cementitious material, and silica fume (10 per cent on the same basis).[13.11] Likewise, good resistance to sulfate attack was observed with Class C fly ash contents up to 50 per cent and 10 per cent of silica fume.[13.11]

Control of the alkali–silica reaction is a specialized topic in which a detailed knowledge of the aggregate to be used is necessary (see p. 144). However, the beneficial effects of the incorporation of fly ash (about 30 to 40 per cent by mass) or of ggbs (about 40 to 50 per cent by mass) in the blended cement should be noted.[13.7] These materials contain only a small amount of *water-soluble alkalis* so that, at a given content of cementitious material which includes Portland cement with a high alkali content, the presence of

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ggbs or fly ash in the blended cement reduces the total alkali content in the mix.[13.10] Thus, the use of these materials may obviate the need for low-alkali cement but the absence of expansive reactions should be verified by tests.

The beneficial effects of the inclusion of silica fume in steam-cured concrete at 65 °C (149 °F) upon its penetrability by chlorides was confirmed by Campbell and Detwiler.[13.4] For significant improvement, the minimum silica fume content was 10 per cent in Portland-cement-only concrete, but 7.5 per cent was highly effective in mixes containing 30 to 40 per cent of ggbs in the total cementitious material.[13.4] It may be added that curing Portland-cement-only concrete at 50 °C (122 °F) was found to result in increased penetrability by chlorides.[13.3]

Further studies by Detwiler *et al.*[13.2] confirmed the beneficial effect of inclusion of both silica fume and ggbs in concrete cured at 50 and 70 °C (122 and 168 °F) upon penetrability of

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chlorides. These findings were obtained on concretes with water/cement ratios of 0.40 and 0.50 and with silica fume and ggbs contents of 5 and 30 per cent, respectively, by mass of the total cementitious material. Generalizations of optimum contents or proportions are not possible because the penetrability of the resulting concrete is affected by the degree of hydration at the time of exposure to chlorides. Information on the influence of inclusion of both silica fume and fly ash in concrete cured at high temperatures upon its permeability to chlorides does not seem to be available.

Variability of materials

The three cementitious materials discussed in this chapter are not manufactured specifically for use in concrete but are industrial by-products. This situation is reflected in their variability.

Fly ash is a by-product of burning pulverized coal to generate electric power. The power station operators are aware of the commercial value of a

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uniform fly ash, but periodic variations in the operation of a power station (especially if not a base supply station) can result in occasionally varying properties of the fly ash. There are, of course, also differences in the fly ash produced by different power stations. Moreover, even the same power station will produce fly ash with varying properties if the coal used is non-uniform in the short- or long-term. Ash classification and beneficiation would be helpful but they would increase the cost of the fly ash.

It follows that the users of fly ash have to be aware of the properties of the actual material used in concrete, and they cannot rely on standardized assumptions about the particle size distribution of fly ash or its carbon content. In consequence, a simple picture of the behaviour of concrete containing fly ash cannot be presented because fly ash is not a single material of nearly constant composition. Fly ashes are rather like the various Types of Portland cements in that the fly ashes also have a range of physical and chemical char-

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acteristics. It is, therefore, not surprising that the use of fly ash, especially because its content in concrete can vary widely, results in a range of effects.

On the other hand, slag, being a by-product of a highly controlled process (see p. 79), is much less variable; the same applies to silica fume.

Returning to the subject of fly ash, we should note that the hydration of any given fly ash depends on the chemical properties and the fineness of the Portland cement in the mix. It is not surprising that there is no simple relation between the proportion of fly ash in the total cementitious material and the properties of the resulting concrete of otherwise fixed proportions. Inevitably, attempts to relate, by a single equation, the strength of concrete, even of fixed proportions, to the various properties of fly ash such as fineness, residue of particles above a certain size, pozzolanic indices, carbon content, glass content, and chemical composition, have been unsuccessful.[13.6] Indeed, this situation is to be expected,

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given that no single equation can predict the strength properties of Portland cements alone from their physical and chemical properties.

Fly ash and ggbs are very valuable ingredients of concrete. They are also economically advantageous because they are by-products of other processes, and are continuously available – indeed, in need of being disposed of. It is worth reflecting that, in consequence of changes in our industrial patterns, especially in the consumption of iron and in the sources of energy, less fly ash and less slag may become available in the future (see also p. 656). New cementitious materials might need to be developed.

Concrete containing fly ash

A brief description of the physical and chemical properties of fly ash was presented in Chapter 2. We shall now consider the use of fly ash in concrete and discuss the properties of the resulting concrete; a further discussion of the properties of

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the fly ash itself, in so far as they affect the properties of concrete, will also be included.

The importance of fly ash cannot be exaggerated: it is no longer a cheap substitute for cement, nor an ‘extender’ or an addition to the mix. Fly ash bestows important advantages upon concrete, and it is, therefore, essential to understand the role and influence of fly ash.

The variability of the properties of fly ash was mentioned in the preceding section. This variability arises from the fact that fly ash is not a specially manufactured product and cannot, therefore, be governed by strict requirements of a standard. The main influences are the nature of the coal and the manner of its pulverization, the operation of the furnace, the process of precipitation of ash from the combustion gases, and especially the extent of classification of the particles in the exhaust system. Even when all these are constant, a power station which varies its operation in response to the power demand produces a variable fly ash; this is not so with a base-load

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power station. The variations in the fly ash are those in glass content, carbon content, particle shape and size distribution, as well as in the presence of magnesia and other minerals, and even in colour. It is possible to improve the size distribution of fly ash particles by classification and by grinding.

As just mentioned, the burning process of pulverized coal influences the shape of the fly ash particles. High temperature favours the formation of spherical particles, but the need to reduce the emission of NO_x gases requires the use of lower peak burning temperatures so that the minerals with a high melting point do not always fuse completely. A consequence of this is a reduction in the proportion of spherical particles of fly ash and also in the proportion of particles smaller than 10 μm; however, the proportion of particles larger than 45 μm is not affected.[13.12,13.34] These changes militate against the beneficial effects of fly ash in concrete. Thus, there is need for changes in technology which will satisfy both the

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NO_x emission requirements and the particle properties desirable from the standpoint of their use in concrete.

It should be pointed out, however, that, in most countries, much uniform and excellent fly ash for use in concrete is consistently produced, and there is no doubt that, world-wide, the consumption of fly ash in concrete increases and is expected to continue to do so. What is not possible is to provide information about a 'standard', or even typical, fly ash. Consequently, specific guidance on the use of fly ash as a generic material cannot be presented.

Influence of fly ash on properties of fresh concrete

The main influence is that on water demand and on workability. For a constant workability, the reduction in the water demand of concrete due to fly ash is usually between 5 and 15 per cent by comparison with a Portland-cement-only mix having the same cementitious material content;

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the reduction is larger at higher water/cement ratios.[13.12]

A concrete mix containing fly ash is cohesive and has a reduced bleeding capacity. The mix can be suitable for pumping and for slipforming; finishing operations of fly ash concrete are made easier.

The influence of fly ash on the properties of fresh concrete is linked to the shape of the fly ash particles. Most of these are spherical and solid, but some of the large particles are hollow spheres, known as cenospheres, or are vesicular and irregular in shape.

The reduction in water demand of concrete caused by the presence of fly ash is usually ascribed to their spherical shape, this being called a ‘ball-bearing effect’. However, other mechanisms are also involved and may well be dominant. In particular, in consequence of electrical charges, the finer fly ash particles become adsorbed on the surface of the cement particles. If enough fine fly ash particles are present to

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cover the surface of the cement particles, which thus become deflocculated, the water demand for a given workability is reduced.[13.156] An amount of fly ash in excess of that required to cover the surface of the cement particles would confer no further benefit with respect to water demand. Indeed, the reduction in water demand becomes larger with an increase in the fly ash content only up to about 20 per cent.[13.156] The effect of fly ash is not additional to the action of superplasticizers. Thus, it seems likely that the action of fly ash, like that of superplasticizers, on water demand is through dispersion and adsorption of the fly ash on the particles of Portland cement.[13.156] Malhotra recommends the use of fly ash above 50 per cent of cementitious material but this is not universally accepted.[13.160]

The presence of carbon in fly ash was referred to on p. 85. One consequence of a high carbon content in fly ash is that it adversely affects workability. Variation in carbon content may also lead to erratic behaviour with respect to air entrain-

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ment, some air-entraining agents becoming adsorbed by the porous carbon particles.

Fly ash in the mix has a retarding effect, typically of about 1 hour, probably caused by the release of SO_4^{--} present at the surface of the fly ash particles. The retardation may be advantageous when concreting in hot weather; otherwise, an accelerator may be needed. Only initial setting is delayed, the time interval between setting and final stiffening being unaffected.

The delayed setting is additional to the retarding effect of some admixtures by lower temperatures. This may lead to blistering and delamination.[13.160] The preceding statement is not an argument against using a high content of fly ash in the cementitious material, but simply as an indication of the need to establish the properties of such concrete in the presence of admixtures.

Hydration of fly ash

Pozzolanic reactions were considered in Chapter 2. In the case of fly ash, the products of reaction

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closely resemble C-S-H produced by hydration of Portland cement. However, the reaction does not start until sometime after mixing. In the case of Class F fly ash (see p. 85), this can be as long as one week or even more. An explanation of this delay, offered by Fraay *et al.*[13.15] is as follows. The glass material in fly ash is broken down only when the pH value of the pore water is at least about 13.2, and the increase in the alkalinity of the pore water requires that a certain amount of hydration of the Portland cement in the mix has taken place. Moreover, the reaction products of Portland cement precipitate on the surface of the fly ash particles, which act as nuclei.

When the pH of the pore water becomes high enough, the products of reaction of the fly ash are formed on the fly ash particles and in their vicinity. A consequence of these early reactions is that their products often remain in the shape of the original spheres of fly ash. With the passage of time, further products diffuse away and precipitate within the capillary pore system; this

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results in a reduction in the capillary porosity and, consequently, a finer pore structure (see Fig. 13.1).[13.15]

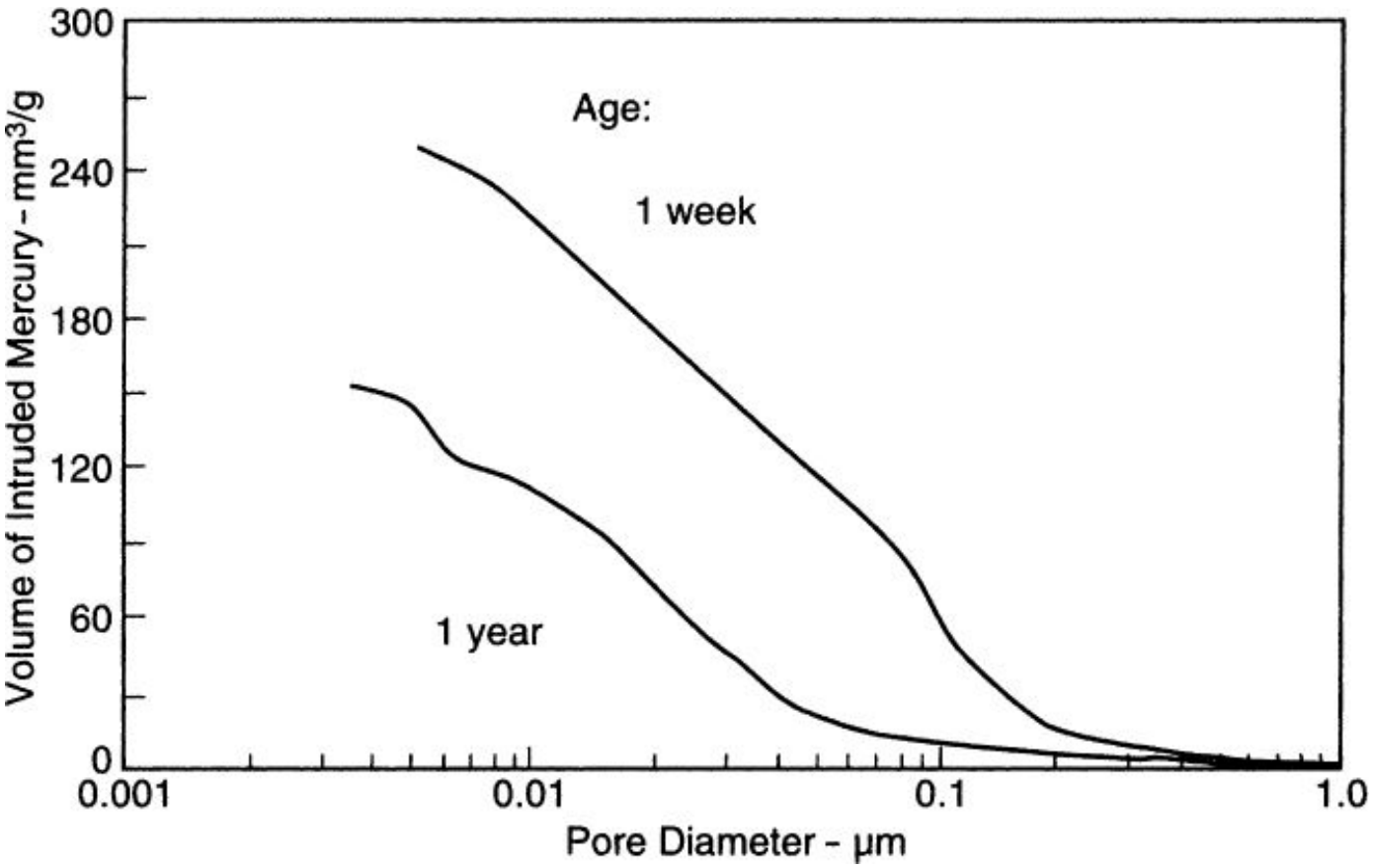


Fig. 13.1. Change in pore size distribution (determined by mercury porosimetry) in cement paste containing 30 per cent of Class F fly ash by mass of total cementitious material (based on ref. 13.15)

The sensitivity of the fly ash reaction to the alkalinity of the pore water means that the react-

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ivity of fly ash is influenced by the alkali content of the Portland cement with which the fly ash is to be used. (This is, however, disproved by Osbæck.[13.114]) For example, because rapid-hardening Portland (Type III) cement leads to a more rapid development of alkalinity of pore water than ordinary Portland cement, the pozzolanic reaction of fly ash starts earlier when Type III cement is used. The preceding observations illustrate the complexity of the behaviour of fly ashes which makes generalizations difficult and points to the need for tests involving both the fly ash and the Portland cement which are to be used together.

A consequence of the delay in the reactions of fly ash is the beneficial pattern of heat evolution by hydration (see Chapter 8).

Further progress of the pozzolanic reaction of Class F fly ash is slow: the presence of as much as 50 per cent of unreacted fly ash after one year is quoted by Fraay *et al*.[13.15]

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Whereas Portland-cement-only concrete with a medium or a high water/cement ratio, under suitable storage conditions, continues to gain strength over a long period, this is not so when fly ash is incorporated in the mix. No further strength development beyond the age of 3 to 5 years was found in concretes with water/cement ratios of 0.5 to 0.8; the Class F fly ash content, expressed as a percentage of the mass of the total cementitious material, ranged from 47 to 67.[13.16,13.17]

Class C fly ash (BSEN Class W) (see p. 85) which has a high lime content, reacts, to some extent, direct with water; in particular, some C_2S may be present in the fly ash[13.157] and this compound reacts to form C-S-H. Also, crystalline C_3A and other aluminates are reactive.[13.9] In addition, as with Class F fly ash, there is a reaction of silica with calcium hydroxide produced by the hydration of Portland cement. Thus, Class C fly ash reacts earlier than Class F fly ash, but some Class C fly ashes do not show a long-term increase in strength.[13.18]

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Because the reactions of fly ash in concrete take a long time, prolonged wet curing is essential. A consequence of this is that tests on compression specimens cured under standard wet conditions may be misleading with respect to the strength of concrete in situ. This, of course, is also the case with Portland-cement-only concrete, but the influence of curing on strength is more pronounced when fly ash is included in the mix.

Higher temperature, between 20 and 80 °C (68 and 176 °F) accelerates the reactions of fly ash to a greater extent than is the case with Portland cement alone. However, the usual retrogression of strength follows (cf. p. 361).[13.21] The reduction in strength with an increase in temperature between 200 and 800 °C is also similar to, or possibly even greater than, that in concrete made with Portland cement only.[13.20]

Because the reactivity of fly ash sharply increases with an increase in temperature, the behaviour of concrete containing fly ash may be dif-

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ferent in massive sections (where hydration of the Portland cement component raises the temperature) from the behaviour in small concrete elements at room temperature.[13.9] This observation is relevant to any prediction of the rate of gain of strength of concrete containing fly ash.

Strength development of fly ash concrete

The test method of ASTM C 311-07 provides for the measurement of strength of mortars containing fly ash representing 20 per cent by mass of the total cementitious material and establishes a strength activity index. However, as already discussed, the reactions of fly ash are affected by the properties of Portland cement with which it is used. Moreover, in addition to the effect of chemical reactions, fly ash has a physical effect of improving the microstructure of the hydrated cement paste. The main physical action is that of packing of the fly ash particles at the interface of coarse aggregate particles, which are ab-

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sent in the mortar used in the test of ASTM C 311-07.[13.12]

For these reasons, strength activity measurements do not adequately establish the contribution of fly ash to the development of strength of a particular *concrete* in which the fly ash is to be incorporated. This is an example of the inappropriateness of tests on mortar for the purpose of establishing the effect of a given factor on concrete.

The extent of packing depends both on the fly ash and on the cement used: better packing is achieved with coarser Portland cement and with finer fly ash.[13.12] One beneficial effect of packing on strength is a reduction in the volume of entrapped air in the concrete,[13.12] but the main contribution of packing lies in a reduction in the volume of large capillary pores.

It is worth noting that the positive influence of the fineness of fly ash is coupled with its spherical shape. Therefore, grinding of fly ash, although it increases fineness, may result in the destruction

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of spherical particles, with a consequent increase in water demand of the mix due to the irregular angular shape of the fly ash particles.[13.26]

Control of particle size of fly ash is usually effected on the basis of residue larger than 45 μm (No. 325 ASTM) sieve, but this is not sufficiently discriminatory with respect to the reactivity of fly ash and its contribution to strength development in concrete.

Typically, about one-half of the particles in fly ash are smaller than 10 μm, but there may be wide variations. It is particles of that size that are most reactive.[13.22] The reactivity is very high when the median diameter of fly ash particles is smaller still: 5 or even 2.5 μm.

As far as the coarse particles of fly ash are concerned, Idorn and Thaulow[13.23] suggested that these particles can be considered as 'microaggregate' which improves the density of the hydrated cement paste in a manner similar to the effect of unhydrated remnants of Portland cement

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particles. This is beneficial with respect to strength, resistance to crack propagation, and stiffness. The resulting system of capillary pores is better able to retain water which can be available for long-term hydration.[13.23]

The glass content of the fly ash strongly affects its reactivity. In the case of Class C fly ash, the lime content is also a factor influencing reactivity. However, knowledge of these characteristics does not make it possible to predict the performance of any given fly ash, and tests are necessary; tests with the actual Portland cement to be used are preferable.

It was mentioned on p. 656 that the beneficial influence of fly ash upon water demand does not extend beyond a fly ash content of 20 per cent by mass. An excessive content of fly ash is not beneficial from the point of view of strength development either. The limiting content is probably around 30 per cent by mass of total cementitious material, as can be seen from Fig. 13.2.[13.19]

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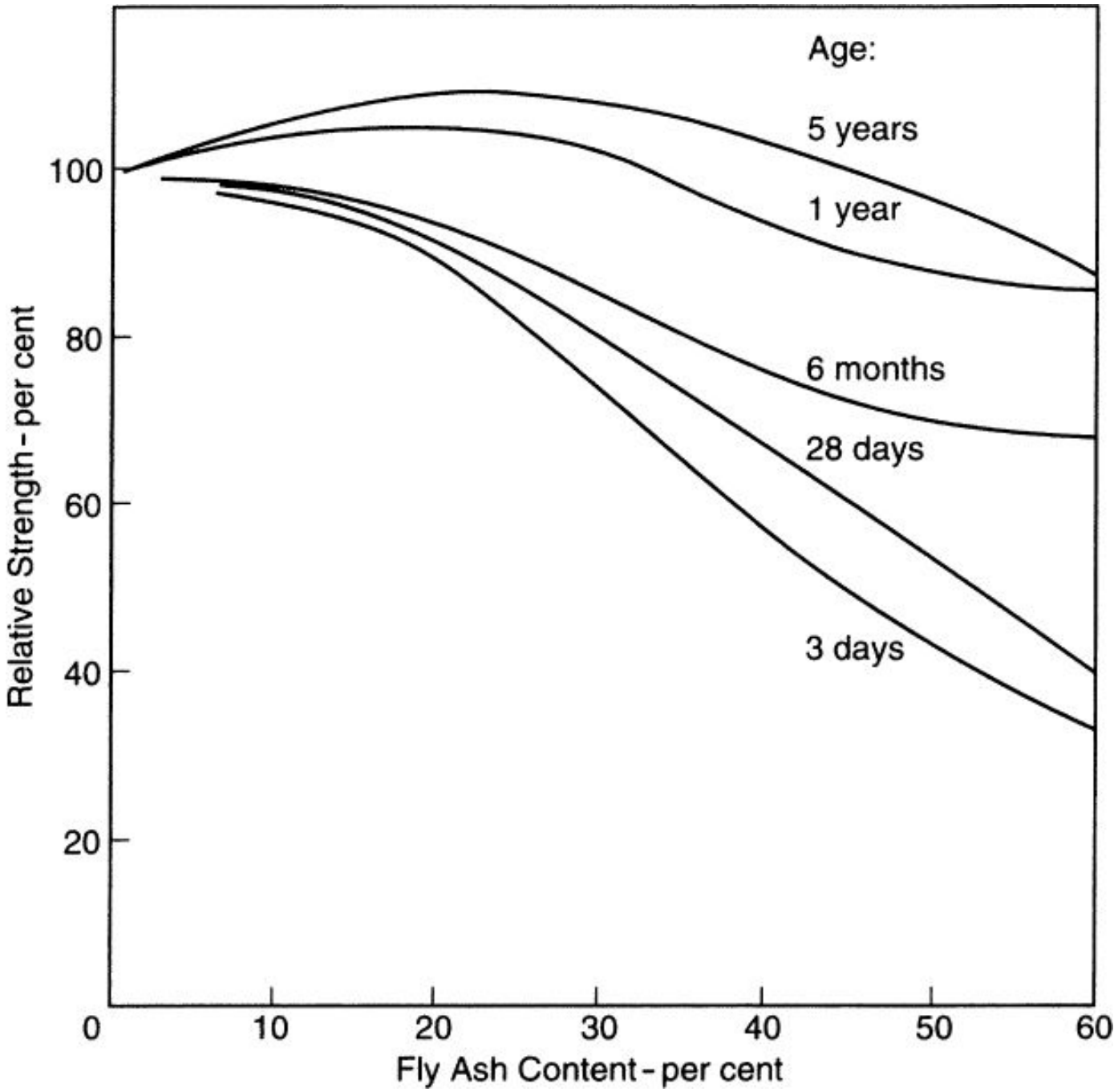


Fig. 13.2. Influence of content of fly ash in the cementitious material (by mass) on strength of hardened cement paste[13.19]

As has been repeatedly stated, quantified predictions of the influence of fly ash on strength are

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not possible. For example, the data of Fig. 13.2 can be contrasted with the apparent lack of a positive influence of fly ash upon strength even as late as one year, which was reported by the Portland Cement Association.[13.14]

Average values of strength of concrete cylinders moist cured at 23 °C (73 °F) (obtained from tests on six Class F fly ashes and four Class C fly ashes) are shown in Table 13.1.[13.14] All the mixes had a total cementitious material content of 307 kg/m^3 (517 lb/yd^3) with a 25 per cent content of fly ash by mass of total cementitious material. The water/cement ratio was 0.40 to 0.45, and the mixes had a slump of 75 mm. The same table gives the strength of a Portland-cement-only concrete with the same cement content and the same water/cement ratio. It is worth adding that the maximum size of aggregate was 9.5 mm ($\frac{3}{8}$ in.) so that the beneficial effect of fly ash with respect to packing around the coarse aggregate particles was smaller than would be the case with con-

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ventional concrete; therein may lie the explanation of the apparently limited effect of fly ash on strength.

Table 13.1. Typical Compressive Strength of Fly Ash Concretes[13.14]

Cementitious material	*Compressive strength, MPa (psi) at age (days):*						
	1	*3*	*7*	*14*	*28*	*91*	*365*
Portland cement	12.1 (1750)	21.2 (3070)	28.6 (4150)	33.9 (4910)	40.1 (5810)	46.0 (6670)	51.2 (7420)
Class F fly ash* (25%)	7.1 (1030)	13.9 (2010)	19.4 (2820)	24.3 (3520)	30.3 (4400)	39.8 (5770)	47.3 (6860)
Class C fly ash** (25%)	8.9 (1290)	19.0 (2760)	24.1 (3490)	28.5 (4140)	29.4 (4260)	40.5 (5880)	45.6 (6620)

*BS Class V
**BSEN Class W

In this connection, it should be noted that, because the specific gravity of fly ash is much lower than that of Portland cement (typically 2.35 as compared with 3.15), for the same mass, the volume of fly ash is about 30 per cent higher than that of cement. This must be taken into account in determining the mix proportions of concrete: usually, a lower content of fine aggregate is used than with Portland-cement-only concrete.

As for physical properties of concrete other than strength, it appears that creep and shrinkage

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are not fundamentally affected by the use of fly ash.

Durability of fly ash concrete

As discussed in Chapters 10 and 11, the selection of ingredients of a concrete mix must include consideration of their effect on durability. As in the case of strength, much depends on the actual fly ash used.

One consequence of the slow reaction of fly ash in the concrete is that, initially, the concrete has a higher permeability than concrete with a similar water/cement ratio (on the basis of the total cementitious material) but containing Portland cement only. However, with time, fly ash concrete acquires a very low permeability.[13.15] It is, nevertheless, essential that the concrete containing fly ash undergoes prolonged curing. The detrimental effect of inadequate curing on the absorption properties of the outer zone of concrete is greater the higher the fly ash content.[13.101] This effect is even more pronounced than the effect on

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the strength of concrete containing fly ash. Thus, reliance on strength alone may not be adequate for the purpose of assessing the durability of fly ash concrete in cases where penetration of concrete by aggressive agents is critical.

With respect to the resistance to sulfate attack, it should be noted that alumina and lime in the fly ash may contribute to the sulfate reactions. Specifically, when present in the glass part of the fly ash, alumina and lime provide a long-term source of material which can react with sulfates to form expansive ettringite.[13.25] A high silica/alumina ratio probably reduces the vulnerability to sulfate attack[13.28] but no reliable generalization is possible.

It seems that inclusion of Class F fly ash in concrete improves its sulfate resistance, probably mainly through the removal of calcium hydroxide. The content of fly ash should generally be between 25 and 40 per cent of the total cementitious material. Reliable information on the behaviour of Class C fly ash is not available. Indeed,

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the role of Class C fly ash with respect to sulfate resistance is not clear.[13.18]

Tests on air-entrained concrete with a water/cement ratio of 0.33 and a Class F fly ash content of 58 per cent by mass of cementitious material have shown an excellent resistance to freezing and thawing.[13.30] It should be noted that, for concrete exposed to de-icing agents, ACI 318-02[13.116] limits the mass content of fly ash and other pozzolanas to 25 per cent, in quantities up to 20 per cent of the total mass of cementitious material, this fly ash has no adverse effect on the resistance to freezing and thawing of air-entrained concrete. At high contents of Class C fly ash, the resistance was found to be impaired, possibly due to an increase in the porosity of the hardened cement paste caused by the movement of fibrous ettringite into the air voids.[13.1]

With respect to air entrainment of fly ash concrete, the problems caused by carbon, discussed on p. 551, should be borne in mind.

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Bilodeau *et al.*[13.124] found that fly ash, both Class F and Class C, at least when present in large proportions, results in concrete with a poor resistance to de-icing agents, even though the concrete has a good resistance to freezing and thawing. The reasons for this have not been established.

Because of the reduced permeability of mature concrete containing fly ash, the chloride ingress into such concrete is reduced. Even when the content of Class F fly ash is as high as 60 per cent by mass of cementitious material, the passivation of steel embedded in mortar and the risk of corrosion were found to be unimpaired.[13.24] This was confirmed by other tests on concretes with high fly ash contents (58 per cent of the total cementitious material) and water/cement ratios between 0.27 and 0.39, which have shown a very good resistance to chloride penetration.[13.24]

Nevertheless, in some countries[13.12] the use of fly ash in prestressed concrete is not permitted, it being thought that carbon in the fly ash may

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contribute to stress corrosion of the prestressing steel.

The abrasion resistance of concrete containing fly ash, Class F or Class C, is unimpaired[13.29] or possibly even improved.[13.31]

Fly ash, in adequate quantity in the mix, is beneficial in reducing the alkali–silica reaction (see p. 523) but the mechanisms involved are complex and imperfectly understood. The beneficial effects may arise from the denser structure of the hydrated cement paste which impedes the movement of ions, or from the preferential reaction of the alkalis with the fly ash so that they are not available for reaction with the silica in the aggregate.[13.28] It should be pointed out that fly ash itself contains alkalis, but typically only about one-sixth of the total alkali content in the fly ash is water-soluble, and therefore potentially reactive, the remainder being combined. Whether or not the fly ash contributes alkalis to the pore wa-

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ter in concrete seems to depend on the alkalinity of the cement used.[13.27]

There is no beneficial effect of fly ash with respect to the alkali–carbonate reaction.

Concretes containing ground granulated blastfurnace slag (ggbs)

Portland blastfurnace slag cement (see Chapter 2) has been used for more than a century, although in recent years there has been an increasing use of mixing the Portland cement and the ground granulated blastfurnace slag (ggbs) components direct in the concrete mixer. An advantage of this procedure is that the proportion of Portland cement and ggbs can be varied at will; a concomitant disadvantage is that an additional silo is required.

Because slag is produced at the same time as pig iron, the production control ensures a low variability of both materials. The slag is subsequently granulated or pelletized; for convenience, the term ‘granulated’ is generally used. The gran-

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ulated slag can be ground to a fineness of any desired value, but usually greater than 350 m^2/kg, that is, finer than Portland cement. Increased fineness leads to increased activity at early ages, and occasionally ggbs with a fineness in excess of 500 m^2/kg is used.[13.34]

There are several possible beneficial effects of incorporating ggbs in the mix. These are: the fresh concrete has an improved workability; the heat development is slower so that the peak temperature is lower; a denser microstructure of hydrated cement paste is achieved and this improves long-term strength and, especially, durability; and the risk of alkali–silica reaction can be eliminated, regardless of the alkali content of the Portland cement or the reactivity of the aggregate.[13.69]

The choice of the fineness of ggbs and of its content in the total cementitious material depend on the purpose of the use of ggbs in the concrete.

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Influence of ggbs on properties of fresh concrete

The presence of ggbs in the mix improves workability and makes the mix more mobile but cohesive. This is the consequence of a better dispersion of the cementitious particles and of the surface characteristics of the ggbs particles, which are smooth and absorb little water during mixing.[13.32] However, the workability of concrete containing ggbs is more sensitive to variations in the water content of the mix than is the case with Portland-cement-only concrete. When ground to a high fineness, ggbs reduces bleeding of concrete.

Mixes containing ggbs have been sometimes found to exhibit an early loss of slump, but there are also reports of a low rate of slump loss.[13.32]

The presence of ggbs in the mix leads to retardation at normal temperatures, typically 30 to 60 minutes.[13.32]

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Hydration and strength development of concrete containing ggbs

Because a blend of Portland cement and ggbs contains more silica and less lime than Portland cement alone, hydration of the blended cement produces more C-S-H and less lime than Portland cement alone. The resulting microstructure of the hydrated cement paste is dense. However, the initial hydration of ggbs is very slow because it depends upon the breakdown of the glass by the hydroxyl ions released during the hydration of the Portland cement. In a manner similar to blended cements containing pozzolanas, reaction of ggbs with calcium hydroxide takes place.

The progressive release of alkalis by the ggbs, together with the formation of calcium hydroxide by Portland cement, results in a continuing reaction of ggbs over a long period. Thus, there is a long-term gain in strength[13.132] (see Fig. 13.3). As an example, Roy[13.9] quoted that 8 to 16 per cent of ggbs has hydrated at 32 days, and 30 to 37 per cent at 28 days. However, the later rate of

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hydration of blended cement containing ggbs is accelerated. Thus, overall, the peak temperature of concrete caused by hydration of cement is reduced by the inclusion of ggbs in the mix.

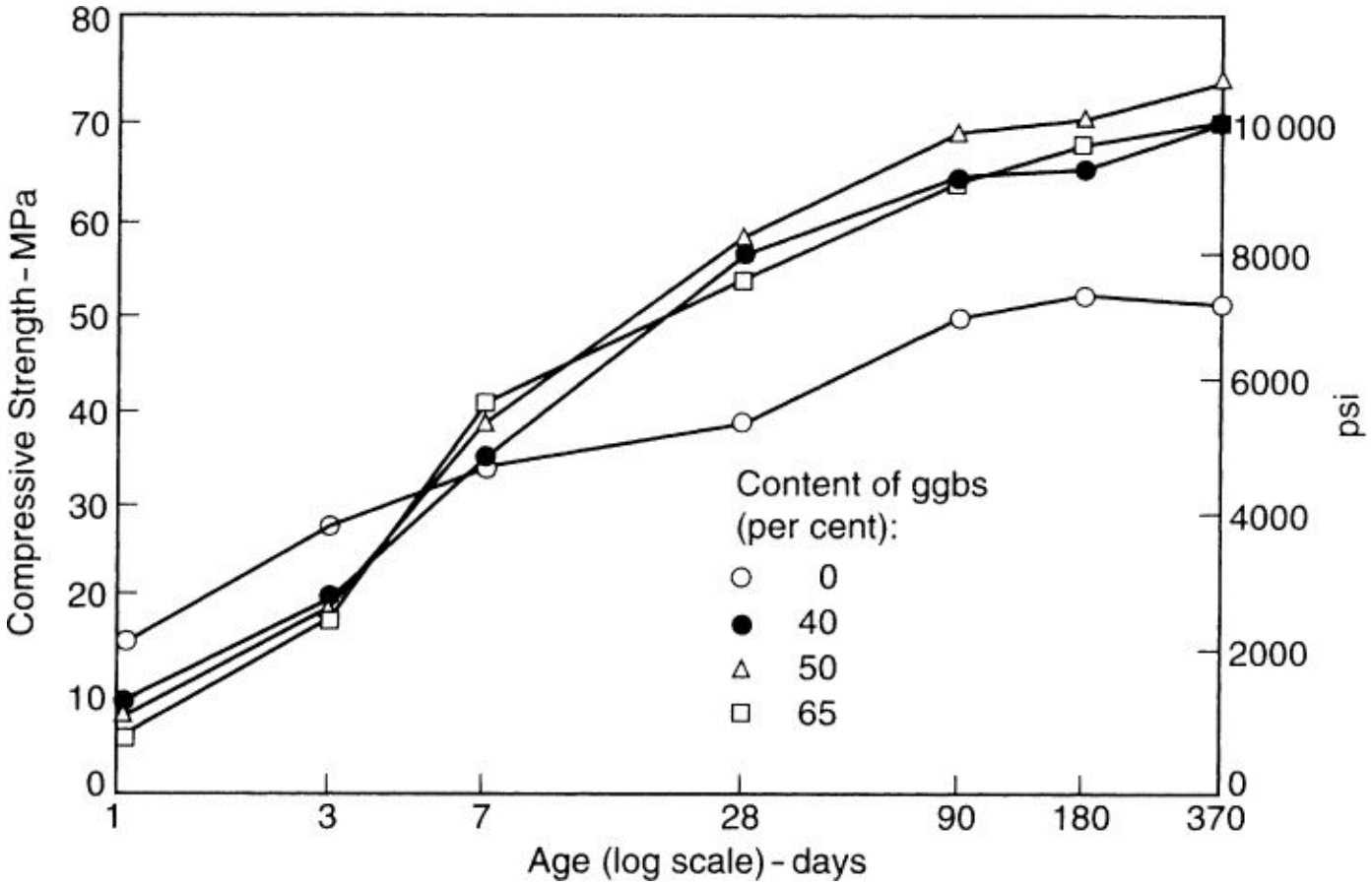


Fig. 13.3. Development of compressive strength of concrete (measured on cubes) moist cured at room temperature for various contents of ggbs by mass of total cementitious material[13.132] (Copyright ASTM–reproduced with permission)

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The solubility of alkali hydroxides increases with an increase in temperature. Consequently, the reactivity of ggbs at higher temperatures is considerably increased. Steam curing of concrete containing ggbs can, therefore, be used.[13.123] Moreover, the harmful effects of high early temperature upon long-term strength and permeability are less pronounced in concrete containing ggbs than in Portland-cement-only concrete.[13.2,13.33] Conversely, at temperatures below about 10 °C (50 °F) the strength development is poor[13.42] and the use of ggbs is undesirable.

Greater fineness of ggbs leads to a better strength development, but only at later ages, because activation of ggbs must first take place. A greater fineness of Portland cement speeds up the activation.

Other factors influencing the reactivity of ggbs are the chemical composition of the slag (see p. 80), and the glass content. However, attempts to relate the reactivity of the slag to its chemical composition by a single ‘chemical mod-

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ulus' or 'hydraulic index' have not proved successful. While a high glass content is essential, a few per cent of crystalline material may be beneficial with respect to the reactivity of ggbs because these crystals act as nuclei for hydration.[13.125] An important factor is the concentration of the alkalis in the *total* cementitious material; thus, the properties of the Portland cement used with a given ggbs are a factor. Generally, a better development of strength is found with finer cements and with cements that have high contents of C_3A and of the alkalis.[13.96]

The proportions of ggbs and Portland cement influence the development of strength of the resulting concrete. For the highest medium-term strength, the proportions are about 1:1, that is, a 50 per cent content of ggbs in the cementitious material;[13.123] the early strength is inevitably lower than with the same content of cementitious material consisting of Portland cement only. In many structures, however, the early strength is not important. An example of the development

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of strength of mortars containing varying proportions of ggbs is shown in Fig. 13.4 which suggests an optimum ggbs content of about 50 per cent from a strength standpoint.[13.36] Very good development of strength of concretes containing 50 to 75 per cent of ggbs, with a total content of cementitious material between 300 and 420 kg/m^3 (300 to 500 lb/yd^3) has been reported.[13.35]

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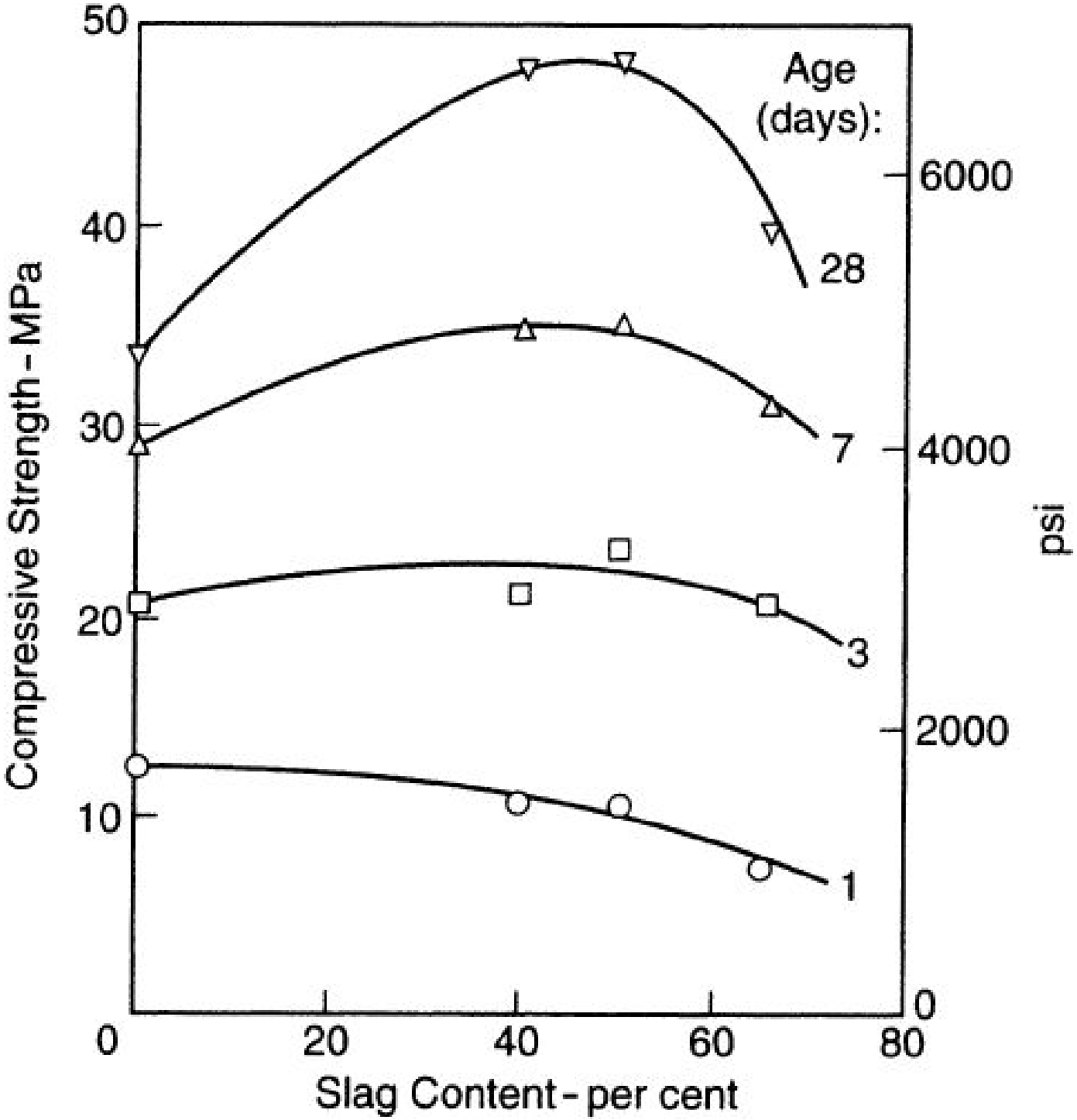


Fig. 13.4. Influence of content of ggbs in the total cementitious material (by mass) on the strength of mortar at various ages[13.36]

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Reference to the beneficial effects of higher temperatures upon the strength of concrete containing ggbs was made earlier in this section. In this connection, it should be noted that tests comparing the development of strength of concretes with and without ggbs, using specimens cured under standard conditions of temperature, may not give a correct picture. In actual structural members, the temperature is likely to rise in consequence of the initial hydration of Portland cement so that the strength development would be greater than in standard test specimens.[13.69]

Prolonged moist curing of concrete containing ggbs is particularly important because the initial low rate of hydration results in a system of capillary pores which allows the loss of water under drying conditions. If this happens, continuing hydration cannot take place. Japanese recommendations for curing may be of interest; these are shown in Table 13.2.

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Table 13.2. Japanese Recommendations for Moist Curing of Concrete Containing Different Percentages (by Mass of Total Cementitious Material) of Ground Granulated Blast-furnace Slag[13.42]

Air temperature, °C (°F)	*Minimum period of moist curing (days) at ggbs content of (per cent):*		
	30 to 40	*40 to 55*	*55 to 70*
≥17 (63)	5	6	7
10 to 17 (50 to 63)	7	8	9
5 to 10 (41 to 50)	9	10	11

The incorporation of ggbs in concrete does not alter significantly the usual relations between compressive strength and fiexural strength or between the compressive strength and the modulus of elasticity.[13.42] Occasional differences have been reported, but the assumption of any special relation has to be based on tests. Shrinkage of concrete containing ggbs is initially increased[13.123] but, overall, shrinkage and creep are not adversely affected by the use of ggbs.[13.42]

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A comment on the colour of concrete containing ggbs may be of interest. The ggbs itself is lighter in colour than Portland cement, and this is reflected in the colour of the resulting concrete, especially at high contents of ggbs. There is an additional effect: several days after placing, the concrete may acquire a bluish hue due to the reactions of iron sulfide in the slag. Upon subsequent oxidation of the sulfide, usually over a period of several weeks, the bluish tinge disappears. However, if the concrete is sealed early or remains wet, the oxidation may be prevented.[13.42]

Durability aspects of concrete containing ggbs

Tests on mortar containing ggbs have shown that its water permeability is reduced by a factor of up to 100.[13.43] There is also a very large reduction in diffusivity of mortar containing ggbs, especially with respect to chloride ions.[13.43]

Tests on concrete containing ggbs have confirmed good resistance to penetration by chloride ions.[13.35] Daube and Bakker[13.126] have shown

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that, when the ggbs content is at least 60 per cent by mass of the cementitious material and the water/cement ratio is 0.50, the diffusion coefficient of the concrete exposed to chloride ions is at least ten times smaller than when the cementitious material consists entirely of Portland cement.

The beneficial effects of ggbs arise from the denser microstructure of hydrated cement paste, more of the pore space being filled with C-S-H than in Portland-cement-only paste.

As a result of the improved microstructure of hydrated paste of a Portland cement–ggbs blend, and also because of a low content of calcium hydroxide, the resistance to sulfate attack is improved. Hooton and Emery[13.128] reported that blended cement containing 50 per cent by mass of ggbs (with 7 per cent of $A1_2O_3$) and Type I Portland cement (with a C_3A content of 12 per cent) exhibits the same sulfate resistance as sulfate-resisting (Type V) cement when tested in mortar. To be effective, the content of ggbs must be at least

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50 per cent by mass of the total cementitious material, and preferably 60 to 70 per cent.

The very low penetrability of concrete which contains ggbs is effective also in controlling the alkali–silica reaction: the mobility of the alkalis is greatly reduced. This effect is complemented by the incorporation of the alkalis in the products of reaction of ggbs, especially at higher temperatures.[13.36] The beneficial effects of ggbs when used in conjunction with siliceous aggregates suspected of alkali reactivity or with Portland cement with an alkali content up to 1.0 per cent are of great importance.

The situation with respect to freezing and thawing is different. Concrete of appropriate mix proportions containing ggbs has the same resistance to freezing and thawing as concrete made with Portland cement only. However, the inclusion of ggbs in air-entrained concrete has no beneficial effect.[13.32,13.123] In view of the beneficial influence of ggbs on the permeability of the resulting concrete, it is not clear why the inclusion

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of ggbs in concrete does not improve its resistance to freezing and thawing in a manner similar to the beneficial effects of a reduced water/cement ratio. In this connection, it is relevant to note that, for concrete exposed to de-icing agents, ACI 318-08[13.116] imposes a limit on the content of ggbs of 50 per cent of the total cementitious material. When both ggbs and fly ash are included in the mix, the mass of both these materials together is limited to 50 per cent of the total mass of cementitious material; the limitation on fly ash alone of 25 per cent (see p. 742) applies also when it is used with ggbs.

It should be pointed out that, in order to achieve the same resistance to freezing and thawing with ggbs in the mix as is offered by Portland-cement-only concrete, prolonged moist curing prior to exposure to freezing and thawing is essential.

A beneficial effect of the inclusion of ggbs in the mix upon the resistance of concrete to de-

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icing salt scaling has been reported by Virtanen,[13.37] but this has not been confirmed.

With respect to carbonation, the effects of ggbs are two-fold. Because of the small amount of calcium hydroxide present in the hydrated cement paste, carbon dioxide does not become fixed near the surface of the concrete so that there is no pore-blocking formation of calcium carbonate. Consequently, at early ages, the depth of carbonation is significantly greater than in concrete containing Portland cement only.[13.34] On the other hand, the low permeability of well-cured concrete containing ggbs prevents a continuing increase in the depth of carbonation.[13.37,13.43] For this reason, except when the ggbs content is very high, there is no increased risk of corrosion of steel reinforcement through a reduction in the alkalinity of the hydrated cement paste and depassivation of the steel.[13.32]

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Concrete containing silica fume

The physical properties of silica fume were described in Chapter 2. The use of this cementitious material continues to increase despite its relatively high cost. Silica fume is particularly valued in making high performance concrete, which will be discussed later in this chapter. In the present section, the general features of the use of silica fume in concrete will be considered. It can be noted that there exists no British Standard on silica fume and the ACI guide for the use of silica fume in concrete, ACI 234R-96[13.159] was first published in 1996.

The very high reactivity of silica fume with calcium hydroxide produced by the hydration of Portland cement was mentioned in Chapter 2. Because of this reactivity, it is possible to use silica fume as a replacement for a small proportion of Portland cement. This is done on the basis of 1 part of silica fume instead of 4 or even 5 parts of Portland cement by mass; a maximum of 3 to

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5 per cent of silica fume is used.[13.40] When using this approach for low- or medium-strength concretes, the strength is unaffected by the silica fume replacement. Because, in such concretes, the water/cement ratio is high or medium, the use of superplasticizers is not necessary. Additional benefits of replacement by silica fume are: reduced bleeding and improved cohesion of the mix. However, such use of silica fume is limited to some geographic areas with an abundant local supply of silica fume which can be used in the low bulk-density form (see p. 87).

By far the largest use of silica fume is for the purpose of producing concrete with enhanced properties, mainly high early strength or low penetrability. The beneficial effects of silica fume are not limited to its pozzolanic reaction: there is also a physical effect of the ability of the extremely fine particles of silica fume to be located in very close proximity to the aggregate particles, that is, at the aggregate–cement paste interface. This zone is known to be a source of weakness in

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concrete, the reason being the wall effect which prevents the particles of Portland cement from packing tightly against the surface of the aggregate. Such packing is achieved by the particles of silica fume, which are, typically, 100 times smaller than cement particles. A contributing factor is the fact that silica fume, because of its high fineness, reduces bleeding so that no bleed water is trapped beneath coarse aggregate particles. In consequence, the porosity in the interface zone is reduced, compared with a mix not containing silica fume. Subsequent chemical reaction of silica fume results in a still lower porosity in the interface zone which, in consequence, is no longer particularly weak, either in terms of strength or of permeability.

The above argument explains why too low a content of silica fume, say below 5 per cent of the total mass of cementitious material, does not lead to a high strength of concrete: the volume of silica fume is inadequate to cover the surface of all coarse aggregate particles. It is also evident

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that a large volume of silica fume is only marginally more beneficial than about 10 per cent because the excess silica fume cannot be located at the surface of the aggregate. It is useful to point out that the beneficial effect of changes in the hardened cement paste in the interface zone cannot exist in neat cement paste because, in the absence of aggregate, there is no interface zone; this was confirmed by Scrivener *et al*.[13.5]

Influence of silica fume on properties of fresh concrete

It is essential that silica fume be thoroughly and uniformly dispersed in the mix. For this reason, the mixing time should be extended, and especially so with silica fume in the densified form of micropellets. The sequence of feeding materials into the mixer is of importance and is best established by trial and error.

The very large surface area of the particles of silica fume, which have to be wetted, increases the water demand, so that, in mixes with a low

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water/cement ratio, it is necessary to use a superplasticizer. In this way, it is possible to maintain both the required water/cement ratio and the necessary workability.

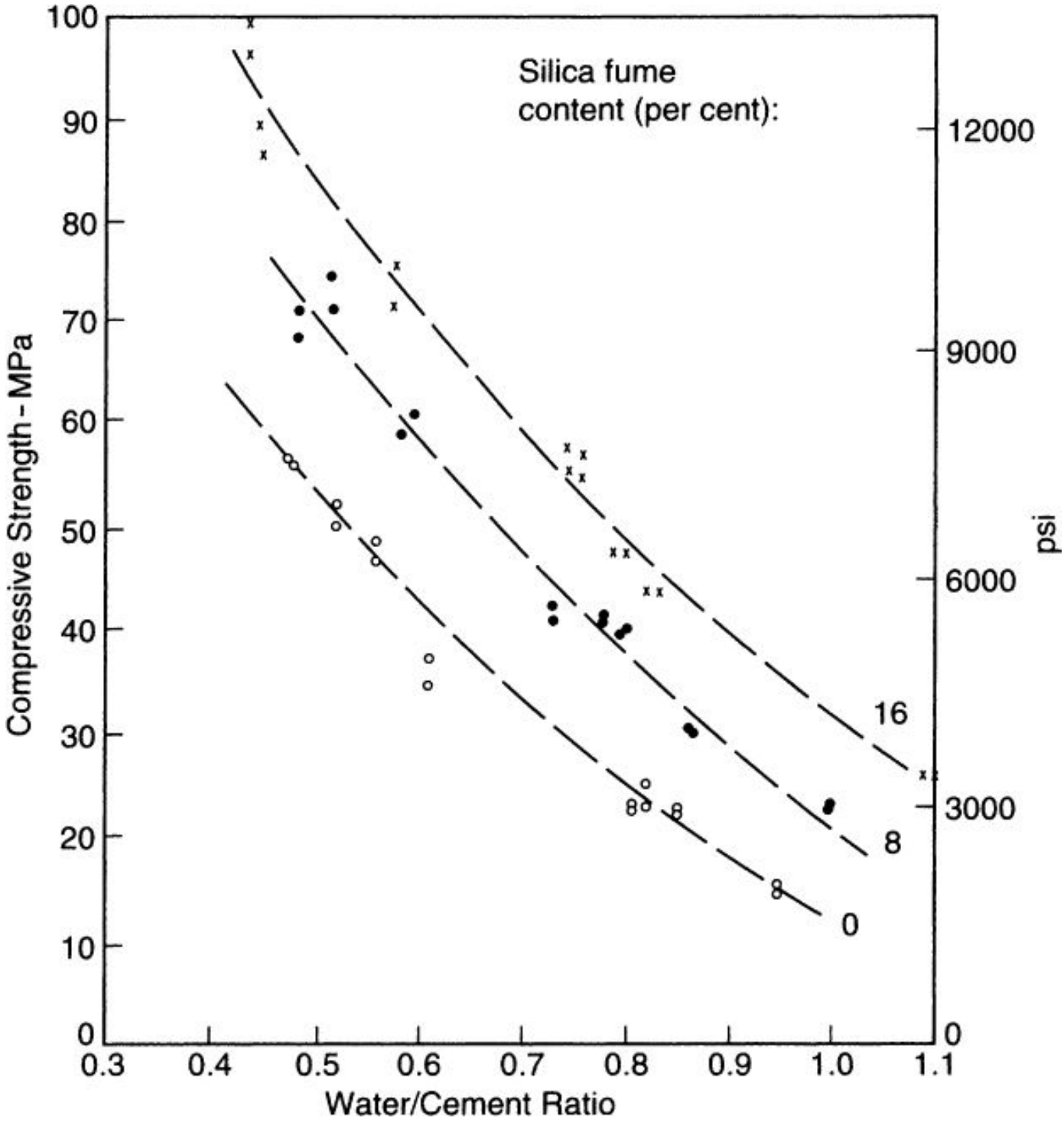
The effectiveness of superplasticizers is enhanced by the presence of silica fume. For instance, in mixes with a slump of 120 mm, a given dosage of a superplasticizer was found to reduce the water demand by 10 kg/m^3 in a Portland-cement-only concrete. The same dosage maintained the slump when the silica fume content was 10 per cent by mass of cementitious material. Without the superplasticizer, the water demand due to the inclusion of silica fume in the mix[13.122] would have risen by 40 kg/m^3. It can, therefore, be seen that the use of both silica fume and a suitable superplasticizer is beneficial and makes it possible to use low water/cement ratios at a given workability.[13.39] The lower water/cement ratio results in an increase in strength which is larger than would be expected solely from the pozzolanic action of silica fume. However, in relative

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terms, the effect of the lower water/cement ratio upon strength is smaller than the overall direct effect of silica fume.[13.5]

At this stage, it may be useful to note that the pattern of the relation between compressive strength and the water/cementitious material ratio is the same for concretes with and without silica fume but, at the same ratio, concrete with silica fume has a higher strength. Examples of the relation between the 28-day compressive strength of 100 mm cubes and the water/cementitious material ratio for concretes with 8 and 16 per cent of silica fume by mass of the total cementitious material, are shown in Fig. 13.5. The same figure also shows the relation for concrete containing Portland cement only.[13.62]

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Fig. 13.5. Relation between the compressive strength (measured on 100 mm cubes) and the water/cement ratio for concretes with different contents of silica fume (by mass of total cementitious material)[13.62]

The presence of silica fume affects significantly the properties of fresh concrete. The mix is strongly cohesive and, in consequence, there is very little bleeding, or even none. The reduced bleeding can lead to plastic shrinkage cracking under drying conditions, unless preventive measures are taken. On the other hand, voids caused by trapped bleed water are absent.

The cohesive character of the mix affects the slump so that, for both mixes equally to be capable of compaction, the mix with silica fume needs a slump 25 to 50 mm (1 to 2 in.) higher than a mix containing Portland cement only.[13.55,13.57] Mixes with a very high content of cementitious material tend to be ‘sticky’ and do not easily allow the slump cone to be lifted. It

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has, therefore, been suggested that the slump test is inappropriate and that the flow test is preferable.[13.38] The 'sticky' nature should not be misinterpreted: as soon as vibration is applied, the mix becomes 'mobile'. However, in order to avoid an excessively sticky mix, it is recommended[13.99] that the water content should not be lower than 150 kg/m^3 (250 lb/yd^3) when the fine aggregate is angular in shape, or not lower than 130 kg/m^3 (220 lb/yd^3) when rounded fine aggregate is used.

The cohesiveness of concrete containing silica fume makes it satisfactory for pumping and for underwater concreting, as well as for use as flowing concrete[13.55] (see p. 259). Entrained air remains stable,[13.57] but an increased dosage of the air-entraining admixture is required because of the high fineness of silica fume. In addition, there are problems in obtaining a suitable air-void system when superplasticizers are used (which is usually the case with silica fume mixes).

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There are no reports of incompatibility of silica fume with admixtures in general. It is useful to observe that the retarding effect of lignosulfonate-based admixtures is smaller when silica fume is present in the mix. Consequently, larger dosages of these admixtures can be used without causing an excessive retardation.[13.55]

Hydration and strength development of the Portland cement–silica fume system

In addition to the pozzolanic reaction between the amorphous silica in silica fume and calcium hydroxide produced by the hydration of Portland cement, silica fume contributes to the progress of hydration of the latter material. This contribution arises from the extreme fineness of the silica fume particles which provide nucleation sites for calcium hydroxide. Thus, early strength development takes place.

Silica fume dissolves in a saturated solution of calcium hydroxide within a few minutes.[13.9] Therefore, as soon as enough Portland cement

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has hydrated to result in saturation of the pore water with calcium hydroxide, calcium silicate hydrate is formed on the surface of the silica fume particles. This reaction proceeds, initially, at a high rate. For example, when the mass of silica fume was 10 per cent of the total mass of cementitious material, one-half of the silica fume was observed to react in 1 day, and two-thirds during the first 3 days. However, subsequent reaction was very slow, only three-quarters of the silica having hydrated at 90 days.[13.8]

The acceleration of hydration processes by silica fume occurs also when, in addition to Portland cement, ggbs is present in the mix.[13.46]

A consequence of the rapidity of the early reactions in concretes containing silica fume is that the development of heat of hydration in such concretes may be as high as when rapid-hardening Portland (Type III) cement is used alone.[13.9]

The behaviour of concrete with silica fume beyond the age of about 3 months depends on the

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moisture conditions under which the concrete is stored. Up to the age of $3\frac{1}{2}$ years, tests showed a small increase in compressive strength of wet-stored concretes with 10 per cent of silica fume (by mass of total cementitious material) and water/cement ratios of 0.25, 0.30, and 0.40.[13.58] Under dry storage conditions, retrogression of strength, typically up to 12 per cent below the peak value at about 3 months, was observed in tests on laboratory specimens.[13.58] However, the strength of concrete containing silica fume, determined on cores up to 10 years old, clearly shows no retrogression of strength.[13.47] This finding is of importance because the behaviour of test specimens in which moisture gradients exist may be misleading.[13.56]

The C-S-H produced by silica fume has a lower C:S ratio than the C-S-H resulting from the hydration of Portland cement alone. Values of the C:S ratio in the products of hydration of silica fume have been found[13.9] to be as low as 1. The

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C:S ratio is lower at high contents of silica fume in the cementitious material.[13.41]

One consequence of the high early reactivity of silica fume is that the mix water is rapidly used up; in other words, self-desiccation takes place.[13.49] At the same time, the dense microstructure of the hydrated cement paste makes it difficult for water from outside, if available, to penetrate toward the unhydrated remnants of Portland cement or silica fume particles. In consequence, strength development ceases much earlier than with Portland cement alone; some experimental data are shown in Table 13.3[13.49] from which it can be seen that there was no increase in strength beyond 56 days. The data of Table 13.3 refer to mixes with a total content of cementitious material of 400 kg/m^3, sulfate-resisting Portland (Type V), cement, silica fume contents of 10, 15, and 20 per cent by mass of total cementitious material, and a water/cement ratio of 0.36; the concrete specimens were maintained under moist conditions.

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Table 13.3. Strength Development of Test Cylinders of Concretes Containing Silica Fume[13.49]

Age	Compressive strength (MPa) of mixes with a silica fume content of (per cent):			
	0	10	15	20
1 day	26	25	28	27
7 days	45	60	63	65
28 days	56	71	75	74
56 days	64	74	76	73
91 days	63	78	73	74
182 days	73	73	71	78
1 year	79	77	70	80
2 years	86	82	71	82
3 years	88	90	85	88
5 years	86	80	67	70

The contribution of silica fume to the early strength development (up to about 7 days) is probably through improvement in packing, that is, action as a filler and improvement of the interface zone with the aggregate.[13.45] The bond of the hydrated cement paste with aggregate, especially the larger particles, is greatly improved,[13.50] allowing the aggregate better to participate in stress transfer. Some contrary arguments about the role of silica fume have been advanced,[13.44] but they

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are likely to reflect specific test conditions rather than intrinsic behaviour.

The contribution of a given amount of silica fume to the strength of concrete arising from packing and interface effects should remain constant with time. This is unlike the effect of pozzolanic activity which continues to take place. Indeed, at a fixed content of silica fume, the increase in strength of concrete between 7 and 28 days was found to be independent of the value of the 7-day strength.[13.59] The contribution of silica fume to strength at, say, 28 days should, however, increase with an increase in the content of silica fume in the mix (up to a certain limit). This was found to be the case for concretes with 28-day strengths between approximately 20 and 80 MPa (3000 to 12 000 psi): the increase in strength was 7 MPa for a 10 per cent content, and 16 MPa for a 20 per cent content of silica fume.[13.59]

The relation between the content of silica fume in the mix and the resulting strength, just mentioned, has encouraged numerous attempts to

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devise a so-called efficiency factor of silica fume for strength. Other efficiency factors have been derived for other properties of concrete made with silica fume, such as permeability.[13.55] The various factors differ from one another. For this reason, and also because the effects of silica fume are influenced by the properties of the Portland cement used, the 'efficiency factor' approach is not considered to have sufficient validity.

The continuing pozzolanic activity of silica fume results in a reduction in the pore size in the hydrated cement paste. Test data demonstrating the existence of very small pores in hydrated paste of blended sulfate-resisting cement (Type V) and silica fume are shown in Table 13.4; mercury intrusion porosimetry was used. The same table shows that the reduction in total porosity of hydrated cement pastes containing silica fume is small, as compared with a paste made with a sulfate-resisting (Type V) cement only.[13.49] It can be seen, thus, that the main effect of silica fume is to reduce the permeability of the hydrated ce-

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ment paste, but not necessarily its total porosity. Whereas the presence of silica fume representing 10 per cent by mass of the total cementitious material has a large effect on the pore system, increasing further content of silica fume in the cementitious material leads only to a small change. This accords with the earlier observations (see p. 669) to the effect that there is no beneficial effect of the presence of particles of silica fume in excess of those required to cover the surface of the aggregate and to infill the space between the particles of Portland cement.

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Table 13.4. Pore Characteristics of Mortars Containing Sulfate-Resisting Cement and Silica Fume[13.49]

Period of moist curing, days	Total porosity, per cent of mixes with a silica fume content of: (per cent)			
	0	10	15	20
7	16.0	14.3	13.7	13.0
28	14.7	13.4	12.9	11.7
91	14.3	13.3	11.7	10.6
182	10.8	10.8	9.6	8.6
365	10.7	9.5	10.5	9.1
	Volume of pores with a diameter larger than 0.05 μm, per cent			
7	8.5	3.0	2.7	2.0
28	6.3	2.8	2.2	2.3
91	7.5	2.8	1.8	1.7
182	5.3	3.2	2.4	2.3
365	5.1	2.1	2.5	2.0

As is the case with all pozzolanic reactions, prolonged moist curing of concrete containing silica fume is necessary, especially because of its strength contribution between the ages of 3 and 28 days.[13.55] Surprisingly, tests on mortar containing silica fume have shown that the beneficial effects of a prolonged moist curing on flexural strength are much smaller than on the compressive strength.[13.89] Confirmation, or otherwise, of

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similar behaviour in concrete is not available. Differences in curing apart, the relation between the tensile or flexural strength and compressive strength is not affected by the presence of silica fume in the concrete.[13.55,13.99]

The modulus of elasticity of concrete containing silica fume is somewhat higher than is the case with Portland-cement-only concretes of similar strength.[13.55] It has been reported that concrete containing silica fume is more brittle, but this has not been confirmed.[13.55]

Durability of concrete containing silica fume

In the preceding section, we discussed the importance of adequate curing of concrete containing silica fume from the standpoint of reactions of hydration. As far as durability is concerned, we should note that a consequence of more advanced hydration is a reduced permeability; as already mentioned, adequate curing is of particular importance. Generally, for concretes of equal strength, the reduction in permeability due to a

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longer period of curing is greater in concrete containing silica fume than in Portland-cement-only concrete.[13.127]

The desirable minimum curing period depends, among other things, on temperature, which in the field may be subject to considerable variation. Low temperature slows down the hydration reactions involving silica fume even more than is the case in Portland-cement-only concretes.[13.55] However, upon a subsequent rise in temperature, the usual reactions take place[13.121] and the accelerating effect of a higher temperature is greater than in the case of Portland cement alone.[13.55] Also, the harmful effects of a higher temperature on pore structure are smaller in the presence of silica fume.[13.127]

It is important to note that carbonation is particularly adversely affected by inadequate curing.[13.55]

The influence of silica fume upon the permeability of concrete is greater than is indicated

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by tests on hydrated cement paste because, in the former case, silica fume reduces the permeability of the transition zone around the aggregate particles, as well as the permeability of the bulk paste.[13.57] The influence of silica fume upon permeability of concrete is very large: a 5 per cent content of silica fume was reported by Khayat and Aïtcin[13.57] to reduce the coefficient of permeability by 3 orders of magnitude. Thus, in relative terms, the influence of silica fume upon permeability is much larger than upon compressive strength.

A consequence of reduced permeability is a greater resistance to the ingress of chloride ions. Even using Portland cements with C_3A contents up to as much as 14 per cent, the presence of 5 to 10 per cent of silica fume in the total cementitious material greatly slows down the ingress of chloride ions into concrete.[13.48,13.138] ACI 318-08[13.116] limits the content of silica fume to 10 per cent when the concrete is to be exposed to de-icing agents. The reduction in the diffusivity of chlor-

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ides due to the presence of silica fume in hydrated cement paste is larger at water/cementitious material ratios greater than 0.4 than at extremely low values of the water/cementitious material ratio;[13.51] in the latter case, the hydrated cement paste has a very low diffusivity, even without silica fume.

The sulfate resistance of concrete containing silica fume is good, partly because of a lower permeability, and partly in consequence of a lower content of calcium hydroxide and of alumina, which have become incorporated in C-S-H. Tests on mortar have shown also the beneficial effect of silica fume upon resistance to solutions of magnesium, sodium, and calcium chlorides.[13.52] The role of pozzolanas in controlling expansive alkali–silica reaction was discussed on p. 520. Silica fume is particularly effective in this respect.[13.53] It can be added that a consequence of the lower C:S ratio of the products of reaction of silica fume is an increased ability of these

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products to incorporate ions such as the alkalis or aluminium.[13.55]

With respect to resistance to freezing and thawing, some investigators[13.61] reported a poor resistance of air-entrained concrete containing silica fume as compared with Portland-cement-only concrete. A possible explanation is that, with an adequate entrained-air content, the concrete containing silica fume had a larger air-void spacing factor and, at the same time, the dense structure of the hydrated cement paste prevented the movement of water. On the other hand, other investigators[13.60] found a good resistance of concretes containing silica fume to freezing and thawing, and also to scaling by de-icing agents. Experience with structures in situ gave variable results.[13.37]

Resolution of this conflict in reports on performance would require a detailed knowledge of test procedures used, including the maturity and moisture condition of the concretes at the time

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of the test. Indeed, the influence of silica fume upon resistance to freezing and thawing is complex. After a period of moist curing, the pore size in the hydrated cement paste becomes smaller (see p. 658); in consequence, the freezing point of pore water is reduced (see p. 539). In the interior of concrete, self-desiccation is likely to have reduced the water content below the critical level of saturation so that freezing would not cause damage. The fine pore system also makes it difficult for the concrete to become re-saturated after drying.[13.88] On the other hand, a dense paste with a very low permeability does not allow a rapid enough movement of water out of pores subjected to freezing and into an air void. Thus, rapid freezing would lead to damage.[13.57]

The preceding discussion shows that generalizations about the influence of silica fume on the resistance of concrete to freezing and thawing, and even more so to scaling by de-icing agents, are not possible: much depends on the particular concrete used, on its treatment prior to freezing

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and thawing, and on the rapidity of temperature changes. It is, therefore, not surprising that many publications present conflicting results, and there would be little value in reviewing them in this book. For practical purposes, the only conclusion which can be drawn is that it is necessary to test any concrete which it is proposed to use, and the test results have to be interpreted in the light of the expected conditions of exposure.

Because silica fume reduces the alkali content in the pore water, the pH of pore water becomes lowered. Tests on mature cement pastes made with Portland cement with a very high alkalinity (pH of 13.9) have shown a reduction in the value of pH caused by the inclusion in the mix of 10 per cent of silica fume to be 0.5; 20 per cent of silica fume reduced the value of pH by 1.0.[13.139] Even with the last-named reduction, the value of pH was 12.9. Havdahl and Justnes[13.129] confirmed that the pH stays above 12.5. Thus, the alkalinity is adequately high for the protection of reinforcing steel from corrosion.[13.55]

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The presence of silica fume in concrete has a beneficial influence upon resistance to abrasion because, in the absence of bleeding, no weak top layer is formed and also because of a better bond between the hydrated cement paste and the coarse aggregate; differential wear and loosening of particles do not, therefore, occur.[13.57]

Shrinkage of concrete containing silica fume is somewhat larger, typically 15 per cent, than in Portland-cement-only concrete.[13.49]

The darker colour of some silica fumes was mentioned on p. 85. This influences the colour of the resulting concrete. However, the colour becomes lighter after a few weeks, but the reasons for this are not clear.[13.55]

High performance concrete

High performance concrete is not a revolutionary material, nor does it contain ingredients which are not used in the concrete considered this far. Rather, high performance concrete is a develop-

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ment of the concretes discussed in the last few sections.

The very name ‘high performance concrete’ smacks of advertising an allegedly distinct product. A former name was ‘high strength concrete’ but, in many cases, it is a high durability that is the required property, although, in others, it is high strength, either very early, or at 28 days, or even later. In some applications, a high modulus of elasticity is the property sought.

With respect to strength, we should note that the meaning of the term ‘high strength’ has changed significantly over the years: at one time, 40 MPa (or 6000 psi) was considered high; later on, 60 MPa (or 9000 psi) became viewed as high strength concrete. In this book, high performance in terms of strength will be taken as a compressive strength in excess of 80 MPa (or 12 000 psi). In passing, it can be said that, at these high strengths, the difference between test results on cubes and on cylinders is minimal, so that, except for compliance purposes, the distinction between

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the two types of test specimens is of little importance. Testing high performance concrete is discussed on p. 685.

One more comment about nomenclature may be in order. In some publications, a subdivision of high performance concrete into classes according to strength is introduced and terms such as 'very high performance concrete' are used. This does not seem to be a rational approach to a material with a continuous gradation in properties and with no discontinuities in ingredients.

High performance concrete contains the following ingredients: common, albeit good quality, aggregate; ordinary Portland (Type I) cement (although rapid-hardening Portland (Type III) cement can be used when high early strength is required) at a very high content, 450 to 550 kg/m^3 (760 to 930 lb/yd^3); silica fume, generally 5 to 15 per cent by mass of the total cementitious material; sometimes, other cementitious materials such as fly ash or ground granulated blast-furnace slag; and always a superplasticizer. The

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dosage of the superplasticizer is high: 5 to 15 litres per cubic metre of concrete, depending on the solids content in the superplasticizer, as well as on its nature. Such a dosage allows a reduction in water content of about 45 to 75 kg/m^3 of concrete.[13.79] Other admixtures can also be present, but polymers, epoxies, fibres, and processed aggregates such as calcined bauxite sand are excluded from consideration in this book. It is essential that high performance concrete is capable of being placed in the structure by conventional methods and that it is cured in the usual manner, although particularly good moist curing is required. What makes the concrete a high performance one is a very low water/cement ratio: always below 0.35, often around 0.25, and occasionally even 0.20.

The above discussion makes it clear that what was said earlier in this chapter about the properties of concrete containing silica fume and a superplasticizer applies to high performance concrete but, because of the very low water/cement

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ratio in the latter, these properties are accentuated. Indeed, high performance concrete can be said to be a logical development of concrete containing silica fume and a superplasticizer. To give an example, it is possible to produce a mix with a slump of 180 to 200 mm (7 to 8 in.) at a water/cement ratio between 0.2 and 0.3; thus, the water content is 130 to 140 kg per cubic metre of concrete (220 to 240 lb/yd^3), as compared with 170 to 200 kg/m^3 (290 to 340 lb/yd^3) in a non-air-entrained mix of ordinary concrete with a slump of 100 to 120 mm (4 to 5 in.).

It was mentioned earlier that high performance concrete means concrete with a high strength or a low permeability. The two properties, although not necessarily concomitant, are linked to one another because high strength requires a low volume of pores, especially of the larger capillary pores. The only way to have a low volume of pores is for the mix to contain particles graded down to the finest size: this is achieved by the use of silica fume which fills the

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spaces between the cement particles and between the aggregate and the cement particles. However, the mix must be sufficiently workable for the solids to be dispersed in such a manner that dense packing is achieved, which requires deflocculation of cement particles. This is achieved by the use of a superplasticizer at a large dosage. The superplasticizer must be effective with the given Portland cement, that is, the two materials must be compatible.

When the above conditions have been satisfied, high performance concrete is achieved. The concrete is very dense, it has a minimal volume of capillary pores, and these pores become segmented upon curing. At the same time, a significant proportion of Portland cement remains unhydrated, even when the concrete is in contact with water because water cannot penetrate through the pore system so as to reach the unhydrated remnants of Portland cement. These remnants can be viewed as very fine 'aggregate'

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particles which are extremely well bonded to the products of hydration.

Properties of aggregate in high performance concrete

Although common aggregates are used in making high performance concrete, in concretes of very high strength, the strength of the coarse aggregate particles themselves can be critical. In consequence, the strength of the parent rock is of importance, but the bond strength of the aggregate particles can also be a limiting factor.[13.91] The mineralogical characteristics of coarse aggregate have been found to influence the strength of the resulting concrete, but no simple guidance on the selection of aggregate is available.[13.64]

The criterion of the strength of aggregate is valid when a high long-term strength of concrete is required. If, however, the desired property of high performance concrete is a high strength at a very early age (say, 40 MPa at 2 days) but a higher strength in the long term is unnecessary,

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then the strength of the aggregate particles is unimportant.

Generally, however, good quality aggregate must be used. To ensure good bond between the coarse aggregate particles and the matrix, these particles should be approximately equi-dimensional.[13.78] It should be remembered that the shape of crushed particles depends, in addition to the parent rock type and its bedding, also on the method of crushing used, impact crushers generally producing few elongated or flaky particles. Gravel is satisfactory as far as shape is concerned and it can be used in high performance concrete,[13.78] but the aggregate–matrix bond may be inadequate when the surface texture of the gravel is very smooth.

Cleanliness of the aggregate, absence of adhering dust, and uniformity of grading are essential. Durability of coarse aggregate particles is vital when the concrete containing the given aggregate is likely to be exposed to freezing and thawing.

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Fine aggregate should be rounded and uniformly graded, but rather coarse, because the rich mixes used in high performance concrete have a high content of fine particles; a fineness modulus of between 2.8 and 3.2 is sometimes recommended.[13.131] However, experience with high performance concrete in terms of the range of aggregates used, both fine and coarse, is limited to only a few geographical areas so that generalizations are not available.

One more comment should be made about the system of solid particles in the mix. At the coarse end, large particles of aggregate are undesirable because they introduce a heterogeneity in the system in that, at the interface, there may be an incompatibility between the aggregate and the surrounding hydrated cement paste in terms of the modulus of elasticity, Poisson's ratio, shrinkage, creep, and thermal properties. This incompatibility may lead to more microcracking than when the maximum size of aggregate is smaller than 10 or 12 mm ($\frac{3}{8}$ or $\frac{1}{2}$ in.). Although a smaller max-

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imum size of aggregate leads to a higher water demand, this is unimportant when the dosage of superplasticizer is high so that the water content of the mix is low.

The larger total surface area of the aggregate with a smaller maximum size also means that the bond stress is lower so that bond failure does not occur. Consequently, in compression tests, failure occurs through the coarse aggregate particles, as well as through the hydrated cement paste. Development of cracks through the coarse aggregate particles was observed also in flexural tests on high performance concrete.[13.70] This behaviour means that the bond strength is no lower than the tensile strength of the aggregate.

The influence of the modulus of elasticity of the coarse aggregate upon the strength of high performance concrete has not been established, but it is arguable that, because of the monolithic behaviour of the concrete, aggregate with a low modulus of elasticity (that is, a modulus not very different from the modulus of elasticity of hy-

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drated cement paste) leads to lower bond stresses with the matrix. This may be beneficial with respect to high performance concrete.

Aspects of high performance concrete in the fresh state

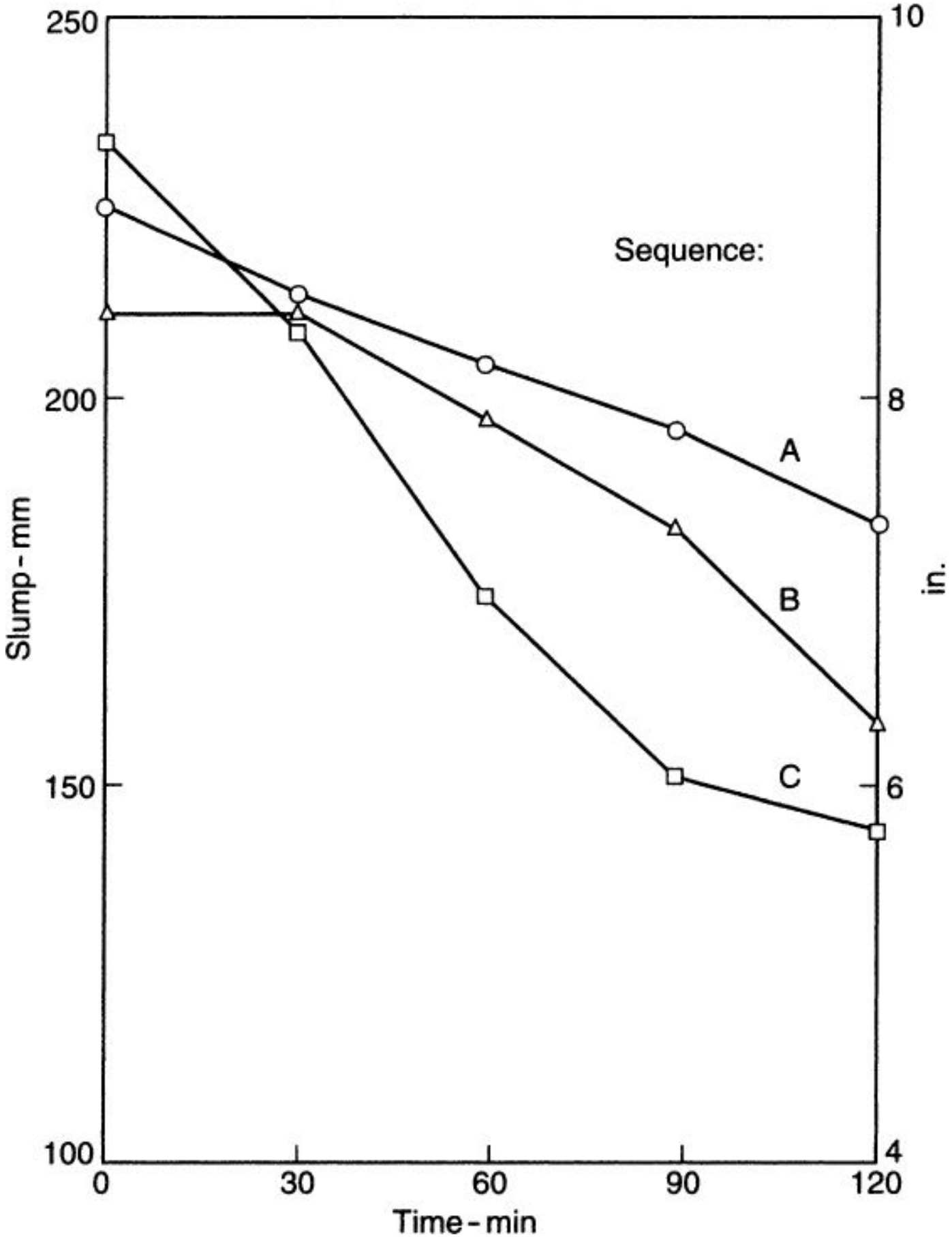
The particular proportions of the ingredients of high performance concrete, namely, the very high cement content, the very low water content, and the high dosage of superplasticizer, influence the properties of the fresh concrete in some respects in a manner different from the usual mixes.

First of all, batching and mixing require particular care. Because of the importance of thorough mixing, using the mixer at less than its rated capacity may be beneficial; a reduction of one-third, or even one-half, may be desirable.[13.98] A longer mixing time than usual is required to ensure homogeneity of what is usually rather a sticky mix: 90 seconds has been recommended,[13.93] but even longer periods may be desirable.

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The sequence of feeding the ingredients into the mixer is best established by trial-and-error, and it can be complicated. In one case, some water and one-half of a superplasticizer were fed first; then, aggregate and cement; finally, the remainder of the water and the superplasticizer. Often, a part of the superplasticizer is added only immediately prior to the placing of concrete. An example of the effect of the mixing sequence upon slump loss of concrete with a water/cement ratio of 0.25, mixed during 225 seconds is shown in Fig. 13.6.[13.81] Three sequences were used: (A) feeding all the ingredients simultaneously; (B) mixing cement and water prior to the feeding of the remaining ingredients; and (C) mixing cement and fine aggregate prior to the feeding of the remaining ingredients. Method A resulted in the lowest slump loss, but this observation may not be generally valid.

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Fig. 13.6. Effect of sequence of batching on slump loss with time since mixing of concrete with a water/cement ratio of 0.25 and a super-plasticizer[13.81]

To optimize setting time and the development of early strength of high performance concrete, a combination of a superplasticizer and a lignosulfonate-based water-reducing admixture or a retarder can be used.[13.55]

Some of the superplasticizer must be introduced early into the mixer in order to achieve adequate workability in the first place. The timing of adding the final portion of the superplasticizer is of particular importance. It is essential to ensure that the superplasticizer does not become fixed by C_3A in the Portland cement and, consequently, is no longer available to maintain a high workability. Such fixing would occur if SO_4^{--} from the calcium sulfate in Portland cement is not liberated fast enough to react with C_3A. It is, therefore, important to avoid the re-

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action between the superplasticizer and C_3A by ensuring a compatibility between the superplasticizer and the Portland cement to be used; this topic is discussed in the next section.

At this stage, one more comment is pertinent. The water requirement is influenced by the carbon content of the silica fume used; a high content can be detected simply by a dark colour of the silica fume.[13.68]

Compatibility of Portland cement and superplasticizer

In the preceding section, we pointed out the difficulty of maintaining an adequate workability if the superplasticizer becomes fixed by C_3A in the Portland cement used. When this happens, the two materials can be said to be incompatible. Conversely, if the difficulty is absent, the Portland cement and the superplasticizer are said to be compatible. Although comparability of cement and admixtures is relevant in ordinary concrete as well, in high performance concrete, the

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very low water content greatly magnifies the consequences of a lack of compatibility because of the competition by the various ingredients for water for surface wetting and early hydration. The rate of solubility of calcium sulfate is critical when there is less water to accept the sulfate ions and, at the same time, there is more C_3A (because of a high cement content) whose reaction must be controlled to ensure workability. For these reasons, tests involving the given materials, but using a water/cement ratio of about 0.5, do not give information on the behaviour at a water/cement ratio in the vicinity of 0.3.

In essence, the problem is that of the length of time after mixing, before the SO_4^{--} ions from Portland cement become available for reaction with C_3A, so that the sulfonate ends of the superplasticizer molecules do not become fixed. The various forms of calcium sulfate in Portland cement were discussed on p. 17, and it should be remembered that gypsum, hemihydrate, and anhydrite have different rates of solubility. The situ-

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ation is complicated by the fact that the solubility of anhydrite depends on its structure and origin.

The solubility and the rate of solution of calcium sulfate are affected by the superplasticizer, both in so far as its type and its dosage are concerned. In the present state of our knowledge, a translation of these qualitative factors into a prediction of compatibility is not possible, and an experimental assessment of the rheological properties of any given combination of Portland cement and superplasticizer is necessary.

Nevertheless, it can be said that the important factors in compatibility are as follows.[13.79] For the cement, these are the content of C_3A and C_4AF, the reactivity of C_3A (which depends on its morphological form and on the degree of sulfurization of the clinker), the content of calcium sulfate, and the final form of the calcium sulfate in the ground cement (namely, gypsum, hemihydrate, or anhydrite). For the polysulfonate, the important factors are the molecular chain length, the position of the sulfonate group in the chain,

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the counter-ion type (that is, sodium or calcium), and the presence of residual sulfates, which affect the cement deflocculation properties.

On the basis of these factors, an ideal cement for high performance concrete from the rheological point of view can be postulated: not too fine (probably up to 400 m^2/kg determined by the Blaine method), with a very low C_3A content whose reactivity is easily controlled by the sulfate ions derived from the solution of the sulfates present in the cement. An ideal polysulfonate superplasticizer should consist of rather long molecular chains in which, for example, the sulfonate groups occupy the β-position in a sodium salt condensate of formaldehyde and naphthalene sulfonates. As far as the content of residual sulfates in the polysulfonate is concerned, this depends on the content and solubility of the sulfates in the cement with which the superplasticizer is to be used: what is necessary is an adequate amount of soluble sulfates in the mixture.[13.79]

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The preceding guidelines make it possible to eliminate inappropriate cements and superplasticizers. The next step is laboratory testing on a trial-and-error basis of a number of neat cement pastes containing combinations of different cements and superplasticizers for the purpose of establishing the best combination from the rheological point of view.

The effectiveness of a given superplasticizer with a given cement can be tested by measuring the time taken by a fixed quantity of neat cement paste made with the two materials to flow through a standard funnel, known as a Marsh flow cone. The time decreases with an increase in the dosage of superplasticizer up to a saturation point, beyond which additional superplasticizer is no longer beneficial. The time to empty the cone becomes longer when the testing of the cement paste is delayed; this is an indication of a loss of workability. A compatible combination of Portland cement and superplasticizer exhibits only a small loss between tests at 5 and 60 minutes, and

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also exhibits a definite saturation point beyond which additional superplasticizer is of no benefit (see Fig. 13.7).[13.63]

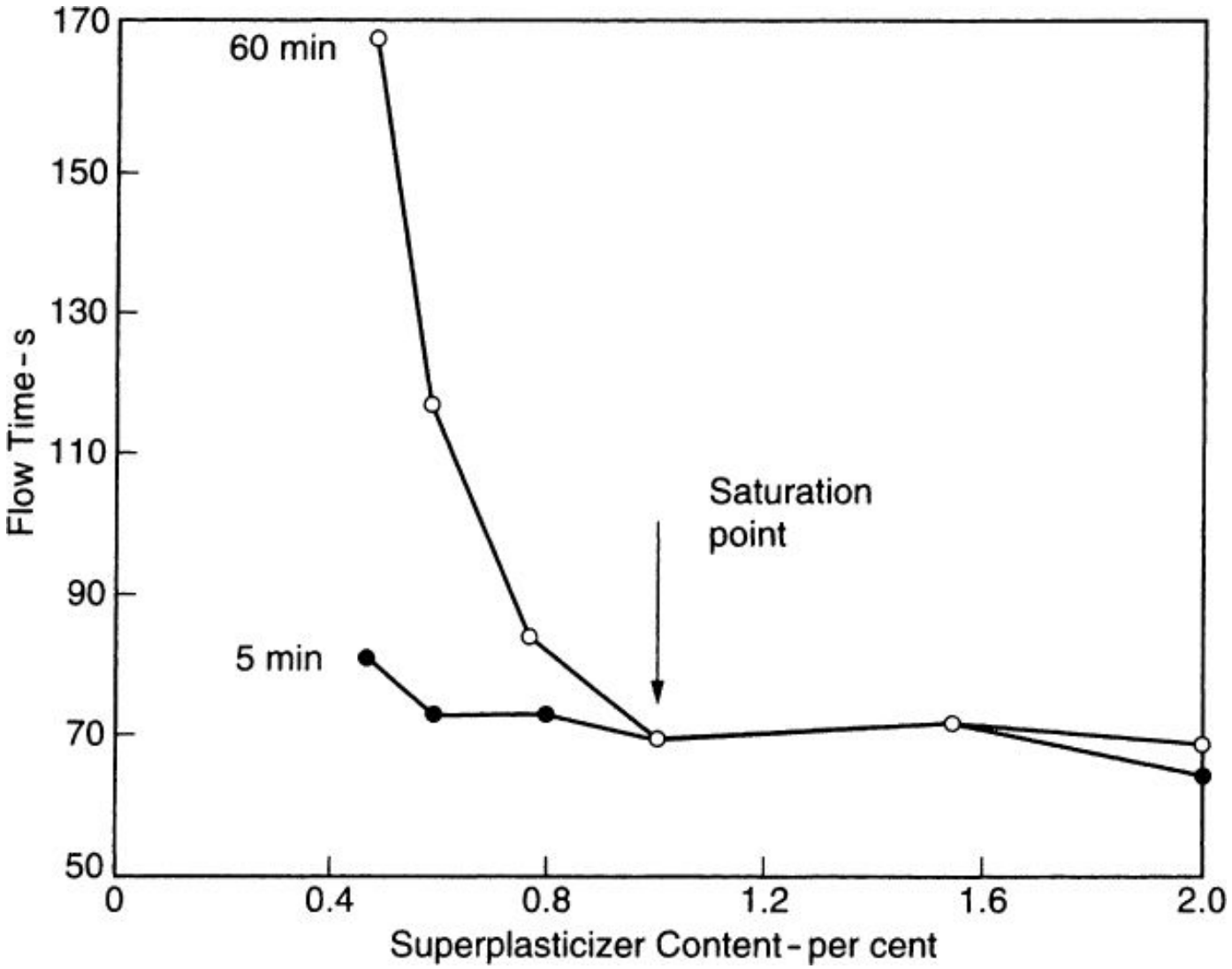


Fig. 13.7. Flow time through a Marsh cone as a function of content of superplasticizer (by mass of solids) in neat cement paste with a water/cement ratio of 0.35 after 5 and 60 minutes since mixing[13.63]

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Such tests on neat cement paste make it possible to narrow the choice to a few cements compatible with one or two superplasticizers which are commercially available. For the final selection of the cement and the superplasticizer, it is necessary to make tests on a trial concrete mix because only such tests give truly reliable data on the slump loss and strength gain.

Aspects of hardened high performance concrete

While there exist no standard, or even typical, mix proportions of high performance concrete, it is useful to present information on several successful mixes; this is given in Table 13.5. Several of these mixes contain, in addition to Portland cement and silica fume, other cementitious materials. There is an economic advantage in using these various cementitious materials, partly because they are cheaper than Portland cement, but also because they allow a reduction in the dosage of superplasticizer.[13.79]

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Table 13.5. Mix Proportions of Some High Performance Concretes*

Ingredient (kg/m³)	Mix								
	A	B	C	D	E	F	G	H	I
Portland cement	534	500	315	513	163	228	425	450	460
Silica fume	40	30	36	43	54	46	40	45	
Fly ash	59	—	—	—	—	—	—	—	—
ggbs	—	—	137	—	325	182	—	—	—
Fine aggregate	623	700	745	685	730	800	755	736	780
Coarse aggregate	1069	1100	1130	1080	1100	1110	1045	1118	1080
Total water	139	143	150	139	136	138	175†	143	138
Water/cementitious material ratio	0.22	0.27	0.31	0.25	0.25	0.30	0.38	0.29	0.30
Slump, mm	255	—	—	—	200	220	230	230	110
Cylinder strength (MPa) at age (days)									
1	—	—	—	—	13	19	—	35	36
2	—	—	—	65	—	—	—	—	—
7	—	—	67	91	72	62	—	68	—
28	—	93	83	119	114	105	95	111	83
56	124	—	—	—	—	—	—	—	—
91	—	107	93	145	126	121	105	—	89
365	—	—	—	—	136	126	—	—	—

*Further information about mixes: (A) United States;[13.97] (B) Canada;[13.79] (C) Canada;[13.79] (D) United States;[12.79] (E) Canada;[13.95] (F) Canada;[13.95] (G) Morocco;[13.82] (H) France;[13.82] (I) Canada.[13.125]
†It is suspected that the high water content was occasioned by a high ambient temperature in Morocco.

A mix of particular interest is Mix E of Table 13.5 which had a water/cement ratio of 0.25, a total content of cementitious material of 542 kg/m^3 of which only 30 per cent was Portland cement, and 10 per cent silica fume. The compressive strength at 28 days was 114 MPa but it reached 136 MPa at the age of 1 year. It should be emphasized that this was not a laboratory concrete but it was produced in a ready-mixed plant.[13.79] It is worth adding that commercial pro-

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duction of high performance concrete necessitates a very strict and consistent quality control.

At the outset of the discussion of high performance concrete, it was said that this material is simply an extension of the range of the more usual concrete mixes. This is confirmed by the continuous nature of the broad relation between strength and the water/cement ratio, illustrated in Fig. 13.8. This figure is based on data cited by Fiorato[13.54] for test cylinders cured in a variety of ways and tested at ages from 28 days onwards; results of tests on zero-slump concrete without silica fume have been omitted.

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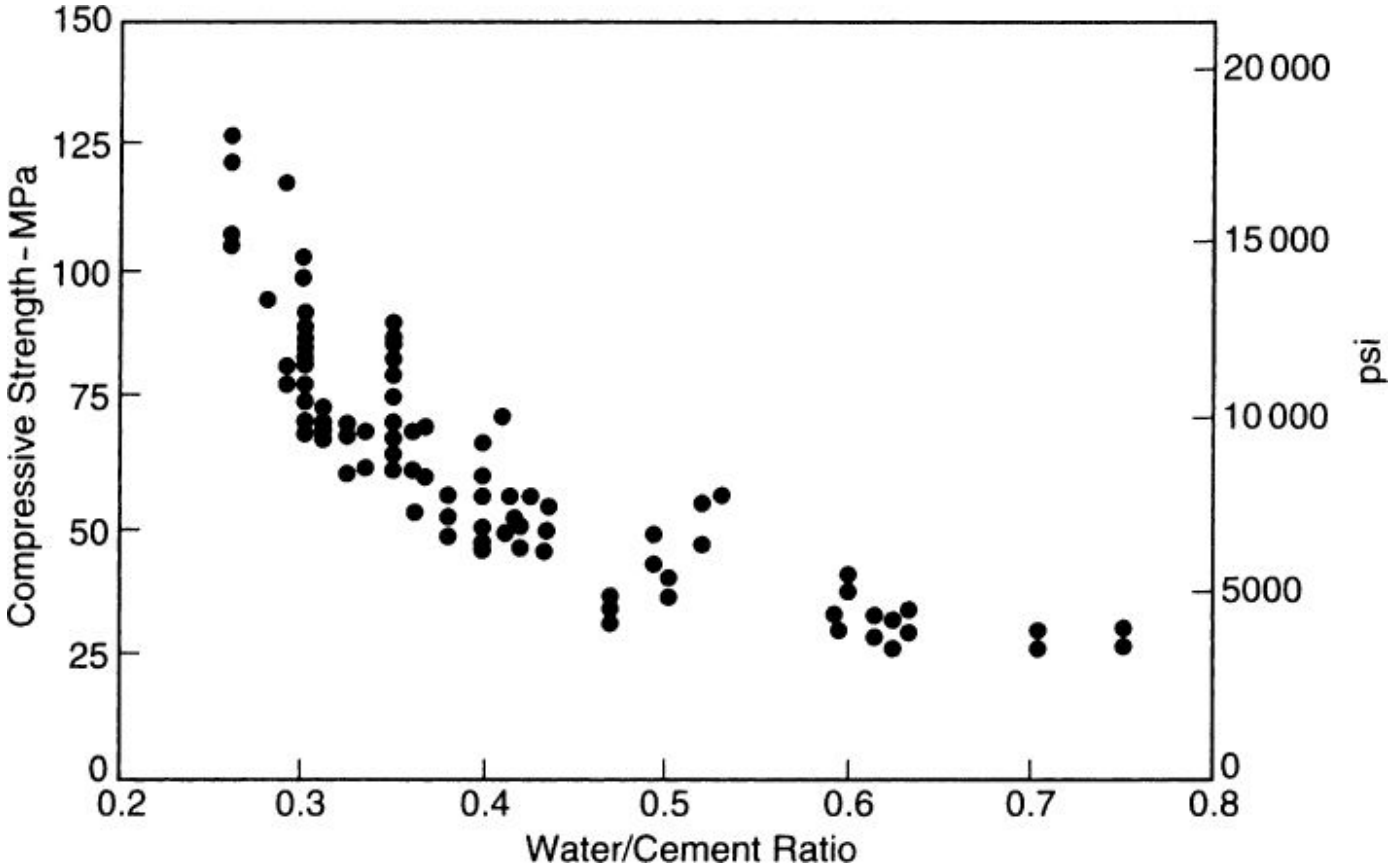


Fig. 13.8. Relation between compressive strength and water/cement ratio for test cylinders of non-air-entrained concretes containing various cementitious materials tested at ages between 28 and 105 days (based on ref. 13.54)

In the case of a relatively new material, such as high performance concrete, it is useful to know whether any retrogression of strength occurs. Tests on cores from a simulated column made of concrete with a 28-day compressive strength of

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85 MPa (12 300 psi) have shown no change in strength after 2 or 4 years.[13.74] Reported retrogression of strength of test cylinders stored under dry conditions between the ages of 90 days and 4 years can be explained by self-stressing through drying of the surface zone of the cylinders;[13.56] such drying is absent in concrete in structures.

Information on the relation between the modulus of rupture or the splitting tensile strength and the compressive strength for high performance concrete is not available, but ACI 363R-92[13.91] suggests expressions applicable up to 83 MPa (12 000 psi). There are indications that, for compressive strengths above about 100 MPa (14 000 psi), there is no further increase in the modulus of rupture or in the tensile strength.[13.72]

In the case of high performance concrete with a high very *early* compressive strength, there is a possibility that, because of a limited amount of hydration, the bond between the aggregate and the matrix is not commensurately developed.

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Consequently, with high strength at very early ages, flexural strength and the modulus of elasticity are likely to be lower than would be expected from the usual relations between these properties and the compressive strength.[13.99]

The elastic deformation of high performance concrete is of particular interest. Because the modulus of elasticity of the very strong hardened cement paste and of the aggregate differ less from one another than in medium strength concrete, the behaviour of high performance concrete is more monolithic and the strength of the aggregate–matrix interface is higher. There is, therefore, less bond microcracking, and the linear part of the stress–strain curve extends up to a stress which may be as high as 85 per cent of the failure stress, or even higher (see Fig. 13.9). Subsequent failure takes place through the coarse aggregate particles as well as through the matrix. Thus, the coarse aggregate particles do not act as crack arrestors so that failure is rapid.[13.71]

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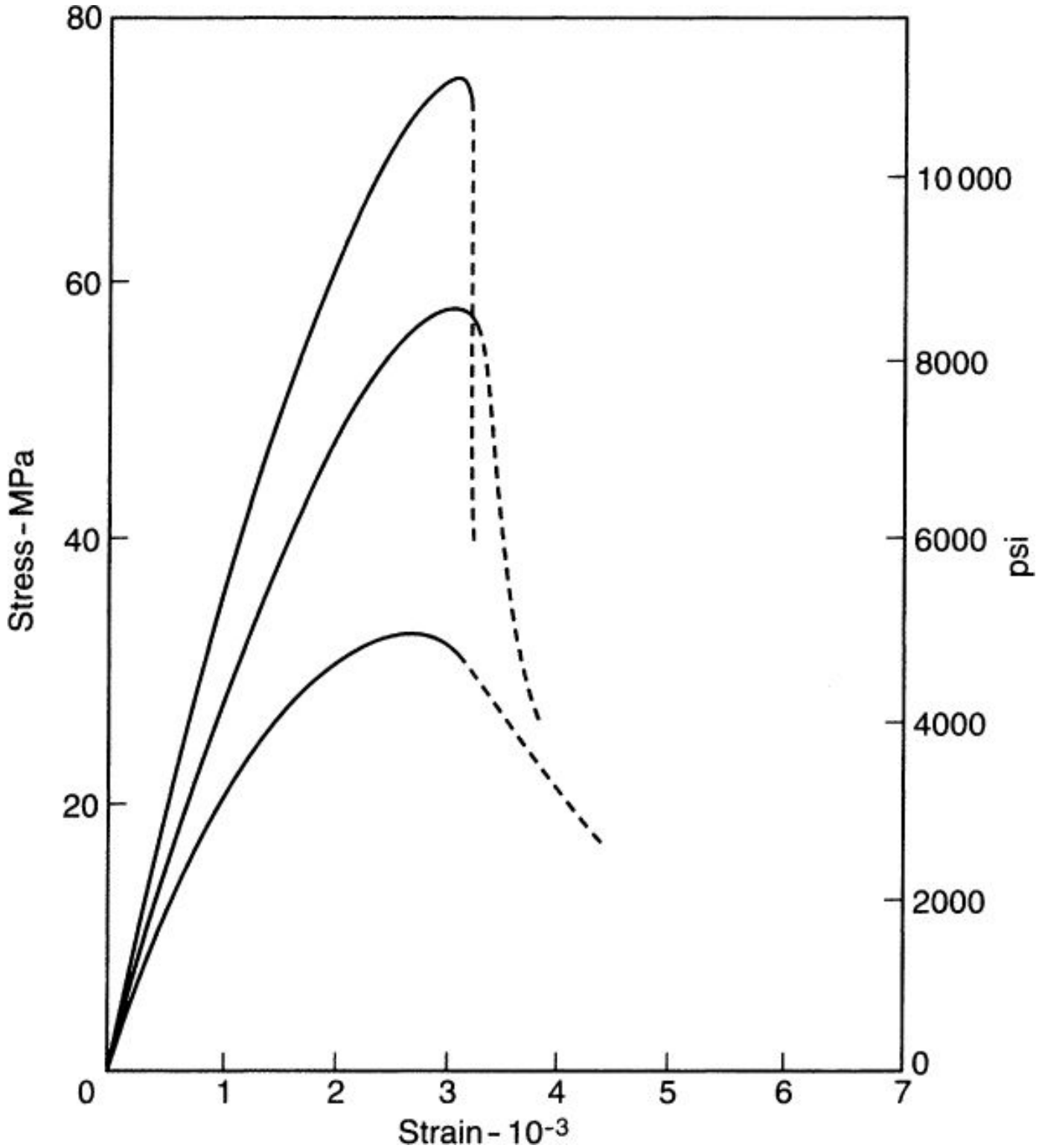


Fig. 13.9. Typical stress–strain curves for concretes of different strengths[13.71]

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A reasonable relation between the modulus of elasticity of concrete, E_c, and its 28-day compressive strength, both in MPa, is:[13.91]

$$E_c = 3320\sqrt{f'_c} + 6900.$$

In psi units, this equation becomes:

$$E_c = 40\,000\sqrt{f'_c} + 10^6.$$

It is doubtful that this expression is valid at strengths well above 83 MPa (12 000 psi): generally, the modulus of elasticity at very high strengths is lower than would be expected from an extrapolation of the above expressions. Some Japanese data on the modulus of elasticity of concrete with strengths between 75 and 140 MPa (11 000 and 20 000 psi) are shown in Fig. 13.10.[13.81]

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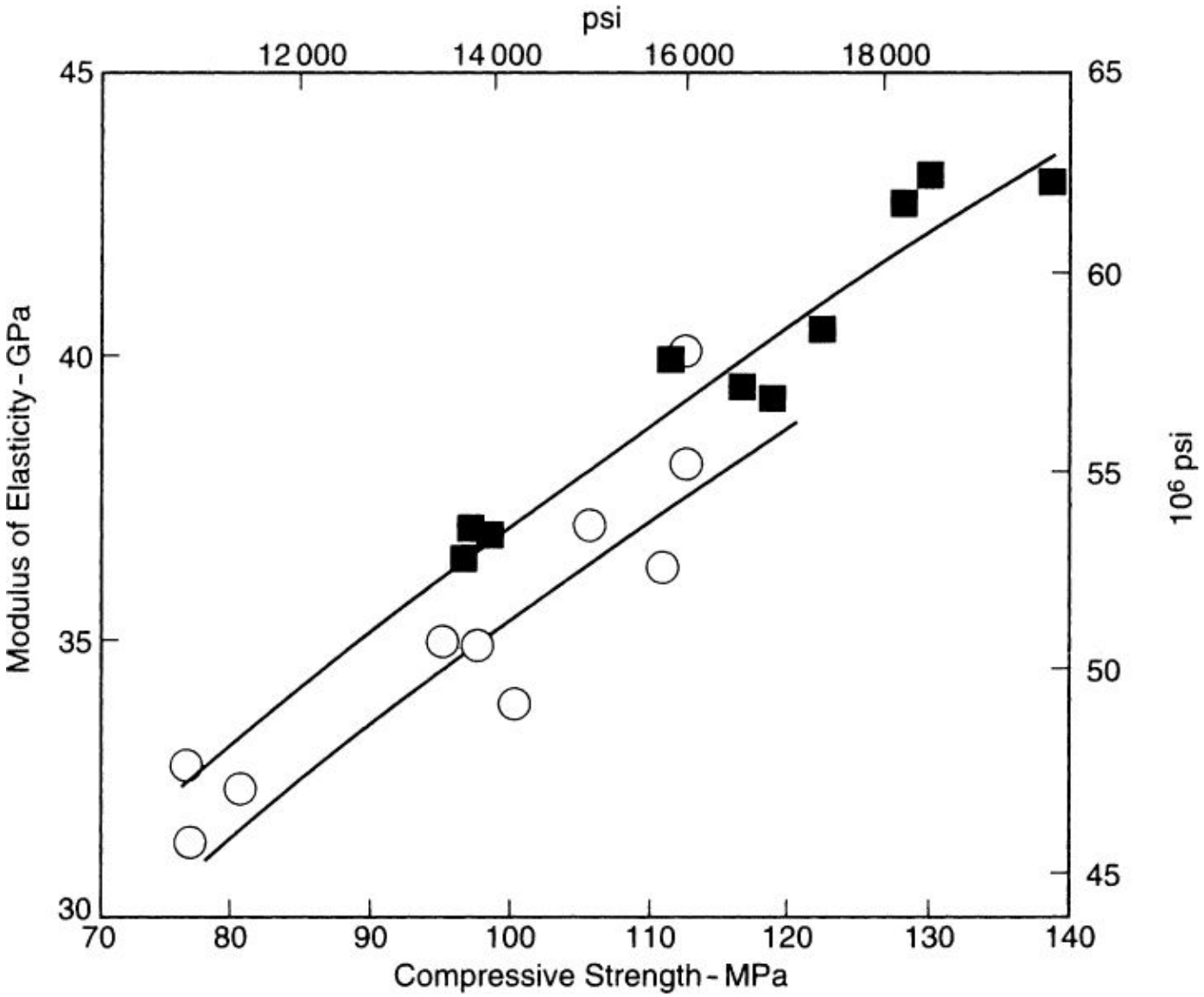


Fig. 13.10. Relation between the modulus of elasticity and compressive strength for high strength concretes with a water/cement ratio of 0.25 tested at different ages (based on ref. 13.81)

Because of the strong bond between the coarse aggregate and the matrix, the elastic properties of aggregate have a considerable influence on

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the modulus of elasticity of the concrete.[13.73] In consequence, the relation between the modulus of elasticity of high performance concrete and its strength is much less consistent than in the case of usual concrete; this is true regardless of the particular relation used.[13.73] Accordingly, for structural design purposes the modulus of elasticity of high performance concrete should not be assumed to be a simple function of compressive strength.

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Testing of high performance concrete

Testing standard compression specimens – 150 by 300 mm (6 in. by 12 in.) cylinders or 150 mm (6 in.) cubes – may present a problem with respect to the capacity of the testing machine; as 80 per cent of the capacity should not be exceeded,[13.131] a capacity of 4 MN (or 1 million lb) may be required. The use of smaller specimens is, therefore, preferable: specifically, 100 by 200 mm (4 by 8 in.) cylinders or 100 mm (4 in.) cubes are satisfactory, given that, in high performance concrete, the maximum aggregate size is generally smaller than 12 mm ($\frac{1}{2}$ in.). Such smaller test specimens give a measured strength about 5 per cent higher than standard specimens[13.63,13.77] (see also p. 609).

In addition, the capping material used with test cylinders must not affect the recorded failure load on the specimen. For this reason, grinding of end surfaces is preferable.[13.77]

The use of accelerated strength tests on high performance concrete is an attractive proposition

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in so far as production control is concerned. A relation between this strength and the desired strength at the specified age should be established experimentally prior to the commencement of concreting.

Durability of high performance concrete

One of the main features of high performance concrete is its very low penetrability; the consequences of this merit considerable attention.

The fact that high performance concrete has a particularly dense structure of hydrated cement paste – indeed, this is what imparts high performance – with a discontinuous capillary pore system, means that high performance concrete possesses a high resistance to external attack. This is particularly true with respect to the ingress of chlorides into the concrete. For instance, tests similar to those of ASTM C 1202-10, on 3-month-old cores from columns made with 120 MPa concrete (17 000 psi) have shown a negligible chloride-ion permeability.[13.65] Even con-

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crete with a water/cement ratio of 0.22, subjected to drying at 105 °C (221 °F), which removes the evaporable water from the hardened cement paste, was found, on subsequent exposure to chloride ions, to have an extremely low permeability to chloride ions.[13.66]

With respect to the risk of alkali–silica reaction, high performance concrete containing silica fume can be expected to be particularly resistant because it has a very low permeability, which limits the mobility of ions, as well as a very low water content.[13.80] It should be remembered that the presence of water is essential for the alkali–silica reaction to take place. Figure 13.11 shows the very low relative humidity in the interior of concretes with 28-day strengths above 80 MPa (or 12 000 psi).[13.75] This militates against the occurrence of the alkali–silica reaction. Indeed, no cases of alkali–silica reaction in high performance concrete have been reported in the literature up to 1994,[13.75] but the harmful effects

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of such a reaction take a very long time to manifest themselves.

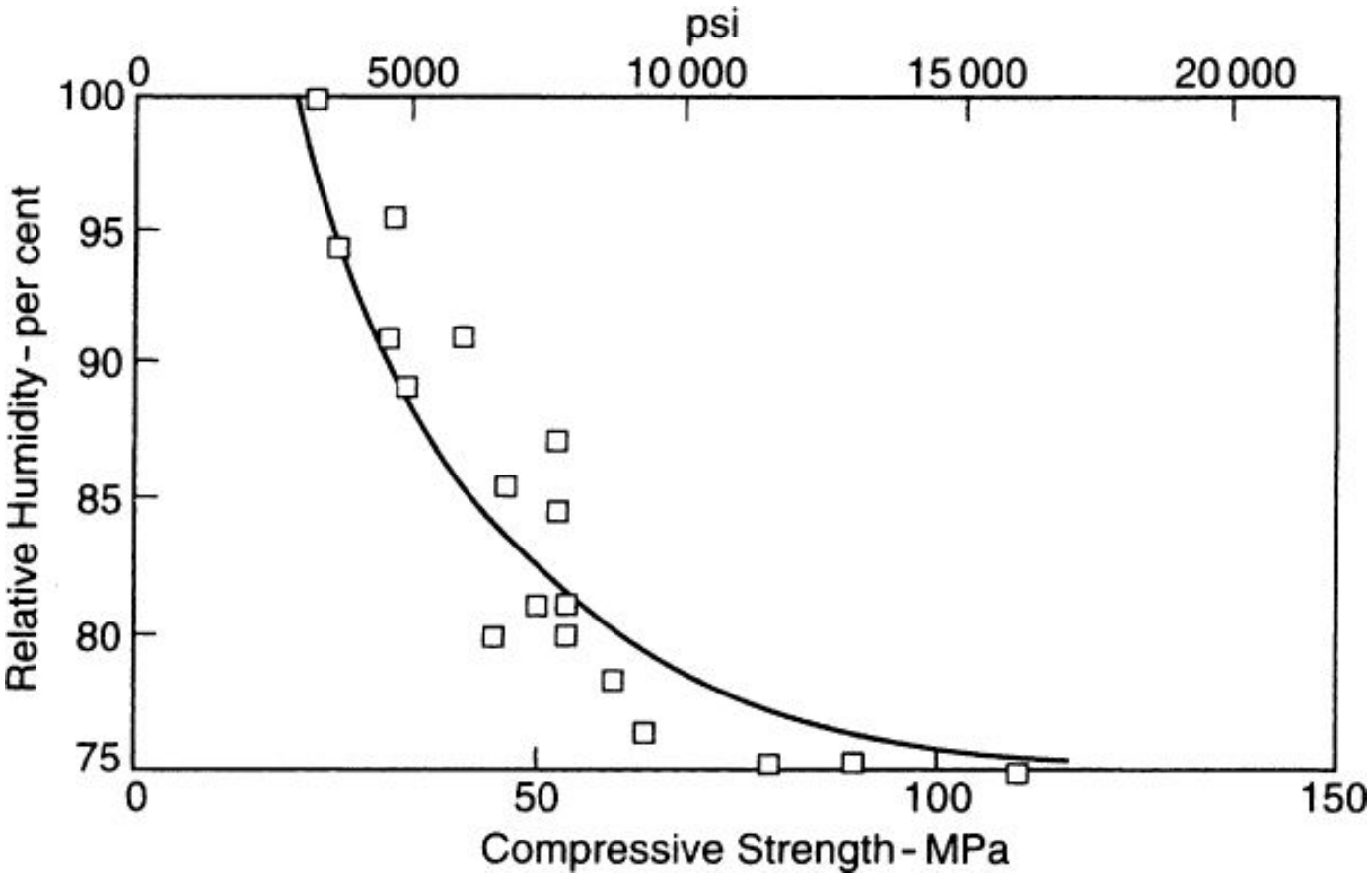


Fig. 13.11. Relation between internal relative humidity in concrete aged 3 months and its characteristic strength at the age of 28 days[13.75]

As far as the resistance to freezing and thawing is concerned, several aspects of high performance concrete should be considered. First, the structure of hydrated cement paste is such that

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very little freezable water is present. Second, entrained air reduces the strength of high performance concrete because the improvement in workability due to the air bubbles cannot be fully compensated by a reduction in the water content in the presence of a superplasticizer. In addition, air entrainment at very low water/cement ratios is difficult. It is, therefore, desirable to establish the maximum value of the water/cement ratio below which alternating cycles of freezing and thawing do not cause damage to the concrete.

However, some other factors also influence the resistance of concrete to freezing and thawing. These include the characteristics of the cement and the effectiveness of curing prior to exposure to freezing and thawing.[13.67] While there are indications that the limiting value of the water/cement ratio, referred to above, is 0.25 or 0.30,[13.76] it cannot be assumed that concrete with a water/cement ratio below such a limiting value is necessarily resistant to alternating freezing and thawing. On the other hand, it is possible that an

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air-void spacing factor higher than necessary in usual concrete will ensure protection from freezing and thawing, but reliable data are not available. Even greater uncertainty applies to the resistance to scaling by deicing agents because the surface zone of the concrete, if not well cured and then dried out, is likely to be vulnerable.

It is worth noting that mixes selected so as to have a high strength at the age of several hours (presumably because they will then be put into service which may include exposure to freezing and thawing) must contain entrained air, even if they have a very low water/cement ratio. The reason for this requirement is that, because there has been no adequate curing, the water in the capillary voids may freeze.

The categorization of high performance concrete as resistant, or not resistant, to freezing and thawing is complicated by the fact that ASTM C 666-03 (2008) considers to be usual a test at an early age without allowing the concrete to dry out. It is likely that, under service conditions in

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structures such as bridge decks or dense overlays on decks, the surface zone of the concrete would dry out before exposure to freezing and, because of the very low permeability of high performance concrete, re-saturation could not take place. Consequently, in service, the exposure conditions are probably less severe than in the tests prescribed by ASTM C 666-03 (2008), especially in Procedure A, which requires both freezing and thawing to take place in water.

The abrasion resistance of high performance concrete is very good, not only because of the high strength of the concrete, but also because of the good bond between the coarse aggregate and the matrix, which prevents differential wear of the surface.

On the other hand, high performance concrete has a poor resistance to fire because the very low permeability of high performance concrete does not allow the egress of steam formed from water in the hydrated cement paste. This may cause bursting, which, of course, is undesirable.

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Explosive failure may occur in the compressive zone but also in the tensile zone.[13.161] (see also p. 391.)

The absence of open pores in the surface zone of high performance concrete prevents growth of bacteria: this can be exploited in floor slabs in areas used by piglets and chickens, where a decrease in morbidity was reported.[13.130]

Because of its high cement content, high performance concrete is sensitive to problems arising from the development of the heat of hydration of cement; appropriate measures need to be taken (see Chapter 8). It is worth repeating that, because high performance concrete is essentially a modification of ordinary concrete, high performance concrete is affected by the various cementitious materials in the usual manner. For example, fly ash can be incorporated in high performance concrete for the purpose of reducing the early development of the heat of hydration, as well as of improving workability and reducing slump loss.

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There exist no consistent data which suggest that shrinkage or creep of high performance concrete is different from what could be expected from the properties and proportions of the mix ingredients. The influence of silica fume is particularly relevant: because it greatly reduces the movement of water, and therefore drying creep, the magnitude of creep is not affected by the volume/surface ratio of the concrete element.[13.94]

The future of high performance concrete

Full structural exploitation of very high strength concrete is still to come, most design codes not taking into account strengths in excess of 60 MPa (9000 psi). Nevertheless, use of higher strengths in structural design is being progressively introduced. In some structures, it is not the high strength as such that is required, but high strength is specified in order to exploit the associated high modulus of elasticity. On the other hand, in very many practical applications, it is the high durability, arising from the very low permeability

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and high resistance to abrasion that is of paramount importance. These properties of high performance concrete are already being exploited.

There is thus little doubt that the use of high performance concrete in construction will continue to grow. No technical difficulties exist. However, such growth necessitates a provision by ready-mixed concrete producers of concrete made under a very high control of quality of ingredients and of processing. At the same time, such provision is conditional upon demand from engineers and owners who, not surprisingly, are reluctant to specify a material which is not readily available. It is this impasse that needs to be broken in order to benefit from the use of this highly valuable and economic material; the value-for-money should be assessed not in terms of initial cost alone but also of enhanced durability, as well as often smaller, and therefore lighter, structural members which encroach less on usable space and require smaller foundations.

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Lightweight concrete

Concrete made with natural aggregate originating from hard rock has a density within a narrow range because the specific gravity of most rocks varies little (see Table 3.7). Although the volumetric content of the aggregate in the mix affects the density of the concrete, this, too, is not a major factor. Thus, in practice, the density of normal weight concrete lies within the range of 2200 to 2600 kg/m^3 (140 to 160 lb/ft^3). Consequently, the self-weight of concrete elements is high and can represent a large proportion of the load on a structure. Using concrete with a lower density can, therefore, result in significant benefits in terms of load-bearing elements of smaller cross-section and a corresponding reduction in the size of foundations. Occasionally, the use of concrete with a lower density permits construction on ground with a low load-bearing capacity.[13.85] Furthermore, with lighter concrete, the formwork need withstand a lower pressure than would be the case with normal weight concrete,

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and also the total mass of materials to be handled is reduced with a consequent increase in productivity. Concrete which has a lower density also gives better thermal insulation than ordinary concrete (see Fig. 13.16). On the other hand, lightweight concrete has a higher cement content than normal weight concrete. This represents additional cost, and so does the more expensive lightweight aggregate. A meaningful comparison of cost, however, cannot be limited to the cost of materials but should be made on the basis of the design of the *structure* using lightweight concrete.

In most respects, concrete which has a low density behaves like normal weight concrete. However, there are certain features of concrete which are linked to the lower density; only these will be specifically discussed in what follows.

Classification of lightweight concretes

The density of concrete can be reduced by replacing some of the solid material in the mix

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by air voids. There are three possible locations of the air: in the aggregate particles, which are known as *lightweight aggregate*; in the cement paste, the resulting concrete being known as *cellular concrete*; and between the coarse aggregate particles, the fine aggregate being omitted. Concrete made in the last-named manner is known as *no-fines concrete*. Concrete made with lightweight aggregate is known as *lightweight aggregate concrete*, and is a particular category of lightweight concrete.

The practical range of densities of lightweight concrete is between about 300 and 1850 kg/m^3 (20 and 115 lb/ft^3) (see Fig. 13.12). Classification on the basis of density is sensible because density and strength are largely concomitant, and ACI 213R-87[13.141] uses density to categorize concrete according to its application. There are three categories. *Structural lightweight concrete* has a density between 1350 and 1900 kg/m^3 (85 and 120 lb/ft^3); as its name implies, this concrete is

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used for structural purposes and has a minimum compressive strength of 17 MPa (2500 psi). *Low-density concrete* has a density between 300 and 800 kg/m^3 (20 and 50 lb/ft^3); this concrete is used for non-structural purposes, mainly for thermal insulation purposes. In between these two categories lies *moderate strength concrete*. Its compressive strength, measured on standard cylinders, is between 7 and 17 MPa (1000 and 2500 psi) and the thermal insulation characteristics are in-between those of low-density concrete and structural lightweight concrete. Typical properties of common lightweight concretes are shown in Table 13.6.

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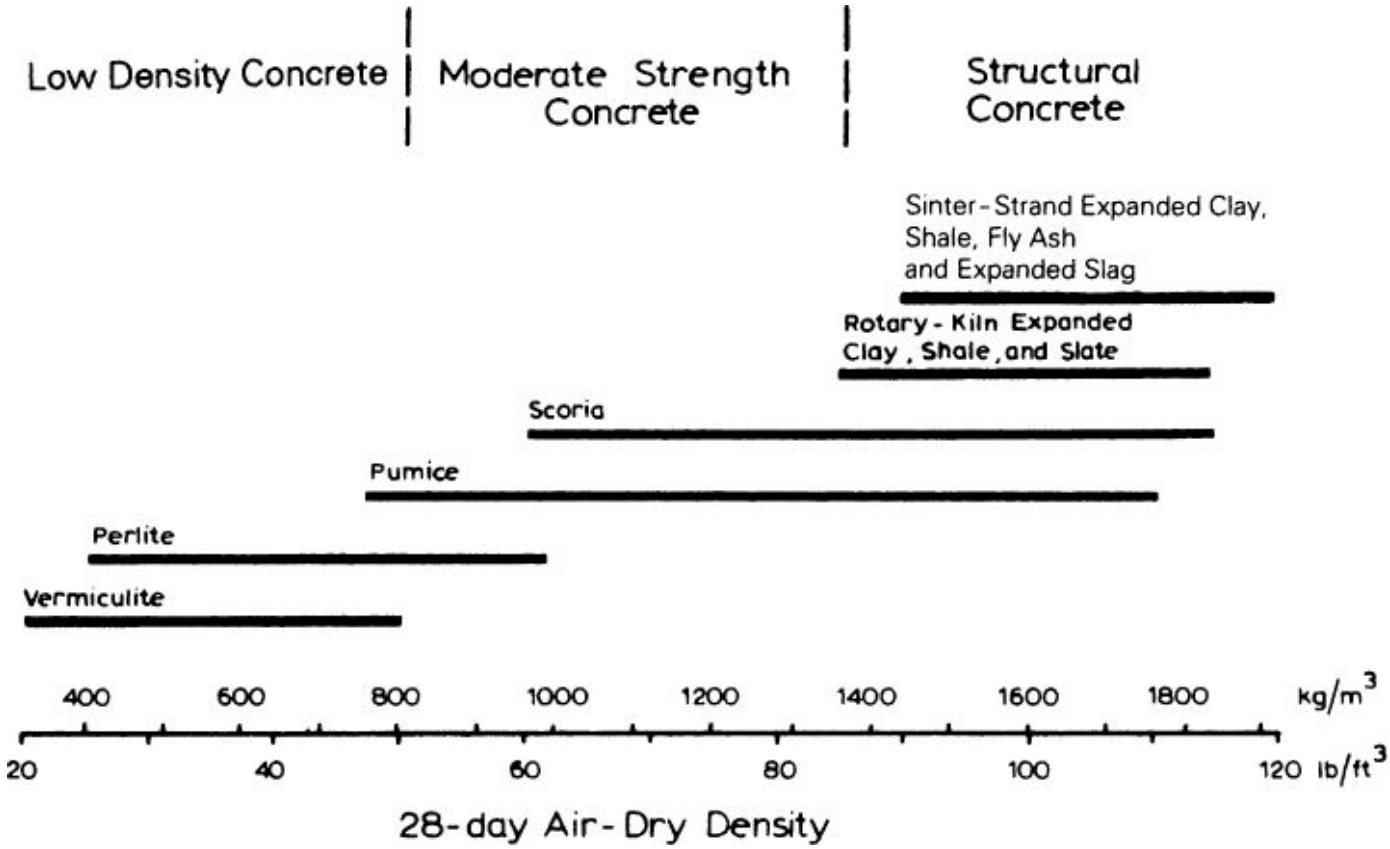


Fig. 13.12. Typical ranges of air-dry densities of concretes made with various lightweight aggregates (partly based on ACI 213R-87[13.141])

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Table 13.6. Typical Properties of Common Lightweight Concretes

Type of concrete		Bulk density of aggregate		Dry density of concrete		Compressive strength at 28 days		Drying shrinkage, 10^{-6}	Thermal conductivity	
		kg/m³	lb/ft³	kg/m³	lb/ft³	MPa	psi		J/m/m²s °C	Btu/ft²h °F/ft
Expanded slag	Fine	900	50	1 850	115	21	3 000	500	0.69	0.40
	Coarse	650	40	2 100	130	41	6 000	600	0.76	0.44
Rotary-kiln expanded clay	Fine	700	45	1 200	75	17	2 500	600	0.38	0.22
	Coarse	400	25	1 300	80	20	3 000	700	0.40	0.23
Rotary-kiln expanded clay with natural sand	Coarse	400	25	1 500	95	20	3 000	—	0.57	0.33
				1 600	100	35	5 000	—	—	—
				1 750	110	50	7 000	—	—	—
				1 900*	120	70†	10 000	—	—	—
Sinter-strand expanded clay	Fine	1 050	65	1 500	95	25	3 500	600	0.55	0.32
	Coarse	650	40	1 600	100	30	4 500	750	0.61	0.35
Rotary-kiln expanded slate	Fine	950	60	1 700	105	28	4 000	400	0.61	0.35
	Coarse	700	45	1 750	110	35	5 000	450	0.69	0.40
Sintered fly ash	Fine	1 050	65	1 500	95	25	3 600	300	—	—
	Coarse	800	50	1 540	96	30	4 400	350	—	—
				1 570	98	40	5 800	400	—	—
Sintered fly ash with natural sand	Coarse	800	50	1 700	106	25	3 600	300	—	—
				1 750	109	30	4 400	350	—	—
				1 790	112	40	5 800	400	—	—
Pumice		500–800	30–50	1 200	74	15	2 000	1 200	—	—
				1 250	77	20	2 800	1 000	0.14	0.08
				1 450	90	30	4 200	—	—	—
Perlite		40–200	3–13	400–500	—	1.2–3	—	2 000	0.05	0.03
Vermiculite		60–200	4–13	300–700	20–30	0.3–3	50–400	3 000	0.10	0.06
Cellular	Fly ash	950	60	750	47	3	500	700	0.19	0.11
	Sand	1 600	100	900	55	6	800		0.22	0.13
Autoclaved aerated		—	—	800	55	4	600	800	0.25	0.14

*With fly ash and silica fume.
†At 1 year.

Lightweight aggregates

The essential characteristic of lightweight aggregate is its high porosity, which results in a low apparent specific gravity. Some lightweight aggregates occur naturally; others are manufac-

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tured from natural materials or from industrial by-products.

Natural aggregates

The main aggregates in this category are diatomite, pumice, scoria, volcanic cinders, and tuff; except for diatomite, all of these are of volcanic origin. Because they are found only in some parts of the world, natural lightweight aggregates are not extensively used, but they make good moderate strength concrete.

Pumice is a light-coloured, froth-like volcanic glass with a bulk density in the region of 500 to 900 kg/m^3 (30 to 55 lb/ft^3). Those varieties of pumice which are not too weak structurally make a satisfactory concrete with a density of 800 to 1800 kg/m^3 (50 to 110 lb/ft^3) and with good insulating characteristics, but having high absorption and high shrinkage. The use of pumice in making concrete goes back to ancient Rome: the Pantheon and the Coliseum are extant examples. *Scoria*, which is a vesicular glassy rock, rather

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like industrial cinders, makes a concrete of similar properties to that containing pumice.

Manufactured aggregates

These aggregates are often known by a variety of trade names, but are best classified on the basis of the raw material used and the method of manufacture which causes expansion and, therefore, a reduction in apparent specific gravity.

The lightweight aggregates for use in structural concrete which are manufactured from natural materials are: expanded clay, shale, and slate. They are obtained by heating suitable raw materials in a rotary kiln to incipient fusion (temperature of 1000 to 1200 °C) when expansion of the material takes place due to the generation of gases which become entrapped in a viscous pyroplastic mass. This porous structure is retained on cooling so that the apparent specific gravity of the expanded material is much lower than before heating. Often, the raw material is reduced to the desired size before calcining, but crushing after

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expansion may also be applied. Expansion can also be achieved by the use of a sinter strand. Here, the moistened material (which either contains carbonaceous material or is mixed with fuel) is carried by a travelling grate under burners so that burning gradually penetrates the full depth of the bed of the material. Its viscosity is such that gases are entrapped. As with the rotary kiln, the cooled mass is crushed. Alternatively, initially pelletized clay or pulverized shale can be used.

The use of pelletized material produces particles with a smooth shell or coating (50 to 100 μm (0.002 to 0.004 in.) thick) over a cellular interior. These nearly spherical particles with a semi-impervious glaze have a much lower water absorption than uncoated particles. Coated particles are easier to handle and to mix, and produce concrete of higher workability.

Oven-dry particles of lightweight aggregate of the expanded clay or shale variety have an apparent specific gravity in the range of 1.2 to 1.5 for coarse aggregate and between 1.3 and 1.7 for

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fine aggregate. The bulk density of the aggregate is 650 to 900 kg/m^3 (40 to 55 lb/ft^3) when made by the sinter strand process, and 300 to 650 kg/m^3 (20 to 40 lb/ft^3) when made in a rotary kiln. These aggregates produce concrete with a density usually within the range of 1400 to 1800 kg/m^3 (85 to 110 lb/ft^3), although values as low as 800 kg/m^3 (50 lb/ft^3) have been obtained. Concrete made with expanded shale and clay aggregates generally has a higher strength than when other lightweight aggregates are used.

There exist some other lightweight aggregates manufactured from natural materials which produce low-density concrete: vermiculite and perlite; the latter can sometimes be used to make moderate strength concrete. These aggregates are specified in ASTM C 332-09.

Vermiculite is a material with a platy structure, somewhat similar to that of mica. When heated to a temperature of 650 to 1000 °C, vermiculite expands to several, or even as many as

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30, times its original volume by exfoliation of its thin plates. As a result, the bulk density of exfoliated vermiculite is only 60 to 130 kg/m^3 (4 to 8 lb/ft^3) and the concrete made with it is of very low strength and exhibits high shrinkage but is an excellent heat insulator.

Perlite is a glassy volcanic rock which, when heated rapidly to the point of incipient fusion (900 to 1100 °C), expands due to the evolution of steam and forms a cellular material with a bulk density of 30 to 240 kg/m^3 (2 to 15 lb/ft^3). Concrete made with perlite has a very low strength, a very high shrinkage, and is used primarily for insulating purposes. An advantage of such concrete is that it is fast setting and can be finished rapidly.

The main industrial by-products used to manufacture lightweight aggregate are fly ash and blastfurnace slag. The very fine fly ash is moistened, made into pellets and then sintered in a suitable furnace: the small amount of unburnt fuel present in the ash will usually maintain this process without an addition of fuel. The sintered

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nodules provide a very good, rounded aggregate, with a bulk density of about 1000 kg/m^3 (60 lb/ft^3); the fine particles have a bulk density of about 1200 kg/m^3 (75 lb/ft^3).

Expanded blastfurnace slag is produced in three ways. In one, a limited amount of water in the form of a spray comes into contact with the molten slag when being discharged from the furnace (in the production of pig iron). Steam is generated and it bloats the still plastic slag, so that the slag hardens in a porous form, rather similar to pumice. This is the water-jet process. In the machine process, the molten slag is rapidly agitated with a controlled amount of water. Steam is entrapped and there is also some formation of gases due to chemical reactions of some slag constituents with water vapour. With both these methods, crushing of the expanded slag is required. A more modern method is to produce pelletized expanded blastfurnace slag. Here, molten slag containing gas bubbles is projected through a water spray so as to form pellets. These are rounded in shape

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and have a smooth coated (or sealed) surface. However, crushing (which destroys the coating) has to be used to obtain fine particles. The bulk density of pelletized blastfurnace slag is typically 850 kg/m^3 (55 lb/ft^3). Appropriate production control ensures the formation of crystalline material, which is preferable for use as aggregate; this is in contrast to blastfurnace slag pellets used in the manufacture of blastfurnace cement (see p. 79).

Only aggregates produced by expansion of clay, shale, slate, fly ash, or blastfurnace slag can be used to make structural concrete.

While each proprietary brand of lightweight aggregate has consistent properties, the variation between different lightweight aggregates is large, but some generalizations are possible. Of particular importance is the fact that sealed-surface (coated) particles of good-quality expanded clay aggregate have a 30-minute absorption of just over one-half of the absorption of the same material with the coating removed by cutting the

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particles.[13.110] However, some aggregates have a coating which is less effective.

For the manufacture of masonry blocks, it is possible to use, in addition to the materials considered so far, also aggregates consisting of end products of combustion of coal and coke. Consideration of these is included in ASTM C 331-05.

Clinker aggregate, known also as *cinders*, is made from well-burnt residue of industrial high-temperature furnaces, fused or sintered into lumps. It is important that the clinker be reasonably free from unburnt coal which may undergo expansion in the concrete, thus causing unsoundness, and from sulfates.

Iron or pyrites in the clinker may result in staining of surfaces, and should, therefore, be removed. Unsoundness due to hard-burnt lime can be avoided by allowing the clinker to stand wet for a period of several weeks: the lime will become slaked and will not expand in the concrete. The use of clinker aggregate in reinforced concrete is not recommended.

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Breeze is the name given to a material similar to clinker but more lightly sintered and less well burnt. There is no clear-cut demarcation between breeze and clinker.

When clinker is used as both fine and coarse aggregate, concrete with a density of about 1100 to 1400 kg/m^3 (70 to 85 lb/ft^3) is obtained, but often natural sand is used in order to improve the workability of the mix: the density of the resulting concrete is then 1750 to 1850 kg/m^3 (110 to 115 lb/ft^3).

Processed domestic garbage and wastewater sludge, mixed with clay and other materials, can be pelletized and fired in a rotary kiln to make lightweight aggregate;[13.117] however, the stage of regular and economic production has not yet been reached.

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Requirements for aggregates for structural concrete

Requirements for lightweight aggregate are given in ASTM C 330-09 and BS 3797 : 1990 (withdrawn) and replaced by BS EN 13055-1 : 2002. The latter standard covers also concrete for masonry units. The Standards place a limit on loss on ignition (5 per cent in ASTM and 4 per cent in BS) and, in the case of BS 3797 : 1970 (withdrawn), on sulfate content: 1 per cent expressed as SO_3 by mass. Some of the grading requirements of these Standards are given in Tables 13.7, 13.8, and 13.9. The replacement of BS 3797, which is BS EN 13055-1 : 2002 is not prescriptive.

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Table 13.7. Grading Requirements of Light-weight Coarse Aggregate According to ASTM C 330-05

Sieve size		Percentage by mass passing sieves			
mm	in.	Nominal size of graded aggregate			
		1 in. to No. 4	$\frac{3}{4}$ in. to No. 4	$\frac{1}{2}$ in. to No. 4	$\frac{3}{8}$ in. to No. 8
25.0	1	95–100	100	—	—
19.0	$\frac{3}{4}$	—	90–100	100	—
12.5	$\frac{1}{2}$	25–60	—	90–100	100
9.5	$\frac{3}{8}$	—	10–50	40–80	80–100
4.75	No. 4	0–10	0–15	0–20	5–40
2.36	No. 8	—	—	0–10	0–20

Table 13.8. Grading Requirements for Light-weight Coarse Aggregate According to BS 3797 : 1990 (withdrawn)*

Sieve size		Percentage by mass passing sieves		
mm	in.	Nominal size of graded aggregate		
		20 to 5 mm	14 to 5 mm	10 to 2.36 mm
20.0	$\frac{3}{4}$	95–100	100	—
14.0	$\frac{1}{2}$	—	95–100	100
10.0	$\frac{3}{8}$	30–60	50–95	85–100
6.3	$\frac{1}{4}$	—	—	—
5.0	No. 4	0–10	0–15	15–50
2.36	No. 8	—	—	0–15

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[*] The replacement BS EN 13055-1 : 2002 is not prescriptive.

Table 13.9. Grading Requirements for Lightweight Fine Aggregate According to BS 3797 : 1990 (withdrawn)[*] and ASTM C 330-05

Sieve size		*Percentage by mass passing sieves*		
BS	*ASTM*	*British*		*American*
		Grade L1	*Grade L2*	
10.0 mm	$\frac{3}{8}$ in.	100	100	100
5.0 mm	No. 4	90–100	90–100	85–100
2.36 mm	No. 8	55–100	60–100	—
1.18 mm	No. 16	35–90	40–80	40–80
600 μm	—	20–60	30–60	—
300 μm	No. 50	10–30	25–40	10–35
150 μm	No. 100	5–19	20–35	5–25

[*] The replacement BS EN 13055-1 : 2002 is not prescriptive.

To avoid confusion, it should be mentioned that BS EN 12620 : 2002 covers air-cooled slag, which is non-expanded.

It is useful to note that lightweight aggregates for use in structural concrete, regardless of their

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origin, are manufactured products and they are, therefore, generally more uniform than natural aggregate. Consequently, lightweight aggregate can be used in producing structural concrete of consistent quality.

Reference was made, on several occasions, to the bulk density of lightweight aggregate. This should be properly defined. *Bulk density* of lightweight aggregate, also known as *unit weight*, is the mass of the aggregate filling a unit volume; the method of filling must be clearly specified. The bulk density is influenced by the degree of packing of the aggregate particles, which depends on their grading. However, even when the particles are of nominally the same size, their shape influences the degree of packing when a fixed method of filling the test container is used. All this is no different from the situation obtaining when normal weight aggregate is used except that the lightweight aggregate is not compacted when its bulk density is being determined. ASTM C 330-09 prescribes the use of the shov-

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elling procedure of ASTM C 29-09 whereas BS 3797 : 1990 (withdrawn) specifies explicitly that there will be no tamping and that no shock will be applied to the container.

Lightweight aggregates have one important feature which is absent in normal weight aggregate and which is of importance with respect to the selection of mix proportions and the associated properties of the resultant concrete. This feature is the ability of the lightweight aggregate to absorb large quantities of water and also to allow a limited ingress of fresh cement paste into the open (surface) pores of the aggregate particles, especially the larger ones. When water is absorbed by the aggregate particles, their specific gravity becomes higher than the apparent specific gravity of oven-dry particles, and it is this higher specific gravity that is relevant to the density of concrete containing lightweight aggregate. The ability of lightweight aggregate to absorb a large amount of water has also other consequences which will be discussed later.

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Effects of water absorption by lightweight aggregate

The term ‘apparent specific gravity’ (see p. 126) applies to the individual particles and is based on their volume inclusive of the internal pores in them. A practical difficulty in calculating the apparent specific gravity lies in establishing the volume of the particles which is measured by fluid displacement. This displacement is affected by the penetration of the test fluid, which is commonly water, into the open pores on the surface of the aggregate particles and into connected pores within the particle. It should be added that knowing whether the pores penetrated by water will also be penetrated by the cement paste is of importance in the establishment of mix proportions. Various test methods prescribe the means of preventing excessive water penetration into the particle pores: spraying with a hydrophobic coating such as kerosene; dipping in hot paraffin; or immersion in water for 30 minutes prior to the measurement of the displacement. There are large

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differences in the measured specific gravity, depending on the test method used.[13.87]

Specific gravity of the aggregate particles in a saturated and surface-dry state is also difficult to determine because the presence of open pores on the surface does not make it possible to establish when this state has been achieved.[13.86]

The term ‘density’ needs careful qualification when applied to lightweight aggregate concrete. The density of freshly mixed concrete can be readily determined; this is *fresh density*. However, upon drying in air under ambient conditions, moisture is lost until a quasi-equilibrium is reached: the concrete then has an *air-dry density*. If the concrete is dried at 105 °C (221 °F), the *oven-dry density* is reached. Whereas similar changes take place also in normal weight concrete, in the case of lightweight aggregate concrete, the differences between the three densities are much larger and more significant for the behaviour of concrete.

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Methods of determining the fresh density and the air-dry density of the fresh concrete are given in ASTM C 567-05a. The air-dry density is established in hygrometric equilibrium with air at a relative humidity of 50 per cent and a temperature of 23 °C (73 °F).

To present a full picture of the absorption of water by lightweight aggregate, it can be added that, unless the aggregate has been fully saturated prior to mixing, its pores do not become entirely full of water. Thus, the fresh density of concrete is lower than the theoretical saturated density. The latter exceeds the former typically by 100 to 120 kg/m^3 (6 to 7 lb/ft^3).[13.84] Saturation is not readily achieved in practice, in consequence of the low permeability of lightweight aggregate concrete except when water pressure is applied.[13.84]

Because of the difficulties in establishing when an equilibrium air-dry density has been reached, it is often recommended[13.84] that experimentally determined fresh density be relied upon.

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The value of air-dry density can then be calculated by subtracting the mass of water lost to the air. This mass is typically between 100 and 200 kg/m^3 (6 and 12 lb/ft^3) for all-lightweight aggregate concrete, and between 50 and 150 kg/m^3 (3 to 9 lb/ft^3) when normal weight fine aggretate is used.[13.84] The equilibrium density, which is of interest in the calculation of self-weight of the concrete, is about 50 kg/m^3 (3 lb/ft^3) above the oven-dry density.[13.143] It should be remembered that there can be considerable departure from the above values: this depends on the pore system of the lightweight aggregate used, on the volume/surface ratio of the concrete element, and on the conditions of exposure.

The high absorption capacity of lightweight aggregate is of relevance also at the mixing stage. When a certain amount of water is batched, the amount of water available for wetting the cement and for reaction with it depends on how much of the water is absorbed by the lightweight aggreg-

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ate. This varies widely, ranging from zero when the lightweight aggregate has been pre-soaked for a considerable time, to a very large amount, depending on the type of aggregate, when the lightweight aggregate is oven-dry. In-between these two extremes, air-dry aggregate introduced into the concrete mixer is likely to absorb between 70 and 100 kg of water per cubic metre of concrete (120 to 170 lb/yd^3).[13.84]

The 24-hour absorption of lightweight aggregate ranges from 5 to 20 per cent by mass of dry aggregate[13.141] but, for good quality aggregate for use in structural concrete, it is usually not more than 15 per cent.

By comparison, the absorption of normal weight aggregate is usually less than 2 per cent[13.141] (see Table 3.11). On the other hand, fine normal weight aggregate may have a moisture content of 5 to 10 per cent, sometimes even more, but this water is on the surface of the aggregate particles. Consequently, this water forms part of the mix water and it is entirely available

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for hydration (see p. 132). From the preceding discussion, it can be seen that the absorbed water is irrelevant with respect to the water/cement ratio and to workability, but may have serious consequences for the resistance of concrete to freezing and thawing.

There is another important consequence of the absorption of water by lightweight aggregate: when hydration of cement lowers the relative humidity of the capillary pores in the hardened cement paste, the water in the aggregate migrates outwards into these capillaries. Enhanced hydration is thus possible. The situation could be termed ‘internal moist curing’. This makes lightweight aggregate concrete less sensitive to inadequate moist curing.

From the preceding discussion, it can be seen that there is a considerable difficulty in determining the free water in the mix. If the aggregate is batched dry, the water required to fill the pores in the aggregate particles must be considered as being in excess of free water. The situation is com-

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plicated by the fact that absorption takes some time. The rate of absorption depends on whether the particles are coated and on the pore system within the particles, but the major part of the 30-minute absorption takes place in 2 minutes from wetting. The 30-minute absorption is more than one-half of the 24-hour absorption, and the proportion is even higher when the particles are not coated.[13.110]

One consequence of the rapidity of absorption of mix water is that, if lightweight aggregate is batched in an oven-dry condition, or nearly so, and, if the concrete is compacted before absorption by the dry lightweight aggregate has been completed, then voids caused by desiccation will develop in the concrete. Unless the concrete is revibrated, the strength will be adversely affected.[13.86]

Lightweight aggregate concrete

The preceding sections made it clear that lightweight aggregate concrete covers an extremely

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wide field: using appropriate materials and methods, the density of concrete can be varied between little over 300 and about 1850 kg/m^3 (20 to 115 lb/ft^3), and the corresponding strength range is between 0.3 and 70 MPa (50 to 10 000 psi), and sometimes even 90 MPa (13 000 psi). This wide range of composition is reflected in various properties of lightweight aggregate concrete.

Aspects of the fresh state

The water demand of lightweight aggregate concrete is strongly affected by the surface texture and shape of the aggregate particles. One important consequence of the large variation in the water demand of concretes made with different lightweight aggregates is that, to achieve a given required strength, there has to be a corresponding variation in the cement content: this way the water/cement ratio is maintained but, as already mentioned, the value of the actual water/cement ratio is not normally known.

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The rheological behaviour of lightweight aggregate concrete is somewhat different from normal weight concrete. Specifically, at the same slump, lightweight aggregate concrete exhibits better workability. Likewise, the compacting factor of lightweight aggregate concrete underestimates the workability because the force of gravity which compacts the concrete is reduced when the density of concrete is lower. However, because the Kelly ball penetration (see p. 197) does not depend on the action of gravity on the concrete, the value recorded in the Kelly ball test is not affected by the aggregate.[13.147] It should be noted that high slump may cause segregation, with the light large aggregate particles floating to the top. Likewise, prolonged vibration may lead to segregation much more readily than with normal weight aggregate.

The workability of mixes containing angular aggregate can be considerably improved by the inclusion of entrained air in the mix: the water demand is reduced, and so is the tendency to bleed-

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ing and segregation. The usual *total* air contents by volume are: for 20 mm ($\frac{3}{4}$ in.) maximum size of aggregate, 4 to 8 per cent; for 10 mm ($\frac{3}{8}$ in.) maximum size of aggregate, 5 to 9 per cent. Air content in excess of these values lowers the compressive strength by about 1 MPa (150 psi) for each additional percentage point of air.[13.141]

Partial replacement of lightweight fine aggregate by normal weight fine aggregate makes placing and compaction of concrete easier.[13.96] However, the density of the resulting concrete is increased depending on the proportion of fine aggregate replaced and on the relative values of the specific gravity of the two materials. Total replacement of lightweight fine aggregate by normal weight fine aggregate would increase the density of the concrete by 80 to 160 kg/m^3 (5 to 10 lb/ft^3).[13.143] The thermal conductivity of the concrete is also increased by the introduction of normal weight fine aggregate.

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It was mentioned earlier that the workability of lightweight aggregate concrete is very much affected by the fact that, depending on its degree of saturation, the aggregate may absorb more or less water from the fresh mix. The rate of absorption of mix water by the aggregate affects the rate of loss of slump. Measures appropriate to the given situation need to be taken, but it is important to remember that unplanned changes in the moisture condition of the aggregate would have serious consequences on slump and on the slump loss. Practical aspects of batching and mixing lightweight aggregate concrete are discussed in ACI 304.5R-91.[13.142]

Superplasticizers can be used with lightweight aggregate concrete, but usually they are included in the mix only when concrete is to be pumped. If the aggregate absorbs water during pumping, a dangerous loss of slump may occur. Using saturated aggregate obviates this problem: saturation can be achieved by vacuum soaking in a pressure vessel, followed by continuous water-spray-

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ing until mixing. However, this condition of the aggregate may reflect its resistance to freezing and thawing. To alleviate the problem in pumping, mixes with partial replacement of the fine aggregate by normal weight fine aggregate are generally used. The properties of mixes containing lightweight aggregate which are destined for pumping are discussed in ACI 213R-87.[13.141]

Strength of lightweight aggregate concrete

As pointed out earlier, there is an insuperable difficulty in determining the amount of free water in the mix with most lightweight aggregates. In consequence, the water/cement ratio based on free water in the mix cannot be established; the water/cement ratio based on total water is meaningless because the water absorbed by the aggregate does not affect the formation of capillary pores, which influence strength.

On the other hand, for a given aggregate, there is a broad relation between the cement content of

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concrete and its compressive strength; this is illustrated in Fig. 13.13.[13.111] Because cement has a much higher specific gravity than lightweight aggregate and than water, for any particular aggregate, strength increases with an increase in density but, depending on the type of aggregate, 20 MPa (3000 psi) concrete may require between 260 and 330 kg of cement per cubic metre of concrete (440 to 560 lb/yd^3); the corresponding range for 40 MPa (6000 psi) concrete is 420 to 500 kg/m^3 (700 to 840 lb/yd^3). Some values quoted in ACI 213R-87 are shown in Table 13.10 but these are not meant to be more than indicative. Higher compressive strengths require very high cement contents; for example a strength of 70 MPa (10 000 psi) may require a cementitious material content of 630 kg/m^3 (1050 lb/yd^3).

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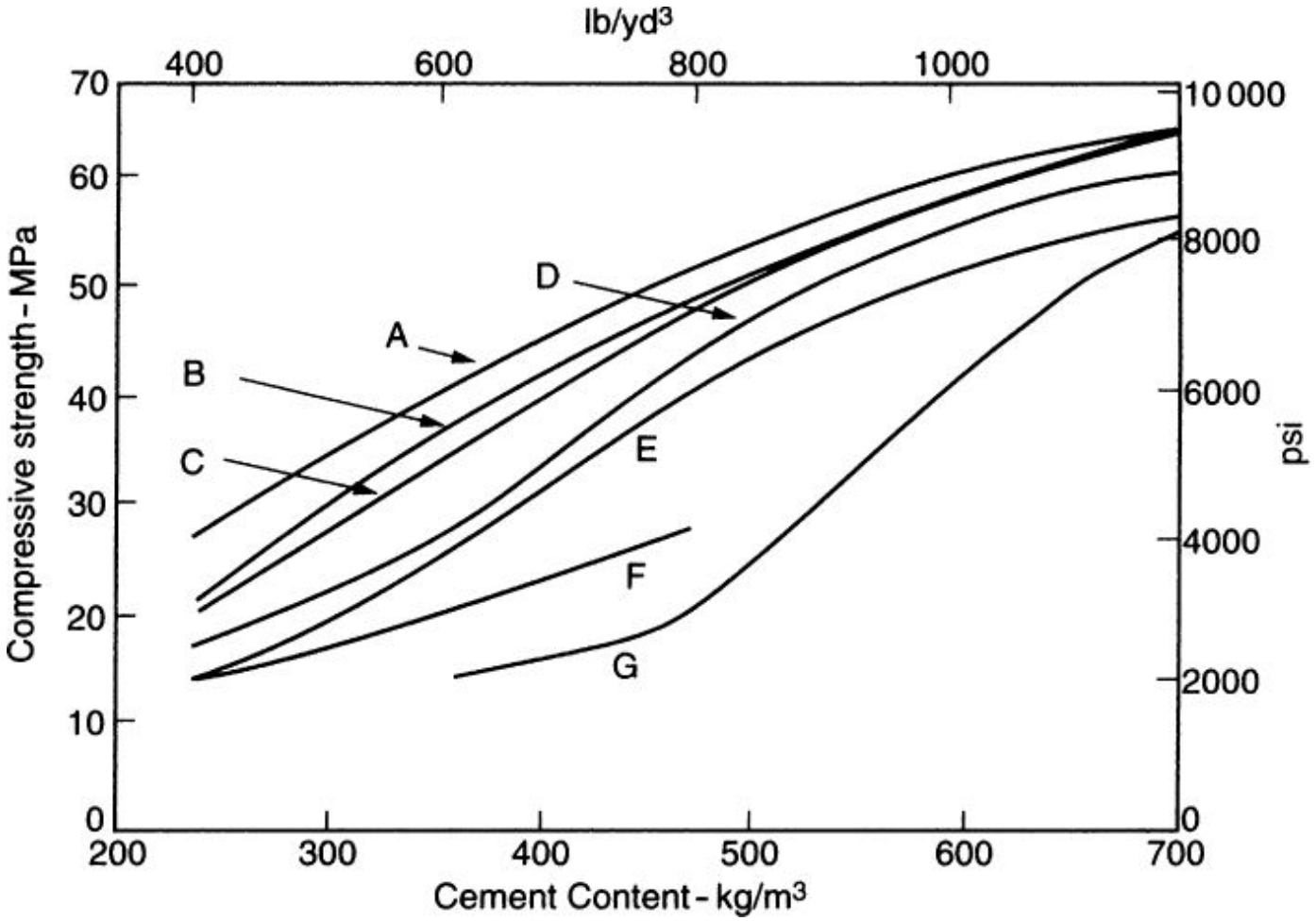


Fig. 13.13. Relation between 28-day compressive strength (measured on cubes) and cement content for various lightweight aggregate concretes with a slump of 50 mm (2 in.) (based on ref 13.111): (*A*) sintered fly ash and normal weight fine aggregate; (*B*) pelletized blastfurnace slag and normal weight fine aggregate; (*C*) sintered fly ash; (*D*) sintered colliery shale; (*E*) expanded slate; (*F*) expanded clay and sand; (*G*) expanded slag

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Table 13.10. Approximate Relation Between Strength of Lightweight Aggregate Concrete and Cement Content[13.141]

Compressive strength of standard cylinders		*Cement content*			
		With lightweight *fine aggregate*		With normal weight *fine aggregate*	
MPa	*psi*	*kg/m³*	*lb/yd³*	*kg/m³*	*lb/yd³*
17	2500	240–300	400–510	240–300	400–510
21	3000	260–330	440–560	250–330	420–560
28	4000	310–390	530–660	290–390	490–660
34	5000	370–450	630–750	360–450	600–750
41	6000	440–500	740–840	420–500	700–840

As in normal weight concrete, silica fume improves the strength development of lightweight aggregate concrete. Other cementitious materials can also be incorporated in lightweight aggregate concrete.

In general terms, for the same strength of concrete, the cement content in a lightweight aggregate mix is higher than in normal weight concrete; at high strengths, the additional cement content can exceed 50 per cent. A high cement content of lightweight aggregate concrete means that it has a low, albeit unknown, water/cement ratio, so that

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the strength of the matrix is high. The lightweight particles of coarse aggregate are relatively weak and their strength may be the limiting factor in the strength of concrete: splitting of coarse aggregate particles in a direction normal to the applied load takes place.[13.104] However, there is no general relation between the strength of the aggregate as such and the strength of the concrete made with the given aggregate.

The limitation on the strength of lightweight aggregate concrete imposed by the strength of the coarse aggregate particles can be alleviated by the use of a smaller maximum size of aggregate. The explanation of this behaviour lies in the fact that in the comminution of large aggregate particles fracture takes place through the largest pores which are thus eliminated. This has a positive influence on the strength of aggregate, but also increases its apparent specific gravity and bulk density; this can be seen in Table 13.6.

In calculating mix proportions of concrete containing lightweight aggregate of different

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sizes, it is important to remember that the apparent specific gravity of lightweight fine particles is higher than that of lightweight coarse particles. This difference is even larger when normal weight fine aggregate is used. The conversion of volume occupied by the various particles into mass must take these differences into account.

Splitting tension strength tests usually show that failure takes place through the coarse aggregate particles, thus confirming good bond of the aggregate.[13.96] An example of the relation between the splitting tensile strength and compressive strength for concretes made with pelletized blastfurnace slag aggregate, and cured under different conditions, is shown in Fig. 13.14. This figure shows also a plot of the relevant expression recommended by FTP[13.115] which is:

$$f_t = 0.23 f_{cu}^{0.67}$$

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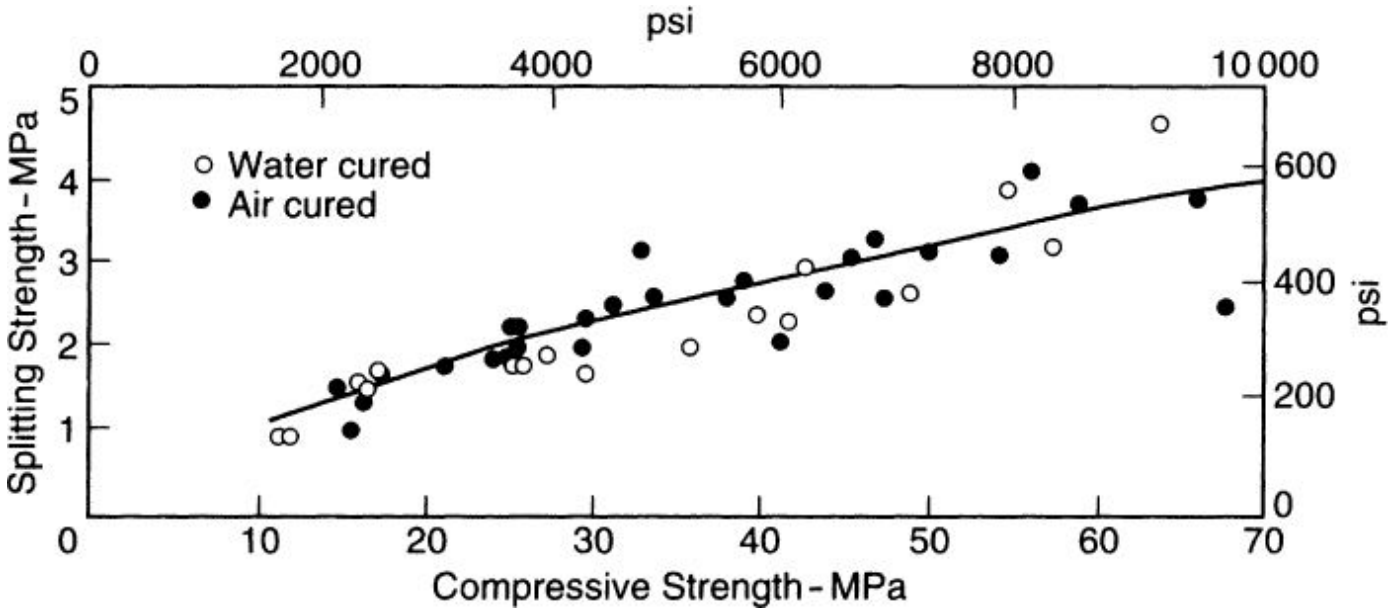


Fig. 13.14. Relation between splitting tensile strength and compressive strength of concrete made with pelletized blastfurnace slag[13.96]

where f_t is splitting strength and f_{cu} is compressive strength measured on cubes, both in MPa.

High-strength lightweight aggregate concretes with some normal weight fine aggregate, with compressive strengths between 50 and 90 MPa (7000 to 13 000 psi), were found to have a flexural strength lower by up to 2 MPa (300 psi) than normal weight concretes with the same compressive strength.[13.110] In the case of the splitting

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strength, the difference between the two concretes was about 1 MPa (150 psi).

The fatigue strength of lightweight aggregate concrete was found to be at least as good as that of normal weight concrete of similar strength.[13.100]

Lightweight aggregate–matrix bond

An important feature of lightweight aggregate concrete is the good bond between the aggregate and the surrounding hydrated cement paste. This is the consequence of several factors. First, the rough surface texture of many lightweight aggregates is conducive to a good mechanical interlocking between the two materials. In fact, there is often some penetration of cement paste into the open surface pores in the coarse aggregate particles. Second, the moduli of elasticity of the lightweight aggregate particles and of the hardened cement paste do not differ much from one another. Consequently, no differential stresses between the two materials are induced,

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either by the applied load, or by thermal or hygrometric changes. Third, the water absorbed by the aggregate at the time of mixing becomes, with time, available for the hydration of the hitherto unhydrated remnants of cement. As most of this additional hydration occurs in the aggregate–cement paste interface zone, the bond between the aggregate and the matrix becomes stronger.

Although lightweight aggregates originating from fly ash or blastfurnace slag could be thought to be potentially pozzolanic, only very limited pozzolanic reaction at the interface of the aggregate particles and the cement paste has been observed.[13.105] The explanation of this absence of reactivity of the aggregate lies in the very high temperature (up to 1200 °C) to which it was subjected during manufacture, so that crystallization of silica and alumina has taken place;[13.105] reactive amorphous material is absent.

It may be instructive to consider, in a more general manner, the bond between the aggregate

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and the surrounding hydrated cement paste in three categories of concrete – ordinary normal weight concrete, high performance concrete, and lightweight aggregate concrete – in so far as the bond is affected by the moduli of elasticity of the aggregate and of the hydrated cement paste. In ordinary normal weight concrete, the modulus of elasticity of the typical cement paste is generally much lower than the modulus of elasticity of the aggregate particles. In high performance concrete, the hydrated cement paste has a very much higher modulus of elasticity so that the *difference* between it and the modulus of elasticity of the aggregate is much smaller. In lightweight aggregate concrete, the modulus of elasticity of the aggregate is much lower than the modulus of elasticity of normal weight aggregate; consequently, the *difference* between the moduli of elasticity of the lightweight aggregate and of the hydrated cement paste is small.

It can be seen thus that high performance concrete and lightweight aggregate concrete have the

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common feature of the absence of a large difference between the moduli of elasticity of the aggregate and of the hydrated cement paste. This is conducive to good bond between these two materials and to a good composite behaviour of the concrete. Ordinary normal weight concrete is least satisfactory in this respect.

In this connection, Bremner and Holm[13.104] observed that air entrainment lowers the modulus of elasticity of mortar, thus bringing it down nearer to the modulus of elasticity of lightweight aggregate. This reduction in the difference in the moduli is conducive to a better stress transfer between the aggregate particles and the matrix.

Elastic properties of lightweight aggregate concrete

One effect of the very good bond between lightweight aggregate and the matrix is the absence of early development of bond microcracking (cf. p. 300); a consequence of this is that the stress–strain relation is linear, often up to a stress

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as high as 90 per cent of the ultimate strength.[13.106] This is especially so in lightweight aggregate concretes which contain also silica fume and have a 28-day strength of about 90 MPa (13 000 psi).[13.106]

Examples of the stress–strain relation for lightweight aggregate concretes are shown in Fig. 13.15; it can be seen that, when all the aggregate is of the lightweight variety, the descending part of the curve is very steep.[13.102] The replacement of lightweight fine aggregate by normal weight fine aggregate results in a less steep descending part of the curve, but it increases the slope of the ascending part; the latter is due to the higher modulus of elasticity of the normal weight fine particles of aggregate.

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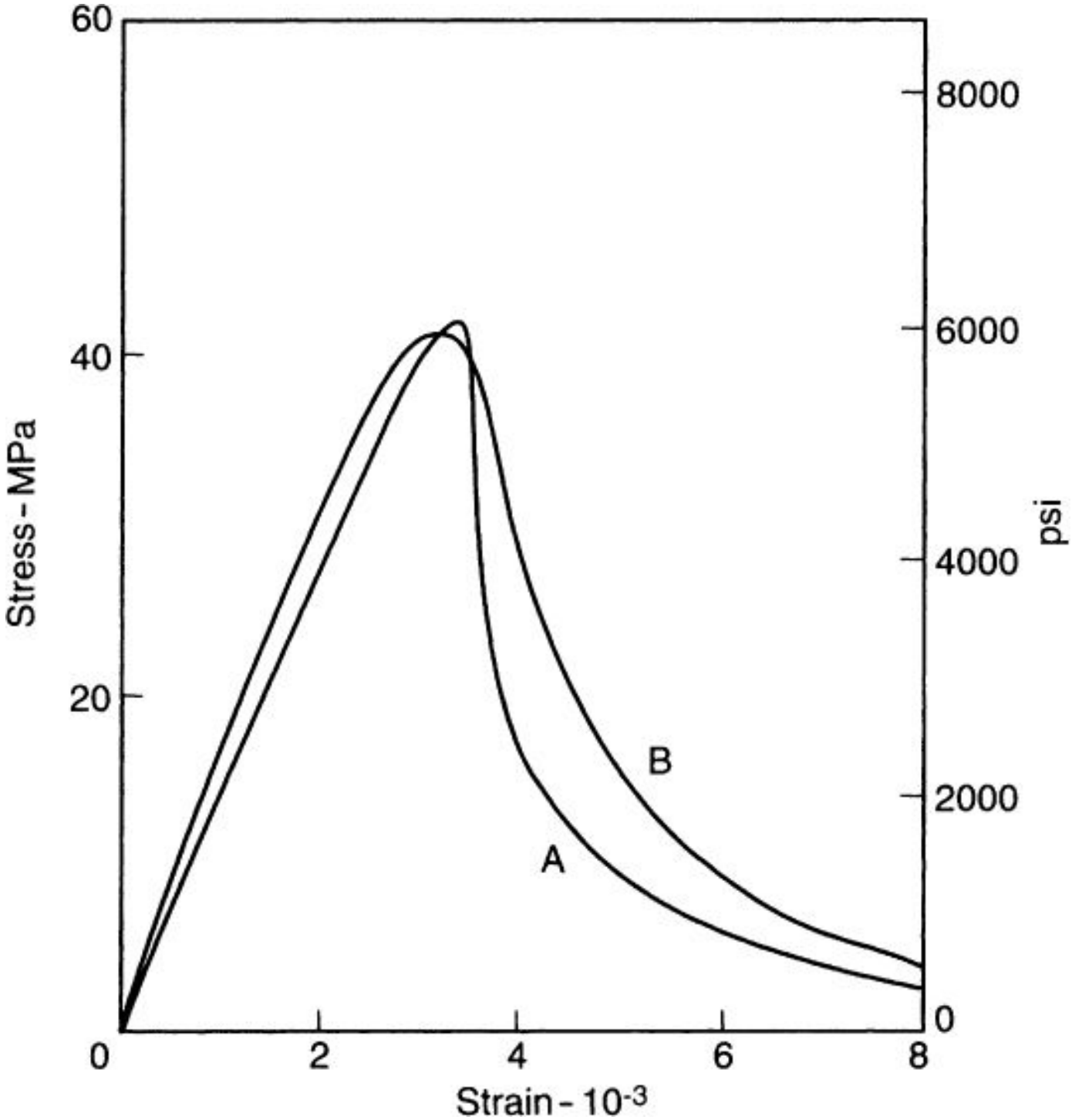


Fig. 13.15. Stress–strain curves for light-weight aggregate concretes made with expan-ded clay: (*A*) all lightweight aggregate; (*B*) normal weight fine aggregate[13.102]

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The modulus of elasticity of lightweight aggregate concrete can be expressed as a function of its compressive strength (see p. 706). However, because of the good bond with the aggregate particles, lightweight aggregate concrete exhibits particularly good composite action so that the elastic properties of the aggregate have a greater influence on the modulus of elasticity of concrete than is the case with normal weight concrete. Because the elastic properties of aggregate are affected by its void content, and therefore by its apparent specific gravity, the modulus of elasticity of lightweight aggregate concrete can be expressed as a function of the density of the concrete, as well as of its compressive strength.

For strengths up to 41 MPa (6000 psi), ACI 318-08[13.116] expresses the modulus of elasticity of concrete, E_c, in GPa as

$$E_c = 43 \times 10^{-6} \rho^{1.5} \sqrt{f'_c}$$

where

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f'_c = standard cylinder strength in MPa, and

ρ = density of concrete in kg/m^3.

This expression is meant to be valid for values of density between 1440 and 2480 kg/m^3 (90 and 155 lb/ft^3) but the actual modulus of elasticity may well deviate from the calculated value by up to 20 per cent.[13.141]

As far as lightweight aggregate concrete with a compressive strength in the range of 60 to 100 MPa (9000 to 14 000 psi) is concerned, the relation of the modulus to the compressive strength seems to be best described by a Norwegian standard expression reported by Zhang and Gjørv[13.106] as:

$$E_c = 9.5 f_c^{0.3} \times \left(\frac{\rho}{2400} \right)^{1.5}$$

where

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E_c = modulus of elasticity in GPa

f_c = compressive strength of 100 by 200 mm cylinders in MPa, and

ρ = density of the concrete in kg/m^3.

The values of the modulus of elasticity of concrete made with expanded clay or sintered fly ash have been found to lie between 18 and 26 GPa, that is, they are typically 12 GPa lower than for normal weight concrete in the same strength range of 50 to 90 MPa.[13.106]

It can be noted that the lower modulus of elasticity of lightweight aggregate concrete allows the development of a higher ultimate strain, compared with normal weight concrete of the same strength:[13.143] values of 3.3×10^{-3} to 4.6×10^{-3} have been reported.[13.106]

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Durability of lightweight aggregate concrete

There are no serious adverse effects on durability of using lightweight aggregate, except when saturated aggregate is subjected to freezing and thawing, as discussed later in this section.

Because the pore system in lightweight aggregate is generally discontinuous, the porosity of the aggregate particles themselves does not influence the permeability of concrete, which is controlled by the permeability of the hardened cement paste.[13.112] Nevertheless, the permeability of the concrete is reduced when normal weight fine aggregate is used to replace a part of the lightweight fine aggregate;[13.112] the probable reason for this is that, in the former case, the water/cement ratio is lower.

The low permeability of lightweight aggregate concrete is the result of several factors: the water/cement ratio of the cement paste is low; the quality of the interface zone around the aggregate is

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high so that easier flow paths around the aggregate are absent; and the compatibility of the moduli of elasticity of the aggregate particles and of the matrix means that little microcracking develops under load or in consequence of temperature variation. Moreover, the supply of water from the aggregate enables continuing hydration of cement to take place with a consequent reduction in permeability.

However, if the lightweight aggregate is saturated prior to mixing, for example to facilitate pumping, there is a risk of failure under conditions of alternating freezing and thawing unless the concrete has been able sufficiently to dry out prior to exposure to freezing.[13.109] In any case, air entrainment is required as with normal weight concrete.

Susceptibility of lightweight aggregate concrete to damage upon exposure to a very low temperature (–156 °C) depends on the properties of the hydrated cement paste in the same manner as in the case of normal weight concrete. It is only

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if the aggregate particles themselves are saturated that they can be the seat of damage: their expansion on freezing may destroy the bond with the surrounding matrix.[13.158]

With respect to carbonation, the voids in the lightweight aggregate facilitate the diffusion of CO_2, and it is often thought that additional cover to reinforcement is required. Nevertheless, little evidence of corrosion of reinforcing steel arising from carbonation in good quality lightweight aggregate concrete has been reported.[13.140]

No evidence of alkali–silica reaction in lightweight aggregate concrete has been recorded.[13.143]

Although lightweight aggregate particles are hard, and therefore resistant to abrasion, the open pores on the surface of the aggregate mean that, once the aggregate has become exposed, the contact surface is reduced compared with non-porous aggregate. On balance, the abrasion resistance of lightweight aggregate concrete may, therefore, be

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reduced in comparison with that of normal weight concrete of similar strength.

Concretes made with lightweight aggregate exhibit a higher moisture movement than is the case with normal weight concrete. They have a high initial drying shrinkage, about 5 to 40 per cent higher than ordinary concrete, but the total shrinkage with some lightweight aggregates may be even higher; the concretes made with expanded clay and shale and with expanded slag are in the lower shrinkage range. In view of the comparatively low tensile strength of lightweight aggregate concrete, there is a danger of shrinkage cracking, although some compensation is afforded by a lower modulus of elasticity and a greater extensibility of lightweight aggregate concrete.

As far as creep of lightweight aggregate concrete is concerned, an allowance must be made for the lower modulus of elasticity of the lightweight aggregate which restrains the creep of the hydrated cement paste. Occasionally, conflicting

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test data on creep of lightweight aggregate concrete have been reported in so far as the influence of drying on creep is concerned.[13.103] It is likely that the internal movement of moisture from the aggregate particles into the surrounding hydrated cement paste affects the development of drying creep, but a quantitative assessment of this effect is not available.

The *sound absorption* of lightweight concrete can be rated as good because the air-borne sound energy becomes converted into heat in the minute voids in the aggregate, so that the absorption coefficient of sound is about twice that for ordinary concrete. A rendered surface, however, would offer a much higher reflection of sound. Lightweight aggregate concrete does not possess particularly good *sound insulation* properties because this insulation is better the higher the density of the material (see p. 353).

The benefits of the combination of a lower coefficient of thermal expansion and a lower thermal conductivity, possessed by lightweight

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aggregate concrete, can be exploited in situations when the concrete surface is exposed to a highly local large rise in temperature, e.g. in a pavement used by vertical take-off aircraft.[13.108] The local expansion on heating, which is restrained by the surrounding cool concrete, is lower when lightweight aggregate concrete is used. This, coupled with a lower modulus of elasticity of lightweight aggregate concrete, results in a lower stress than would be the case with normal weight concrete. Consequently, local damage can be avoided.

Low thermal conductivity of lightweight aggregate concrete reduces the temperature rise of the embedded steel in the case of fire. The combination of a low thermal conductivity and a low coefficient of thermal expansion is beneficial on exposure to fire. Moreover, the aggregate is stable at high temperatures, having itself been processed at temperatures in excess of 1100 °C.[13.143] Some data on the fire resistance of hollow masonry walls are given in Table 13.11.[13.148]

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Table 13.11. Estimated Fire Resistance of Hollow Masonry Walls[13.148]

Type of aggregate used	Minimum equivalent thickness for ratings of:							
	4 hours		3 hours		2 hours		1 hour	
	mm	in.	mm	in.	mm	in.	mm	in.
Expanded slag or pumice	119	4.7	102	4.0	81	3.2	53	2.1
Expanded clay or shale	145	5.7	122	4.8	96	3.8	66	2.6
Limestone, cinders or unexpanded slag	150	5.9	127	5.0	102	4.0	69	2.7
Calcareous gravel	157	6.2	135	5.3	107	4.2	71	2.8
Siliceous gravel	170	6.7	145	5.7	114	4.5	76	3.0

Thermal properties of lightweight aggregate concrete

Some typical values of the coefficient of thermal expansion of lightweight aggregate concrete are given in Table 13.12. From a comparison with Fig. 8.11, it can be seen that lightweight aggregate concrete has generally a lower thermal expansion than normal weight concrete. This can produce some problems when the lightweight and normal weight concretes are used side by side. It can be noted that the low thermal expansion of lightweight aggregate concrete reduces the tend-

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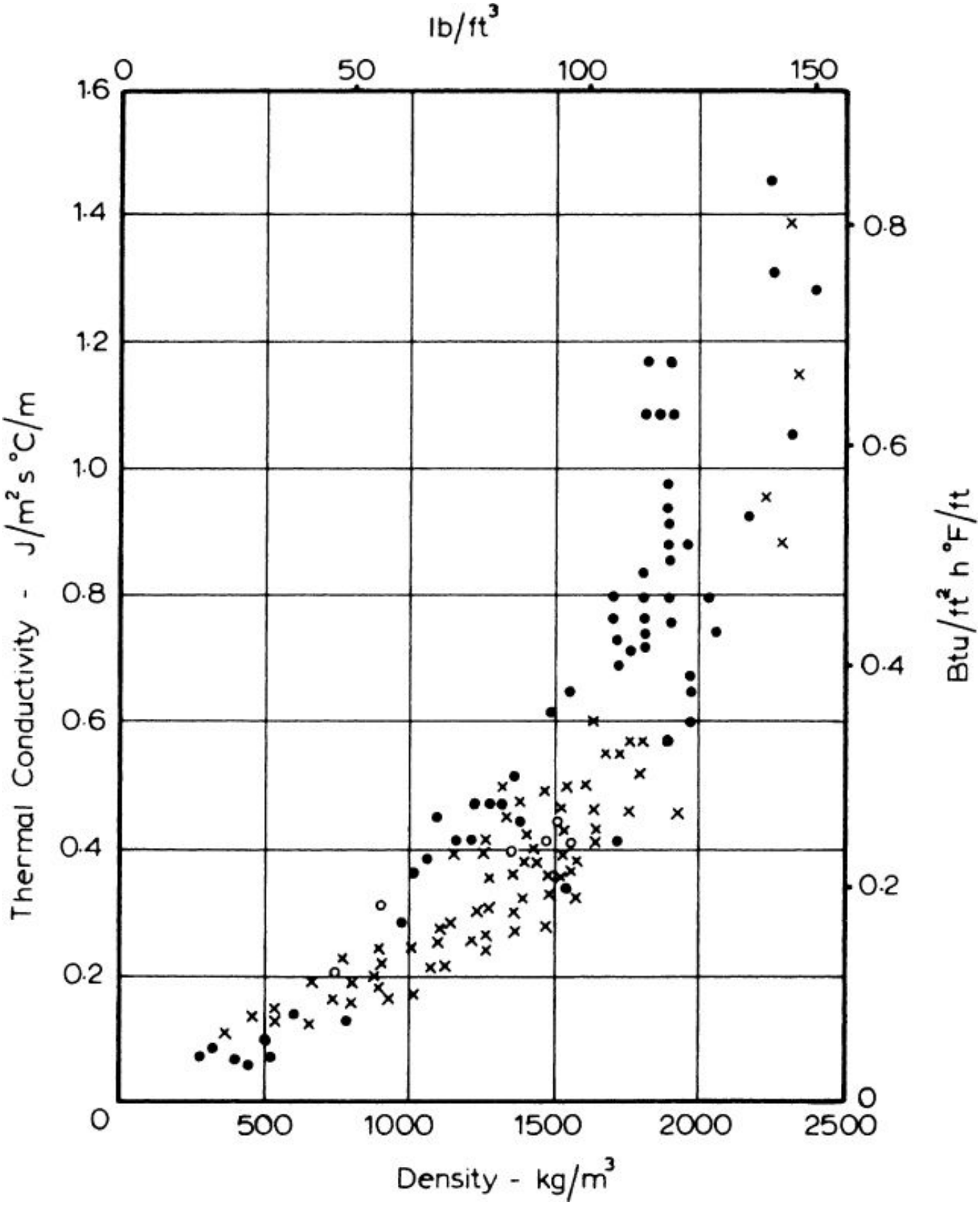
ency to warping (curling) or buckling when the two faces of a concrete element are exposed to different temperatures.

Table 13.12. Coefficient of Thermal Expansion of Concretes made with Lightweight Aggregate[13.148,13.149]

Type of aggregate used	*Linear coefficient of thermal expansion (determined over the range of −22 to 52 °C (−7 to 125 °F))*	
	10^{-6} per °C	*10^{-6} per °F*
Pumice	9.4–10.8	5.2–6.0
Perlite	7.6–11.0	4.2–6.1
Vermiculite	8.3–14.2	4.6–7.9
Cinders	about 3.8	about 2.1
Expanded shale	6.5–8.1	3.6–4.5
Expanded slag	7.0–11.2	3.9–6.2

Some data on thermal conductivity of lightweight aggregate concrete in an oven-dry condition are shown in Fig. 13.16.[13.150] Moisture absorbed by the concrete significantly increases its thermal conductivity.[13.141]

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Fig. 13.16. Thermal conductivity of light-weight aggregate concretes of various types[13.150]

It is worth noting that, in mass pours of light-weight aggregate concrete, the low thermal conductivity results in a reduced loss of heat to the ambient medium.

Cellular concrete

In the initial classification of lightweight concrete it was said that one method of reducing the density of concrete relies on the introduction of stable voids within the hardened cement paste or mortar. The voids can be produced by gas or by air; hence, the names *gas concrete* and *aerated concrete*. Because the air is introduced by a foaming agent, the term *foamed concrete* is also used. Strictly speaking, the term 'concrete' is inappropriate because no coarse aggregate is present.

The introduction of gas is achieved usually by the use of finely divided aluminium powder,

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at the rate of about 0.2 per cent by mass of cement. The reaction of the powder with a hydroxide of calcium or alkalis from the cement, liberates bubbles of hydrogen. The bubbles expand the cement paste or the mortar, which must have such consistency as to prevent their escape.

It can be mentioned in passing that aluminium powder is also used in grout for post-tensioned concrete in order to ensure complete filling of the cavity by the grout expanding in a confined space.

Air bubbles can be produced in the mix either by a pre-formed foam (made in a special foam generator) introduced into the mixer together with the cement, water and fine aggregate, or alternatively by mixing a foam concentrate together with the other mix ingredients in a high-shear mixer. In either case, the foam cells must have 'walls' which remain stable during mixing, transporting (which may include pumping) and placing of the fresh concrete. The cells, or bubbles,

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are discrete and range in size between 0.1 and 1 mm (0.004 and 0.04 in.).

Cellular concrete is free-flowing and can be easily pumped and placed without compaction. The material can be used for floors, trench fills, roof insulation and other insulating purposes, as well as to make masonry units.

Cellular concrete may or may not contain aggregate, the latter generally being the case with concrete required for thermal insulation when an oven-dry density of 300 kg/m^3 (20 lb/ft^3), and exceptionally as low as 200 kg/m^3 (12 lb/ft^3), can be obtained. When fine aggregate, either normal weight or lightweight, is included in the mix, the as-placed density lies between 800 and 2080 kg/m^3 (50 and 130 lb/ft^3).[13.144] Considerable care is required in using values of density because this is greatly affected by the moisture condition of the concrete. The air-dry density is relevant to conditions in service, which of course may be different from case to case. As an approximation,

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the air-dry density is 80 kg/m^3 (5 lb/ft^3) lower than the as-placed density. The lowest value of density is the oven-dry density, which is of interest in the determination of thermal conductivity of the given cellular concrete. The oven-dry density can be calculated on the assumption that the mass of a unit volume of cellular concrete is the sum of the mass of the aggregate (if any), the mass of the cement, and the mass of water chemically combined with the cement, assumed to represent 20 per cent of the mass of the cement.

As in other lightweight concretes, strength varies in proportion to density, and so does the thermal conductivity. Hoff[13.151] suggested that the strength of cellular concrete can be expressed as a function of the void content taken as the sum of the induced voids and the volume of evaporable water. Thus, the strength of moist-cured cellular concrete is governed by the total volume of voids in the concrete; that is, the strength is influenced both by the water/cement ratio of the mix and by the volume of the induced voids.[13.145]

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However, strength may not be of paramount importance, thermal properties being the criterion for the use of cellular concrete. Typical properties of cellular concrete used in the United Kingdom are shown in Table 13.13;[13.146] higher strengths have been reported in the United States.[13.144] The modulus of elasticity of cellular concrete is generally between 1.7 and 3.5 GPa (0.25×10^6 and 0.5×10^6 psi).

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Table 13.13. Guidance Data on Cellular Concrete (based on ref. 13.146)

Cement content				
kg/m³	300	320	360	400
lb/yd³	505	540	610	670
As-placed density				
kg/m³	500	900	1300	1700
lb/ft³	31	56	81	106
Oven-dry density				
kg/m³	360	760	1180	1550
lb/ft³	22	47	74	97
Fine aggregate content				
kg/m³	0	420	780	1130
lb/yd³	0	710	1320	1900
Air content				
per cent	78	62	45	28
Compressive strength				
MPa	1	2	5	10
psi	150	290	730	1450
Thermal conductivity				
Jm/m²s °C	0.1	0.2	0.4	0.6
Btu/ft²h °F/ft	0.06	0.12	0.23	0.29

Cellular concrete exhibits high shrinkage, ranging from 700×10^{-6} for cellular concrete with an oven-dry density of 1600 kg/m^3 to 3000×10^{-6} when the oven-dry density is 400 kg/m^3.[13.146] The moisture movement is also high. The usual range of the coefficient of permeability is 10^{-6} to 10^{-10} m/s.[13.144] However, problems with mois-

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ture in buildings do not generally arise because unprotected cellular concrete should not be exposed to the weather.

Autoclaved aerated concrete

Cellular concrete considered so far is moist cured, usually by means of steam at atmospheric pressure. However, autoclaving, that is high-pressure steam curing (see p. 372) can also be used. The latter method results in a higher strength but autoclaved aerated concrete, as it is usually called, requires factory production. Masonry units are made by cutting the original mass while still soft. Steel reinforcement can be incorporated in the units but, because cellular concrete does not protect the embedded reinforcement, pretreatment of the reinforcement is necessary. The units are ready for use immediately following cooling after production; however, their initial moisture content of 20 to 30 per cent by mass should be reduced by drying in the atmosphere, with concomitant shrinkage occurring.

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The benefits of autoclaving, usually at 180 °C (256 °F), arise from a rapid pozzolanic reaction between Portland cement and often added lime, with very fine siliceous sand or fly ash, or a mixture of the two materials. Fly ash imparts a grey colour whereas, with pure sand, the colour is white. The C-S-H formed initially reacts with added silica in the mix so that the end product has a Ca/(A1 + Si) ratio of about 0.8; some unreacted silica remains.[13.136]

Table 13.14[13.134] gives the properties of aerated autoclaved concrete made in the United Kingdom in the form of masonry units or reinforced panels. Generally, these have lower strengths (2 to 8 MPa (300 to 1200 psi)) than normal weight concrete, but they have the advantage of a lower density (typically 500 to 1000 kg/m^3 (30 to 60 lb/ft^3) and better thermal insulation properties. It must be remembered that thermal conductivity increases linearly with moisture content: when this is 20 per cent, the

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conductivity is typically almost double that when moisture content is zero.[13.152]

Table 13.14. Typical Properties of Autoclaved (High-Pressure Steam-Cured) Aerated Concrete[13.134] (Crown copyright)

Dry density		*Compressive strength (tested wet)*		*Flexural strength*		*Modulus of elasticity*		*Thermal conductivity at 3 per cent moisture content*	
kg/m³	*lb/ft³*			*MPa*	*psi*	*GPa*	*10^6 psi*	*Jm/m²s °C*	*Btu ft/ft²h °F*
450	28	3.2	460	0.65	90	1.6	2.3	0.12	0.07
525	33	4.0	580	0.75	100	2.0	2.9	0.14	0.08
600	37	4.5	650	0.85	120	2.4	3.5	0.16	0.09
675	42	6.3	910	1.00	150	2.5	3.6	0.18	0.10
750	47	7.5	1090	1.25	180	2.7	3.9	0.20	0.12

The air permeability of autoclaved aerated concrete decreases with an increase in its moisture content but, even when the concrete is dry, the permeability at low pressures (such as those generated by wind) is negligible.[13.152]

Autoclaved aerated concrete does not allow water to rise by capillary action through the large pores. In consequence, the material has a good

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resistance to freezing and thawing,[13.152] provided the hydrated cement paste itself is not vulnerable.

Recommendations for determining various properties of autoclaved aerated concrete have been published by RILEM;[13.137] in addition, BS EN 678 : 1994 prescribes the method of determining dry density, and BS EN 679 : 2009 covers the measurement of compressive strength. The determination of shrinkage is prescribed in BS EN 680 : 2005. Typical properties of autoclaved aerated concrete are discussed by the Building Research Establishment.[13.134]

No-fines concrete

This is a form of lightweight concrete obtained when fine aggregate is omitted, i.e. consisting of cement, water, and coarse aggregate only. No-fines concrete is thus an agglomeration of coarse aggregate particles, each surrounded by a coating of cement paste up to about 1.3 mm (0.05 in.) thick. There exist, therefore, large voids within the body of the concrete which are responsible for

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its low strength, but their large size means that no capillary movement of water can take place.

The density of no-fines concrete depends primarily on the grading of the aggregate. Because well-graded aggregate packs to a higher bulk density than when the particles are all of one size, low density no-fines concrete is obtained with one-size aggregate. The usual size is 10 to 20 mm ($\frac{3}{8}$ to $\frac{3}{4}$ in.); 5 per cent oversize and 10 per cent undersize are allowed, but no particles should be smaller than 5 mm ($\frac{3}{16}$ in.). Flaky or elongated particles should be avoided. The use of sharp-edged crushed aggregate is not recommended as local crushing can take place under load. The aggregate should be dampened before mixing in order to facilitate uniform coating by the cement paste.

There are no workability tests for no-fines concrete; a visual check to ensure even coating of all particles is adequate. No-fines concrete must be placed very rapidly because the thin layer of

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cement paste can dry out; this would result in a reduced strength.[13.119]

No compaction should be applied to no-fines concrete, but rodding in corners of the formwork and around obstacles (where there is a risk of arching) may be useful. Vibration, other than of very short duration, can cause the cement paste to run off the aggregate. Because no-fines concrete does not segregate, it can be dropped from a considerable height and placed in very high lifts of up to three storeys;[13.119] the low pressure exerted on formwork is advantageous in this connection. However, because young no-fines concrete exhibits very little cohesion, formwork must remain in place until sufficient strength has been developed to hold the material together. Moist curing is important, especially in a dry climate or under windy conditions, because of the small thickness of cement paste involved.[13.153]

The density of no-fines concrete is calculated simply as the sum of the *bulk* density of the aggregate (in the appropriate state of compaction)

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plus the cement content in kg/m^3 (lb/ft^3) plus the water content in kg/m^3 (lb/ft^3). This is so because no-fines concrete compacts very little. With normal weight aggregate, the density of no-fines concrete varies between 1600 and 2000 kg/m^3 (see Table 13.15) but, using lightweight aggregate, no-fines concrete with a density of only 640 kg/m^3 (40 lb/ft^3) can be obtained.

Table 13.15. Typical Properties of No-fines Concrete with 9.5–19 mm ($\frac{3}{8}$–$\frac{3}{4}$ in.) Aggregate[13.154]

Aggregate/cement ratio by volume	*Water/cement ratio by mass*	*Density*		*28-day compressive strength*	
		kg/m^3	*lb/ft^3*	*MPa*	*psi*
6	0.38	2020	126	14	2100
7	0.40	1970	123	12	1700
8	0.41	1940	121	10	1450
10	0.45	1870	117	7	1000

The compressive strength of no-fines concrete varies generally between 1.5 and 14 MPa (200 and 2000 psi), depending mainly on its density

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which is governed by the cement content[13.154] (Fig. 13.17). The water/cement ratio as such is not the main controlling factor and, in fact, there is a narrow optimum water/cement ratio for any given aggregate. A water/cement ratio higher than the optimum would make the cement paste drain away from the aggregate particles, whereas, with too low a water/cement ratio, the cement paste would not be sufficiently adhesive and proper composition of the concrete could not be achieved.

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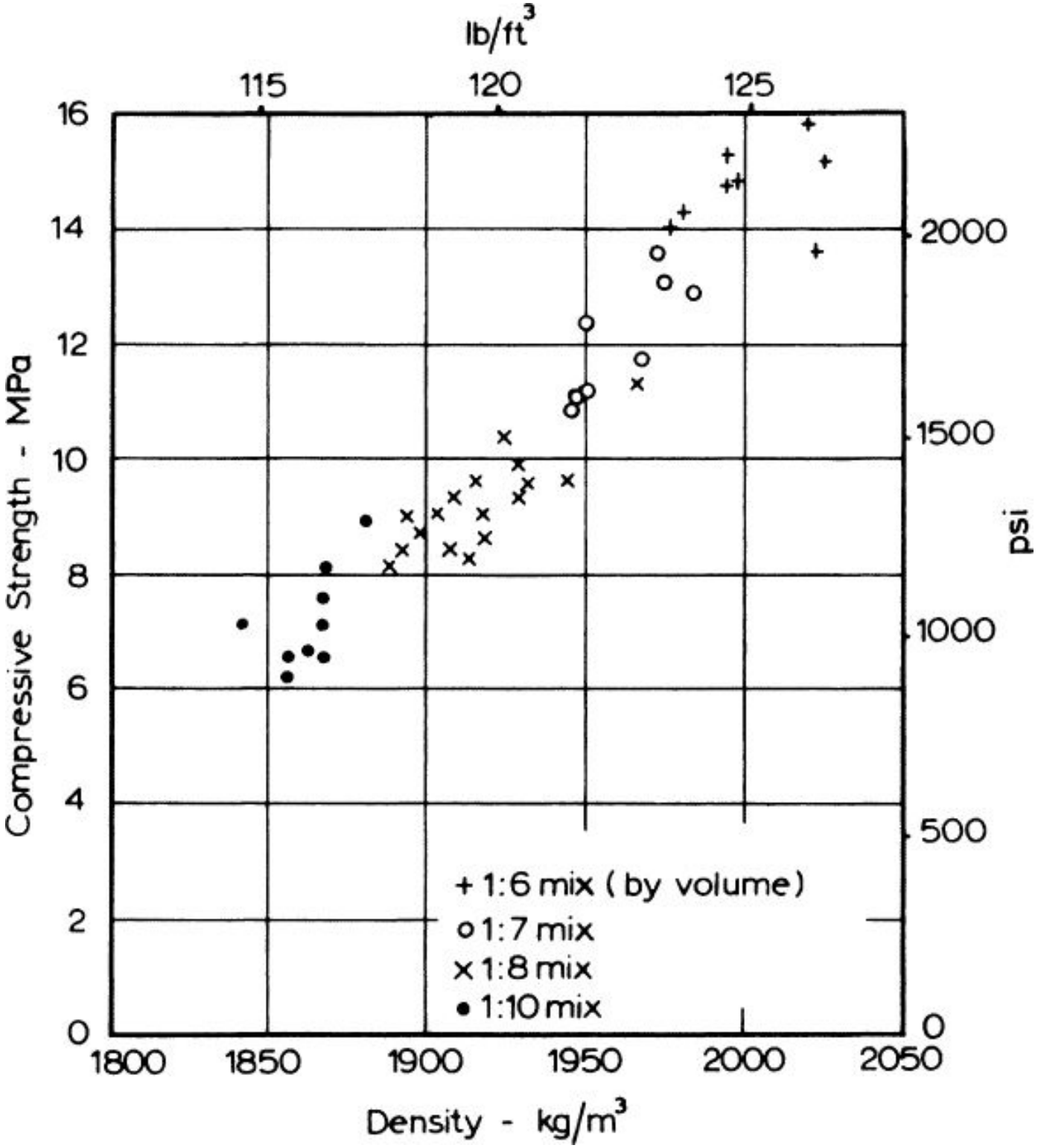


Fig. 13.17. Compressive strength of no-fines concrete at the age of 28 days as a function of its density at the time of testing[13.154]

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It is rather difficult to predict the optimum water/cement ratio, particularly because it is affected by the absorption of the aggregate but, as a general guide, the water content of the mix can be taken as 180 kg per cubic metre of concrete (300 lb/yd^3). The water/cement ratio will then depend on the cement content necessary for a sufficient coating of the aggregate; typically, the water/cement ratio is between 0.38 and 0.52.[13.153] The resulting strength has to be determined by test. In this connection, it should be noted that test specimens for the compression test have to be compacted in a special manner, using an extension piece to the mould and a rammer in a guide tube; the test method is prescribed by BS 1881-113 : 1983.

The increase in strength of no-fines concrete with age is of the same form as in normal concrete. The flexural strength is typically 30 per cent of the compressive strength, i.e. relatively higher than for ordinary concrete.[13.153] The modulus of elasticity varies with strength; for in-

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stance, a modulus of 10 GPa (1.3×10^6 psi) was found at a strength of 5 MPa (700 psi).

Shrinkage of no-fines concrete is considerably lower than that of normal concrete: a typical value is 120×10^{-6} but up to 200×10^{-6} when the relative humidity is extremely low. This is because the cement paste is present as a thin coating only and contraction on drying is largely restrained by the aggregate. Because the paste has a large surface area exposed to air, the rate of shrinkage is very high: the total movement may be completed in little over a month, and half the shrinkage may take place in 10 days.

The coefficient of thermal expansion of no-fines concrete is about 0.6 to 0.8 of that of normal concrete, but the actual value of the coefficient of thermal expansion depends on the type of aggregate used.

The coefficient of thermal conductivity of no-fines concrete is between 0.69 and 0.94 J/m^2s °C/m (0.40 and 0.54 Btu/ft^2h °F/ft) when normal

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weight aggregate is used but only about 0.22 J/m^2s °C/m (0.13 Btu/ft^2h °F/ft) with lightweight aggregate. However, a high moisture content in the concrete very appreciably increases the thermal conductivity.

Because of the large size of the pores, no-fines concrete is not subject to capillary suction. In consequence, no-fines concrete is highly resistant to frost, provided of course that the pores are not saturated; if saturated, freezing would cause a rapid disintegration. High absorption of water, however, makes no-fines concrete unsuitable for use in foundations and in situations where it may become saturated with water. The maximum absorption can be as high as 25 per cent by volume, or half that amount by mass but, under normal conditions, the absorbed water does not exceed one-fifth of the maximum. Nevertheless, external walls have to be rendered on both sides; this has also the effect of reducing the permeability to air. Rendering and painting reduce the sound-absorbing properties of no-fines concrete (through clos-

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ing of the pores) so that, where the acoustic properties are considered to be of paramount importance, one side of a wall should not be rendered. It can be observed that the open texture of no-fines concrete makes it very suitable for rendering.

A beneficial effect of the large pores in no-fines concrete is that it allows easy drainage under appropriate circumstances. This is exploited in the use of no-fines concrete which has at least a 15 per cent air-void content in pavement around trees (which thus do not become deprived of water) and in domestic car parks (overlying a permeable subgrade).[13.133]

The main use of no-fines concrete is in load-bearing walls in domestic buildings and in infilling panels in framed structures. No-fines concrete is not normally used in reinforced concrete but, if this is required, the reinforcement has to be coated with a thin layer (about 3 mm ($\frac{1}{8}$ in.)) of cement paste in order to improve the bond characteristics and to prevent corrosion. The easiest way to coat the reinforcement is by shotcreting.

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Nailing concrete

It is sometimes required to make nailing concrete, and this may be achieved by using sawdust as aggregate. Nailing concrete is a material into which nails can be driven and in which they are firmly held. The last stipulation is made because, for instance, in some low-density concretes, nails, although easily driven in, fail to hold. According to ACI 523. 1R-92[13.118] the concrete should have a minimum holding force of 178 N (40 lb) when applied to a specialty roofing nail. The nailing properties are required in some types of roof construction and in precast units for houses. Because of its very large moisture movement, sawdust concrete should not be used in situations where it is exposed to moisture.

Sawdust concrete consists of roughly equal parts by volume of Portland cement, sand, and pine sawdust, with water to give a slump of 25 to 50 mm (1 to 2 in.). Such a concrete bonds well to ordinary concrete and is a good insulator.

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The sawdust should be clean and without a large amount of bark as this introduces a high organic content and upsets the reactions of hydration. Chemical treatment of sawdust is advisable to avoid adverse effect on setting and hydration, to prevent the sawdust rotting, and to reduce its moisture movement. Best results are obtained with sawdust size between 6.3 mm ($\frac{1}{4}$ in.) and 1.18 mm (No. 16 ASTM) sieves but, because of the variable behaviour of different kinds of sawdust, the use of a trial mix is recommended. Sawdust concrete has a density of between 650 and 1600 kg/m^3 (40 and 100 lb/ft^3).

Sawdust from tropical hardwood has been used to make sawdust concrete with a 28-day compressive strength (measured on cubes) of 30 MPa (4500 psi) and a splitting strength of 2.5 MPa (400 psi); the concrete had a density of 1490 kg/m^3 (93 lb/ft^3).[13.120]

Other wood waste, such as splinters and shavings, suitably treated chemically, have also been used to make non-load-bearing concrete with a

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density of 800 to 1200 kg/m^3 (50 to 75 lb/ft^3). Cork granules can also be used.[13.155]

Nailing concrete can also be made with some other aggregates, such as expanded slag, pumice, scoria, and perlite.

Synthetic organic materials are also used, e.g. expanded polystyrene. This has a bulk density of below 10 kg/m^3 (well below 1 lb/ft^3) and produces concrete with particularly good insulating properties. A mix with 410 kg of cement per cubic metre (700 lb/yd^3) has a density of 550 kg/m^3 (35 lb/ft^3) and a strength of 2 MPa (300 psi). However, because of a wide disparity in the density of the mix ingredients, mixing is difficult and the use of a large volume of entrained air, up to 15 per cent, may be required. Care is necessary in handling polystyrene because it is combustible.[13.118]

Guidance on low-density concrete in general, defined as concrete with an over-dry density of not more than 800 kg/m^3 (50 lb/ft^3), is given in

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ACI 523.1R-92.[13.118] Such concrete has a compressive strength between about 0.7 and 6 MPa (100 and 900 psi). The essential feature of such concrete when used for insulating purposes is its low coefficient of thermal conductivity, which should be below about 0.3 J/m^{2}s °C/m (0.2 Btu/ft^{2}h °F/ft).

If water ingresses into concrete, its thermal conductivity will increase very significantly. This can happen with perlite and vermiculite aggregates, but not when closed-cell polystryrene beads are used.[13.107]

Remark about specialized concretes

The title of this chapter can be, correctly, interpreted to include also other special concretes. Some of these have highly specific applications and are treated in appropriate publications. Others involve additional ingredients which should be dealt with in detail for a meaningful description of the resulting concrete. This cannot be

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done within the confines of the present book. It is, therefore, thought best not to discuss mixes other than those generally understood to be concrete without a qualifying reference to a special additional material.

The only exception to this 'rule' is a development of compacts for structural use, the so-called *reactive powder concrete* (RPC), which actually is not concrete because the sole aggregate is sand. An actual structure made of RPC is a precast footbridge in Sherbrooke, Quebec, Canada.[13.162] This RPC developed a compressive strength of 199 MPa (30,000 psi) with a standard deviation of 9.5 MPa (1300 psi). The flexural strength was 40 MPa (5800 psi).

A typical composition of RPC was (in kg/m^3): cement 705, silica fume 230, crushed quartz 210, sand 1010, water 185, and steel fibres 140. The cement was Type II cement.

The special attributes of RPC are due to: a) increased homogeneity of the material thanks to a

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small content of sand so that there is no interface zone; b) optimised grain-size distribution and application of pressure; and c) heat treatment.

Use of RPC is probably limited to specialised structures where a high cost can be tolerated. Nevertheless, RPC shows the direction of some future developments in concrete technology, but not for common use of concrete.

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Chapter 14. Selection of concrete mix proportions (mix design)

It can be said that the properties of concrete are studied primarily for the purpose of selection of appropriate mix ingredients, and it is in this light that the various properties of concrete will be considered in this chapter.

In the British usage, the selection of the mix ingredients and their proportions is referred to as *mix design*. This term, although common, has the disadvantage of implying that the selection is a part of the structural design process. This is not correct because the structural design is concerned with the required performance of concrete, and not with the detailed proportioning of materials that will ensure that performance. The American term *mixture proportioning* is unexceptional, but it is not used on a world-wide basis. For this reason, in this book the expression at the head of the

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chapter, sometimes abbreviated to *mix selection*, will be adopted.

Although the structural design is not normally concerned with mix selection, the design imposes two criteria for this selection: strength of concrete and its durability. It is important to add an implied requirement to the effect that workability *must* be appropriate for the placing conditions. The workability requirement applies not only to, say, slump at the time of discharge from the mixer but also to a limitation on the slump loss up to the time of placing of concrete. Because of the dependence of the required workability upon the site conditions, workability should generally not be fixed prior to the consideration of the construction procedure.

In addition, the selection of mix proportions has to take into account the method of transporting the concrete, especially if pumping is envisaged. Other important criteria are: setting time, extent of bleeding, and ease of finishing; these three are interlinked. Considerable difficultues

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can arise if these criteria are not properly taken into account during the selection of the mix proportions or when adjusting these proportions.

The selection of mix proportions is thus, simply, the process of choosing suitable ingredients of concrete and determining their relative quantities with the object of producing as economically as possible concrete of certain minimum properties, notably strength, durability, and a required consistency.

Cost considerations

The preceding sentence stresses two points: that the concrete is to have certain specified minimum properties, and that it is to be produced as economically as possible – a common enough requirement in engineering.

The cost of concreting, as of any other type of construction activity, is made up of the costs of the materials, plant, and labour. The variation in the cost of material arises from the fact that cement is several times dearer than aggregate, so

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that, in selecting the mix proportions, it is desirable to avoid a high cement content. The use of comparatively lean mixes confers also considerable technical advantages, not only in the case of mass concrete where the evolution of excessive heat of hydration may cause cracking, but also in structural concrete where a rich mix may lead to high shrinkage and cracking. It is, therefore, clear that to err on the side of rich mixes is not desirable, even if the cost aspect is ignored. In this connection it should be remembered that the different cementitious materials vary in cost per unit mass, being, with the exception of silica fume, cheaper than Portland cement. Their influence on the different properties of concrete also varies, as discussed in the appropriate chapters.

In estimating the cost of concrete, it is essential to consider also the variability of its strength because it is the 'minimum', or characteristic, strength (see p. 734) that is specified by the designer of the structure, and is indeed the criterion of acceptance of the concrete, while the actual

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cost of the concrete is related to the materials producing a certain mean strength. This touches very closely on the problem of quality control. It should be borne in mind that a higher level of quality control represents a higher expenditure both on supervision and on batching equipment, and there are occasions when careful mix selection and quality control may not be justified. The decision on the extent of quality control, often an economic compromise, will thus depend on the size and type of construction. It is essential that the degree of control is estimated at the outset of the process of selection of mix proportions, so that the difference between the mean and the minimum, or characteristic, strength is known.

The cost of labour is influenced by the workability of the mix: workability inadequate for the available means of compaction results in a high cost of labour (or in insufficiently compacted concrete). Dealing with blockages in pumping is also labour-intensive. The exact cost of labour depends on the details of organization of the job

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and the type of equipment used, but this is a specialized topic.

Specifications

This large topic cannot be dealt with in this book and will be considered only in so far as the type of specification affects the mix selection.

In the past, specifications for concrete prescribed the proportions of cement, and fine and coarse aggregate. Certain traditional mixes were thus produced but, because of the variability of the mix ingredients, concretes having fixed cement–aggregate proportions and a given workability vary widely in strength. For this reason, the minimum compressive strength was later added to other requirements. When the strength is specified, the prescription of proportions makes the specification unduly restrictive where good quality materials are available, but elsewhere it may not be possible to achieve an adequate strength using the prescribed mix proportions. This is why, sometimes, clauses prescribing the grading

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of aggregate and the shape of the particles were added to the other requirements. However, the distribution of aggregates in many countries is such that these restrictions are often uneconomic. In this connection it should be noted that, with the exception of specialized construction, such as nuclear containment vessels, only locally available aggregates are used; transportation over long distances is prohibitively expensive.

More generally, specifying at the same time strength as well as mix ingredients and their proportions, and also the aggregate shape and grading, leaves no room for economies in the mix selection, and makes progress in the production of economic and satisfactory mixes on the basis of the knowledge of the properties of concrete impossible.

It is not surprising, therefore, that the modern tendency is for specifications to be less restrictive. They lay down limiting values but sometimes give also as a guide the traditional mix proportions for the benefit of the contractor who does

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not wish to use a high degree of quality control. The limiting values may cover a range of properties; the more usual ones are:

1. 'Minimum' compressive strength necessary from structural considerations;
2. Maximum water/cement ratio and/or minimum cement content and, in certain conditions of exposure, a minimum content of entrained air to give adequate durability;
3. Maximum cement content to avoid cracking due to the temperature cycle in mass concrete;
4. Maximum cement content to avoid shrinkage cracking under conditions of exposure to a low humidity; and
5. Minimum density for gravity dams and similar structures.

In addition, the nature of the cementitious materials, sometimes by a specific requirement with respect to the type or composition of cement, at

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other times by proscription, may be included in the specification.

All these various requirements must be satisfied in the selection and proportioning of mix ingredients.

Specification of quantities almost invariably includes associated tolerances on the various quantities. With respect to strength, most national specifications lay down clear requirements. The tolerances on cement content and water/cement ratio are generally less clear but equally important. Particularly critical is the tolerance on cover to reinforcement which, albeit not a 'mix-proportions item', is closely linked to the specified strength of concrete and to its cement content from the durability standpoint. The tolerance on cover must be explicitly specified and should be logically associated with the tolerance on strength or on the cement content.

The British approach, given in BS EN 206-1 : 2000 and the complementary BS 8500-2 : 2002, is to recognize four methods of specifying con-

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crete mixes. A *designed mix* is specified by the designer principally in terms of strength, cement content, and water/cement ratio; compliance relies on strength testing. A *prescribed mix* is specified by the designer in terms of the nature and proportions of mix ingredients; the concrete producer simply makes the concrete 'to order'. The assessment of mix proportions is used for compliance purposes, strength testing not being routinely used. The use of prescribed mixes is advantageous when particular properties of concrete, for instance with respect to its finish or abrasion resistance, are required. However, a prescribed mix should be specified only when there are sound reasons for assuming that it will have the required workability, strength, and durability.

A *standardized mix* is based on ingredients and proportions fully listed in BS 5328-2 : 2002 for several values of compressive strength up to 25 MPa, measured on cubes. The fourth and last type of mix is the *designated mix*, for which the concrete producer selects the water/cement ratio

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and the minimum cement content, using a table of structural applications coupled with standard mixes. This approach can be used only if the concrete producer holds a special certificate of product conformity based on product testing and surveillance, coupled with certification of quality assurance.

Standard mixes are used only in minor construction such as housing. Designated mixes, although they can be used for strengths up to 50 MPa, are limited in application to routine construction. It is, therefore, only in the selection of designed and prescribed mixes that a full knowledge of properties of concrete can be used. These four types of mixes are varied somewhat in BS 8500-2 : 2002.

In the American practice, when there is no experience on the basis of which mix proportions could be selected and trial mixes made, it is necessary to base the mix proportions on standard proportions which, in order to be safe, are perforce very stringent. This approach can be used

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only for low strength concrete. For example, ACI 318-02[14.8] prescribes, for a specified 28-day compressive strength (measured on cylinders) of 27 MPa (4000 psi), a maximum water/cement ratio of 0.44 in the case of non-air-entrained concrete, and 0.35 in airentrained concrete. In the latter case, higher strengths require a proper use of trial mixes, but, in the case of non-air-entrained concrete, ACI 318-95[14.8] allows the use of a water/cement ratio of 0.38 for concrete with a specified 28-day strength of 31 MPa (4500 psi).

The process of mix selection

The basic factors which have to be considered in determining the mix proportions are represented schematically in Fig. 14.1. The sequence of decisions is also shown down to the quantity of each ingredient per batch. There are, of course, variations in the exact method of selecting the mix proportions. For instance, in the excellent method of the American Concrete Institute[14.5] (see p. 754), the water content in kilograms per cubic

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metre (or pounds per cubic yard) of concrete is determined direct from the workability of the mix (given the maximum size of aggregate) instead of being found indirectly from the water/cement ratio and the cement content.

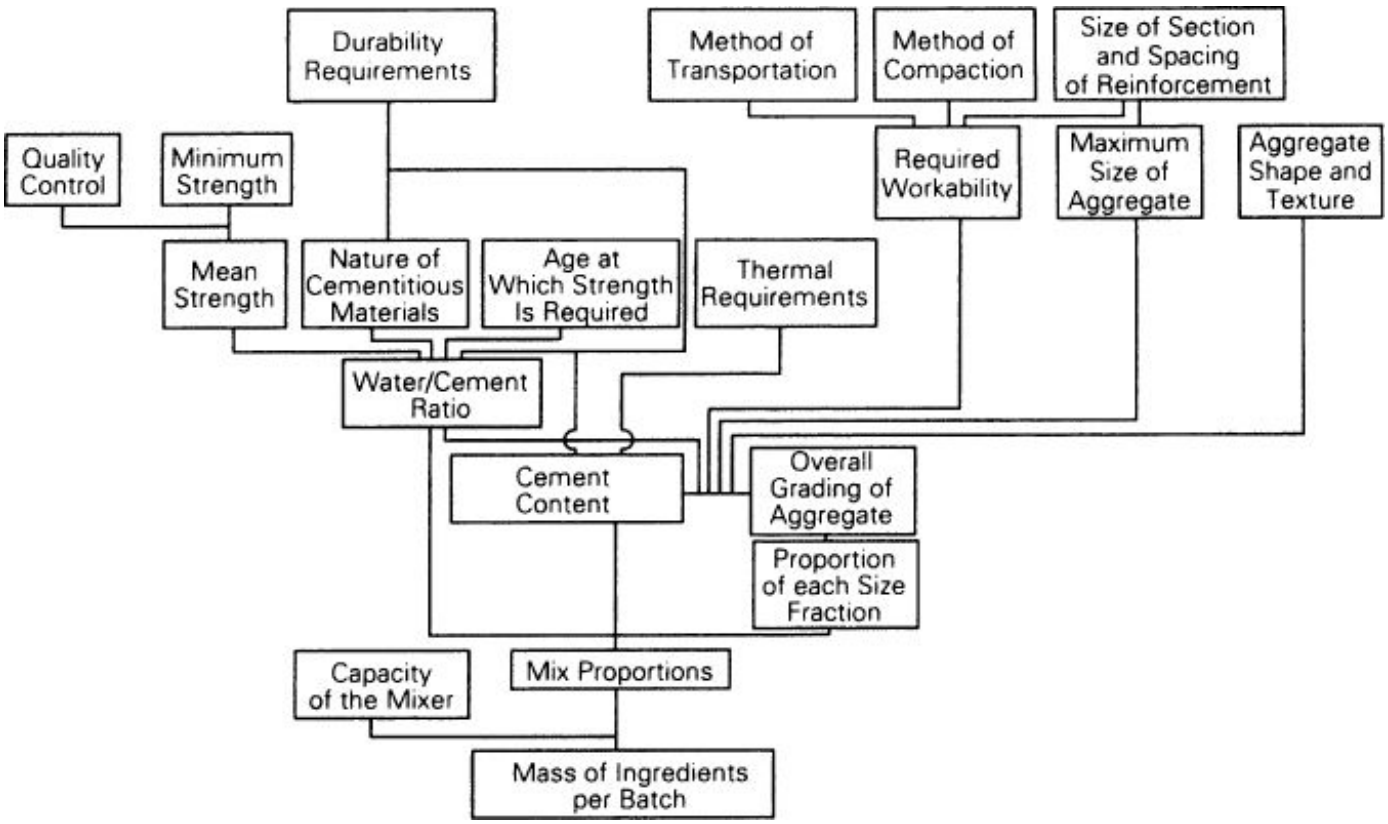


Fig. 14.1. Basic factors in the process of mix selection

It should be explained that an exact determination of mix proportions by means of tables or computer data is generally not possible: the materials used are essentially variable and many of

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their properties cannot be assessed truly quantitatively. For example, aggregate grading, shape and texture cannot be defined in a fully satisfactory manner. In consequence, all that is possible is to make an intelligent guess at the optimum combinations of the ingredients on the basis of the relationships established in the earlier chapters. It is not surprising, therefore, that in order to obtain a satisfactory mix, we not only have to calculate or estimate the proportions of the available materials but must also make trial mixes. The properties of these mixes are checked and adjustments in the mix proportions are made; further trial mixes are made in the laboratory until a fully satisfactory mix is obtained.

However, a laboratory trial mix does not provide the final answer even when the moisture condition of aggregate is taken into account. Only a mix made and used on the site can guarantee that all the properties of the concrete are satisfactory in every detail for the particular job in hand. To justify this statement three points may

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be mentioned. Firstly, the mixer used in the laboratory is generally different in type and performance from that employed on site. Secondly, the pumping properties of the mix may need to be verified. Thirdly, the wall effect (arising from the surface to volume ratio) in laboratory test specimens is larger than in the full-size structure, so that the fine aggregate content of the mix as determined in the laboratory may be unnecessarily high.

It can be seen then that mix selection requires both a knowledge of the properties of concrete and experimental data or experience.

Other factors, such as effects of handling, transporting, delay in placing, and small variations in weather conditions may also influence the properties of concrete on the site but these are generally secondary and necessitate no more than minor adjustments in the mix proportions during the progress of work.

This may be an appropriate place to note that the mix proportions, once chosen, cannot be ex-

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pected to remain entirely immutable because the properties of the ingredients may vary from time to time. In particular, it is difficult to know the precise amount of free water in the mix because of the variation in the moisture content of the aggregate, expecially the fine aggregate. The problem is even greater with lightweight aggregate, especially in pumped concrete. Other variations occur in the grading of aggregate, particularly its dust content, and in the temperature of the concrete due to exposure of the ingredients and of the mixer to the sun or due to the cement being hot. In consequence, periodic adjustments to the mix proportions are necessary.

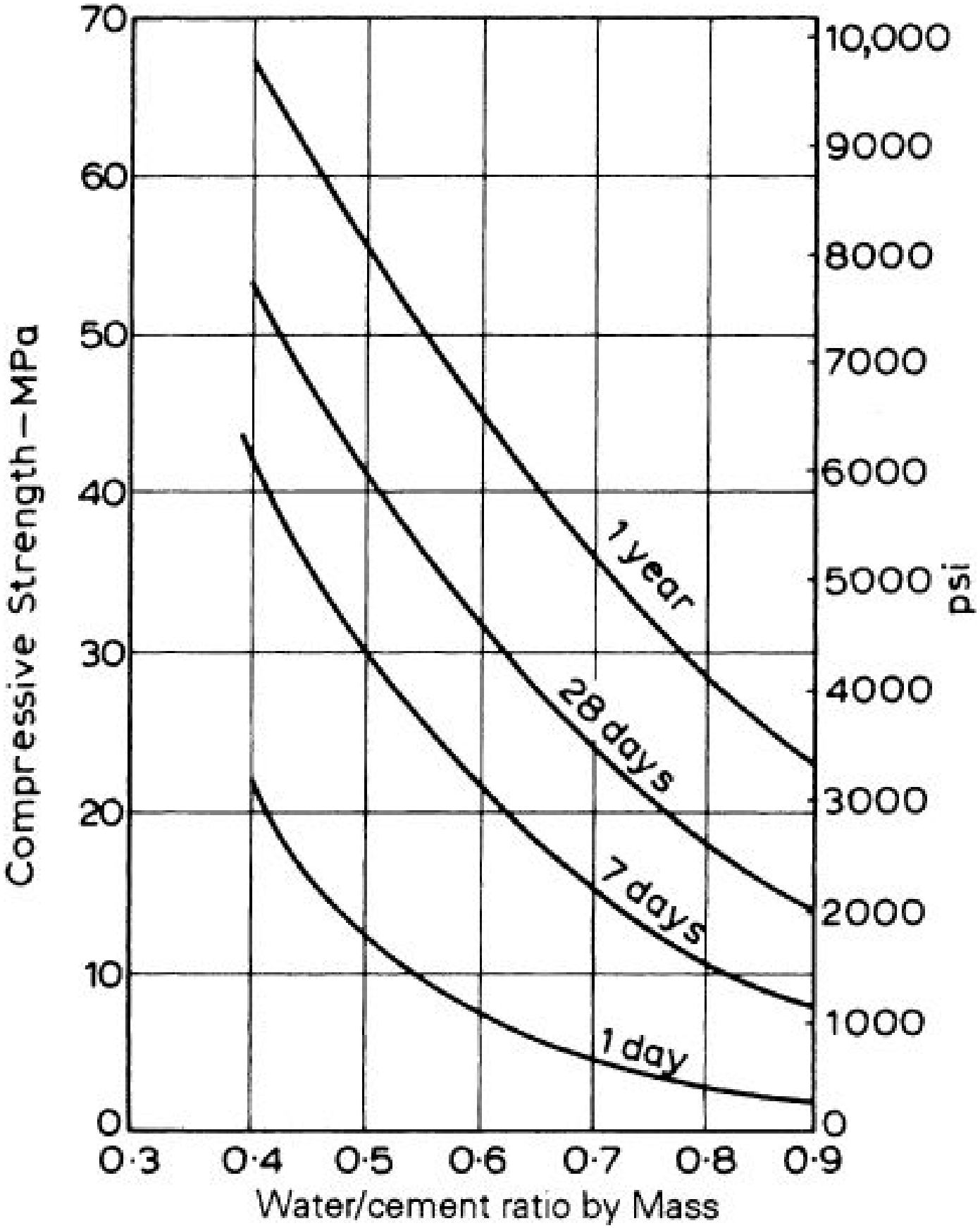
Mean strength and 'minimum' strength

Compressive strength is one of the two most important properties of concrete, the other one being durability. Strength is of importance both *per se* and also in so far as it influences many other desirable properties of hardened concrete. Basically, the *mean* compressive strength required at

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a specified age, usually 28 days, determines the nominal water/cement ratio of the mix. Figure 14.2 gives this relation for concretes made in the late 1970s with British ordinary Portland cements cured at normal temperatures. This figure is intended to serve as no more than an illustration and, in any case, the strength values in the figure err on the safe side. If, however, one batch of cement is to be used throughout the job, it is possible to take advantage of the actual strength of the given cement, that is, to use an experimental relation between strength and the water/cement ratio.

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Fig. 14.2. Relation between compressive strength and water/cement ratio for 102 mm (4 in.) cubes of fully compacted concrete for mixes of various proportions made with typical British ordinary Portland cements of the late 1970s. The values used are conservative estimates

If curves of the type shown in Fig. 14.2 are used, the type of cement must be known because the rate of hardening of cements of different types varies; when different cementitious materials are used, the variation in the rate of gain of strength can be even larger. However, beyond the age of one or two years the strengths of concretes made with different cements tend to be approximately the same.

Structural design is based on the assumption of a certain *minimum* strength of concrete, but the actual strength of concrete produced, whether on site or in the laboratory, is a variable quantity (see p. 639). In selecting a concrete mix we must,

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therefore, aim at a *mean* strength higher than the minimum.

The distribution of strength of test specimens can be described by the mean and the standard deviation. As mentioned on p. 641, the distribution of strength of concrete test specimens is assumed to be normal (Gaussian). For practical purposes, such an assumption is acceptable, even though examples of skewness have been reported: in low strength concrete by McNicoll and Wong,[14.23] and in high strength concrete by Cook[14.24] and also in ACI 363R-92.[14.12] The assumption of normal distribution errs on the safe side with respect to the number of test results expected to fall below the specified value of strength.[14.25]

From the knowledge of the probability of a specimen having a strength differing from the mean by a given amount (Table 14.1) we can define the 'minimum' strength of a given mix. No absolute minimum can be specified because, from the statistical viewpoint, there is always a

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certain probability of a test result falling below a minimum, however low it is set; to make this probability extremely low would be uneconomical. It is, therefore, usual to define the ‘minimum’ as a value to be exceeded by a predetermined proportion of all test results, usually 95 per cent when single test results are considered, and 99 per cent when a running average of three or four test results is used.

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Table 14.1. Percentage of Specimens Having a Strength Lower than (Mean –k × Standard Deviation)

k	*Percentage of specimens having strength below* $(\bar{x} - k\sigma)$
1.00	15.9
1.50	6.7
1.96	2.5
2.33	1.0
2.50	0.6
3.09	0.1

The approach of the Building Code of the American Concrete Institute, ACI 318-02[14.8] is based, in essence, on two requirements for the 'minimum' strength, f'_{cr} in relation to the mean strength f'_c. First, there is a required probability of 1 per cent that the average of three consecutive tests (a test being the average of two cylinders) is smaller than the design strength. Second, there is a required probability of 1 per cent that an indi-

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vidual test result falls below the design strength by more than 3.5 M Pa (500 psi). In terms of standard deviation, σ, the first of these can be written as:

$$f'_{cr} = f'_c + \frac{2.33\sigma}{\sqrt{3}} = f'_c + 1.343\sigma$$

and the second (in MPa units) as

$$f'_{cr} = f'_c - 3.5 + 2.33\sigma.$$

The two conditions are equivalent when the standard deviation σ is approximately 3.5 MPa (500 psi). When it is larger, the first condition is the more severe of the two.

We should note that no absolute limit value is laid down: the approach is probabilistic so that failure to meet these requirements once in a 100 times is inherent in the system. Such failure should not be a sufficient reason for rejection of the concrete. It can be added that all specification schemes imply a risk of wrong rejection and of

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wrong acceptance: it is the two risks that have to be judiciously balanced.[14.31]

The value of the standard deviation to be used in the expression of ACI 318-02[14.8] given above is the value obtained experimentally in previous construction under similar conditions, using similar materials, to produce concrete of similar strength. In the absence of such an experimental value of the standard deviation, ACI 318-02[14.8] prescribes margins by which the mean compressive strength has to exceed the specified value of strength. These margins are very substantial, ranging from 7 MPa (1000 psi) when the specified strength is less than 21 MPa (3000 psi), to 10 MPa (1400 psi) when the specified strength is above 35 MPa (5000 psi).

According to ACI 318-02[14.8] and ASTM C 94-09a, compliance with the specified value of strength, f'_c is achieved when both of the following requirements have been satisfied:

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(a) The average value of all sets of three consecutive test results is at least equal to f'_c; and

(b) No test result falls below f'_c by more than 3.5 MPa (500 psi).

It should be recalled that a test result is the average value of the strengths of two test cylinders from the same batch of concrete, tested at the same age. The average value of three consecutive test results is a running average; this means that a test result number N appears in three sets as follows: $N - 2$, $N - 1$, N; $N - 1$, N, $N + 1$; and N, $N + 1$, $N + 2$. Thus, if the value of test number N is very low, it can significantly depress one or two or three average values. Consequently, all concrete represented by tests numbered from $N - 2$ to $N + 2$ is deemed not to comply with the specification. However, occasional failure to comply with the requirements of ACI 318-02[14.8] has to be expected (probably once in 100 tests) so that automatic rejection of the relevant concrete should not follow.

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The requirements of Eurocode 2 : 2004 parallel those of ACI 318-08, referred to earlier. A test result is the average of the strengths of two specimens but, in the British practice, cubes are used. The British approach is to use a *characteristic strength*, defined as the value of strength below which 5 per cent of all possible test results are expected to fall; the *margin* between the characteristic strength and the mean strength is chosen to achieve this probability. Compliance with the specified value of strength is achieved when both of the following requirements have been satisfied:

(a) The average value of any four consecutive test results exceeds the specified characteristic strength by 3 MPa (450 psi); and

(b) No test result falls below the specified characteristic strength by more than 3 MPa (450 psi).

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Similar requirements are prescribed for flexural tests: the values in (a) and (b) in the preceding paragraph are then 0.3 MPa (45 psi).

The topic of compliance cannot be treated adequately in this book but some statements are worth making. It is not possible to discriminate absolutely between satisfactory and unsatisfactory concrete, short of testing all of it! The object of testing is to discriminate adequately so as to achieve a balance between the producer's risk of 'good' concrete being rejected and the consumer's risk of 'bad' concrete being accepted. The balance is governed by the extent of testing as well as by the rules used.[14.31]

Variability of strength

It may be remembered (p. 640) that the abscissa of any point on the normal distribution curve is expressed in terms of the standard deviation σ, and the number of specimens whose strength differs from the mean by more than $k\sigma$ is represented by the appropriate proportional area under

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the normal curve and is given in statistical tables (Table 14.1).

Thus, if the mean strength of a sample of test specimens is $\bar{x}$, and the percentage of specimens whose strength may fall below a certain value ($\bar{x} - k\sigma$) is specified, then the value of k can be found from statistical tables, and the actual difference between the mean and the minimum, $k\,\sigma$, will depend only on the value of the standard deviation σ. This is illustrated in Fig. 14.3. Because the cement content of the mix of a given workability is related to the mean strength, it can be seen that the larger the standard deviation the higher the cement content required for a given minimum strength.

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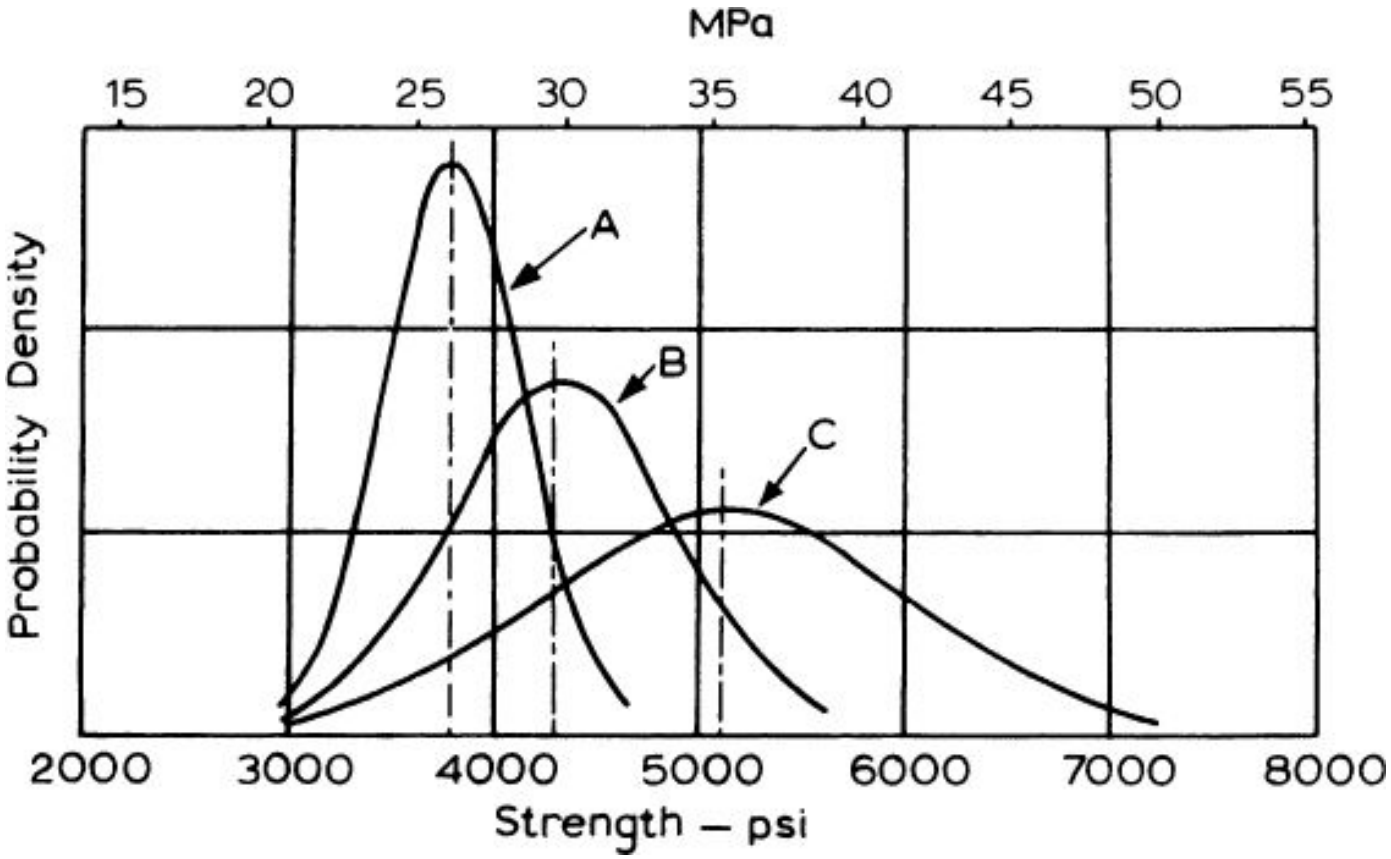


Fig. 14.3. Normal distribution curves for concretes with a minimum strength (exceeded by 99 per cent of results) of 3000 psi (20.6 MPa):

	$\bar{x}$		σ	
	psi	MPa	psi	MPa
A	3800	26.2	350	2.4
B	4300	29.6	560	3.9
C	5100	35.2	900	6.2

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The difference ($\bar{x} - k\,\sigma$) can also be expressed in terms of the coefficient of variation, $C = \sigma/\bar{x}$, as $\bar{x}(1 - kC)$. The two methods of estimating the minimum strength are identical when applied to concrete of the same mean strength but, when the data obtained for one mix are used to predict the variability of a mix of different strength, the result will depend on whether the standard deviation or the coefficient of variation is unaffected by the change in strength.

If a constant standard deviation is assumed then, knowing the estimated value of the standard deviation σ for one mix, we can calculate the mean strength of any other mix by adding a constant value $k\sigma$ to the minimum. This difference between the mean and the minimum would be constant for the same process of manufacture of concrete. On the other hand, if the coefficient of variation is assumed to be constant, the minimum strength would form a fixed proportion of the mean. These two situations are illustrated by the following numerical example.

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Let us assume that concrete produced and tested under a given set of conditions has a mean strength of 25 MPa (3600 psi) with a standard deviation of 4 MPa (580 psi). According to ACI 214-77 (Reapproved 1989),[14.18] this represents 'good' control (see Table 14.2). The coefficient of variation is (4/25) × 100, that is 16 per cent. For illustration purposes, let us assume that the required 'minimum' strength is defined as the strength exceeded by 99 per cent of all results. Using Table 14.1, we find that this 'minimum' strength is:

$$25 - 2.33 \times 4 = 15.7 \text{ MPa}.$$

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Table 14.2. Classification of Standard of Control of Concretes with Strengths up to 35 MPa (5000 psi) According to ACI 214-77 (Reapproved 1989)[14.18]

Standard of control	*Overall standard deviation, MPa (psi)*			
	In the field		*Laboratory trial mixes*	
Excellent	<3	(<400)	<1.5	(<200)
Very good	3–3.5	(400–500)	1.5	200–250
Good	3.5–4	(500–600)	1.5–2	(250–300)
Fair	4–5	(600–700)	2–2.5	(300–350)
Poor	>5	(>700)	>2.5	(>350)

Imagine now that it is desired to produce, under the same conditions and using the same materials, a concrete with a 'minimum' strength of, say, 50 MPa. The mean strength aimed at, according to the 'coefficient of variation method', would be:

$$\frac{50}{1 - 2.33 \times 0.16} = 79 \text{ MPa}$$

where as the figure given by the 'standard deviation method' would be:

$$50 + 2.33 \times 4 = 59 \text{ MPa}.$$

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The practical significance of the difference between the two methods is clearly reflected in the cost of producing a 79 MPa concrete as compared with a 59 MPa concrete under the same control.

An estimate of the difference between the mean strength and the specified 'minimum' or characteristic strength, must be made at the outset of the process of mix selection. The advice of ACI 214-77 (Reapproved 1989)[14.18] is non-committal: "The decision as to whether the standard deviation or the coefficient of variation is the appropriate measure of dispersion to use in any given situation depends on which of the two measures is the more nearly constant over the range of strengths characteristic of the particular situation." Nevertheless, ACI 214-77 (Reapproved 1989)[14.18] includes a table, reproduced here as Table 14.2, based on the assumption of a constant standard deviation, for concretes with strengths up to 35 MPa (5000 psi). However, discussion in Committee 214 of ACI continues as opinions are

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divided. It should be pointed out that the convenience of calculations and simplicity of approach, often brought into the discussions, are not the correct criteria upon which to decide whether it is the standard deviation or the coefficient of variation that should be used. What matters is the actual behaviour of concrete in construction.

The recommendations of ACI 214-77 (Reapproved 1989)[14.18] are based on concretes used up to the mid-1970s, and such concretes did not often have a cylinder strength in excess of 35 MPa (5000 psi). It is, therefore, questionable whether the approach of ACI 214-77 (Reapproved 1989) necessarily applies to high strength concrete with a 28-day compressive strength in excess of 80 MPa (12 000 psi), let alone in the region of 120 MPa (17 000 psi).

Before discussing the variability of high strength concrete, it may be useful to consider changes in concrete-making which occurred between, say, 1970 and the mid-1990s. There is no doubt that the batching equipment has been

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greatly improved, with the consequence of a much smaller variability in the mix proportions between batches. As a result, the between-test standard deviation of compressive strength test results can be expected to be smaller than in the past. On the other hand, there are few grounds for expecting the within-test variation, which arises from the operator error and the testing-machine error, to be different from what it was in the 1970s. Thus, it is likely that the overall standard deviation of test results is smaller, but not much smaller, than in the past.

In this connection it is useful to point out that the within-test and between-test standard deviations are not arithmetically additive; it is the variances that are additive. For instance, if the within-test standard deviation is 3 MPa, and the between-test standard deviation is 4 MPa, then the overall standard deviation is $(3^2 + 4^2)^{1/2} = 5$ MPa. A reduction in the between-test standard deviation to 3 MPa, while the within-test variation remains unaltered, would reduce the overall

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standard deviation to $(3^2 + 3^2)^{1/2} = 4.25$ MPa. Thus, in this particular example, a reduction in the between-test standard deviation of 1 MPa has reduced the overall standard deviation by only 0.75 MPa.

Returning now to high strength concrete, it is reasonable to assume that such concrete is produced only in modern plants with low-variability batching and with highly skilled and motivated personnel. However, the same plants also produce low- or medium-strength concrete whose variability will also be lower than the variability of concrete of similar strength produced in the 1970s. It follows that viewing the variability of high strength concrete (which is all of recent production) against the background of concretes of the 1970s gives a distorted picture.

The approach of ACI 363R-92[14.22] is to recognize that the standard deviation of "high strength concrete becomes uniform in the range of 500 to 700 psi (3.5 to 4.8 MPa)". Thus, the coefficient of variation decreases with an increase in strength

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and, in the words of ACI 363R-92, “the standard deviation method of evaluation appears to be a logical quality control procedure”.

The problem of the constant standard deviation or constant coefficient of variation is still controversial but, for a constant degree of control, laboratory test data, as well as some results of actual site tests, have been shown to support the suggestion of a constant coefficient of variation for well-compacted concretes of different mix proportions with strengths higher than about 10 MPa (1500 psi) (Fig. 14.4). On the other hand, the median values of standard deviation for different characteristic strengths measured in Swedish ready-mixed plants in 1975 suggest a constant standard deviation. The actual values are as follows:[14.32]

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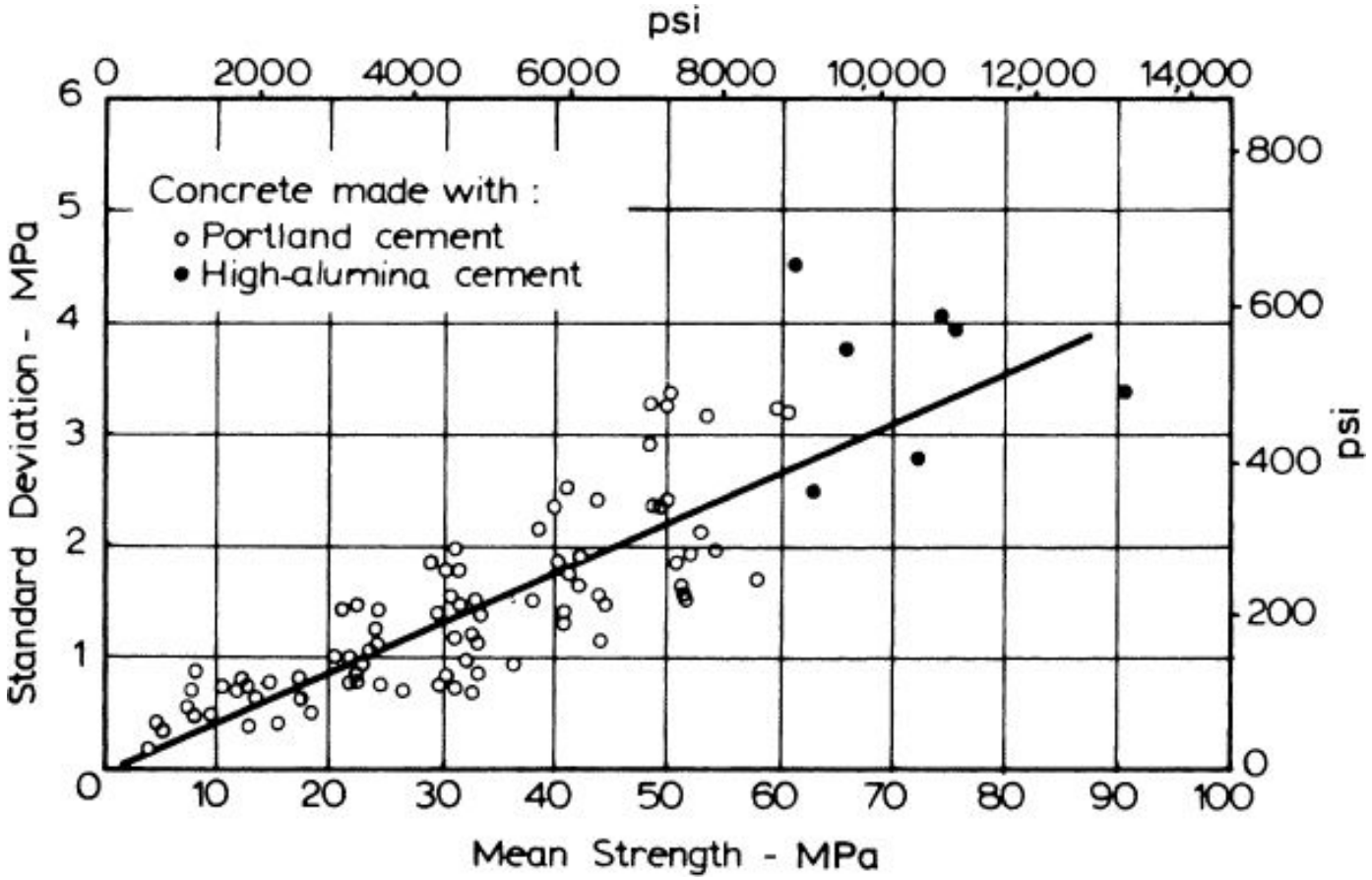
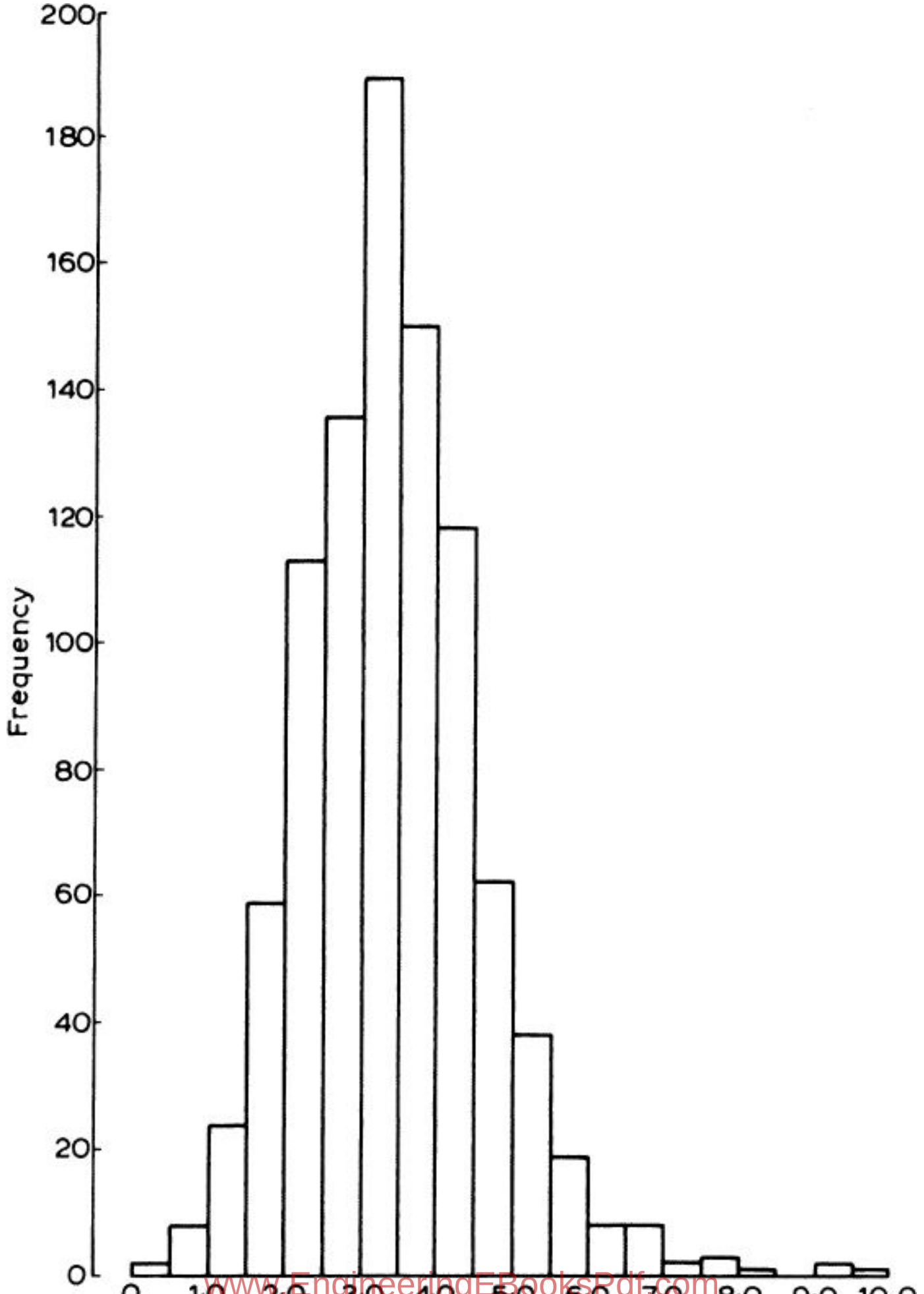


Fig. 14.4. Relation between the standard deviation and mean strength for laboratory test cubes; regression line shown[14.26]

Strength class	20	25	30	40	50	60
Standard deviation (MPa)	3.2	3.3	3.5	3.7	3.4	3.3

The distribution of the standard deviation for all classes of concretes is shown in Fig. 14.5.

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Fig. 14.5. Distribution of the standard deviation (in 0.5 MPa intervals) for all classes of concrete from ready-mixed plants in Sweden in 1975[14.32]

Swiss Standard SIA 162 (1989),[14.21] presumably based on Swiss experience, assumes that the standard deviation is independent of strength for strengths up to 45 MPa measured on single 200 mm cubes.

Surveys of test data on a large number of construction sites suggest that neither the assumption of a constant standard deviation or of a constant coefficient of variation at all ages is generally valid for site-made test specimens. From Newlon's review of the problem,[14.30] it appears that the coefficient of variation is constant up to some limiting value of strength but, for higher strengths, the standard deviation remains constant (see Fig. 14.6). Different investigators have found different values of this limiting strength,

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which may well depend on site conditions and general construction practice.

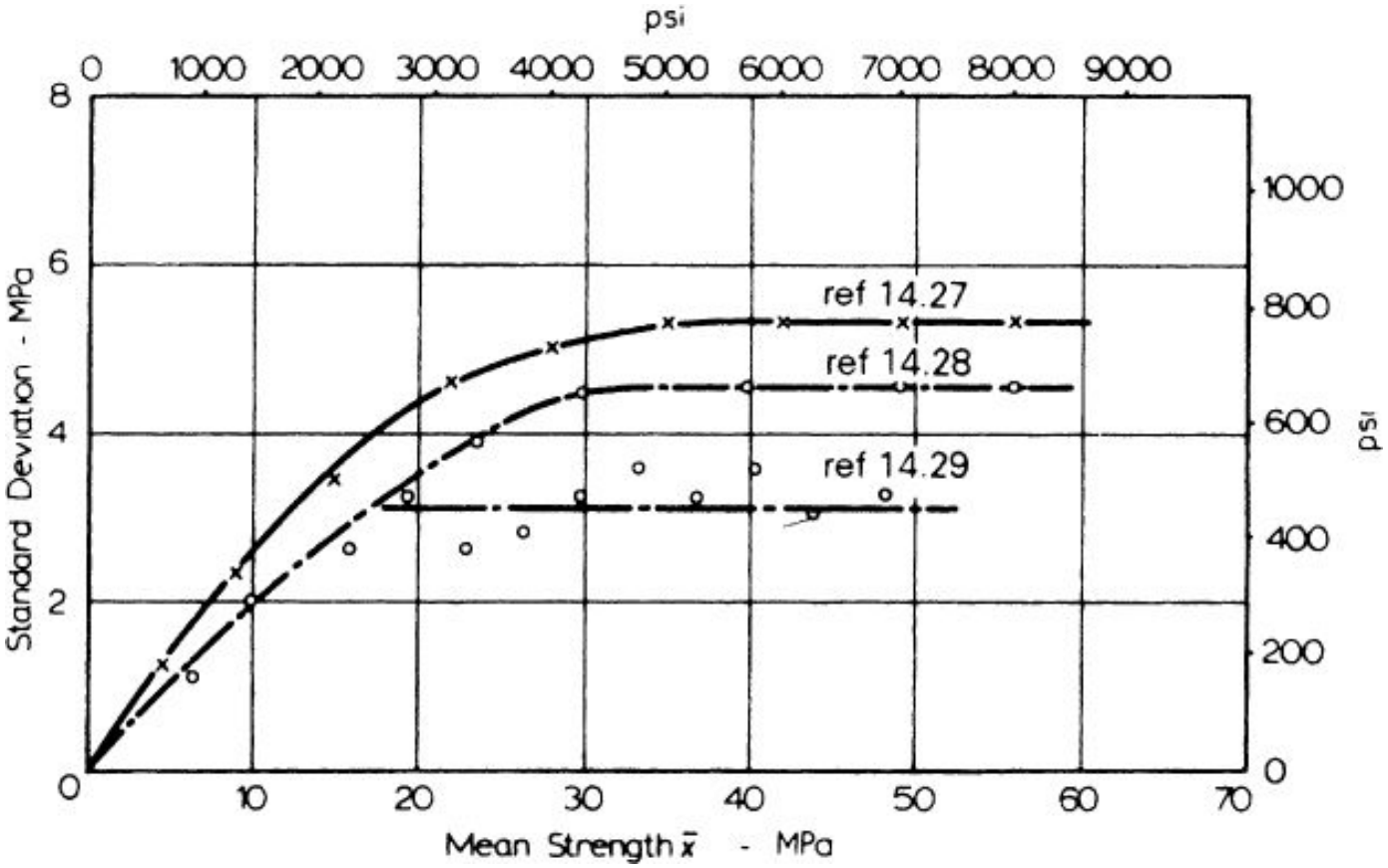


Fig. 14.6. Relation between the standard deviation and the mean strength of test specimens obtained from surveys of site data[14.30]

It is possible, however, to suggest some generalizations. Figure 14.6 indicates that for single cubes, the limiting strength is about 34 MPa (5000 psi); for averages of two cylinders, it is about 17 MPa (2500 psi); and an international

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survey involving both cubes and cylinders, tested singly and in pairs, gives an intermediate value of about 31 MPa (4500 psi). The factors accounting for these differences are not clear, but probably the lower variability of cylinders compared with cubes (cf. p. 596) is relevant. It may also be noted that, for the same intrinsic strength, the cylinder strength is lower than the cube strength. All these data apply to tests at a fixed age: for the same source of concrete, an increase in age leads to a reduction in the coefficient of variation, but the standard deviation increases; thus, it is the strength level and not merely the concrete-making that is relevant.

It is probable that neither the standard deviation nor the coefficient of variation is constant over a wide range of strengths of concrete made in the same plant. This view is supported by the Commentary on ACI 318-02[14.8] which states that "there may be an increase in standard deviation when the average strength level is raised by a significant amount, although the increment of in-

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crease in standard deviation should be somewhat less than directly proportional to the strength increase". The British approach[14.11] is to assume the standard deviation to be proportional to strength up to a value of 20 MPa (2900 psi) but for higher strengths, that is, for structural concrete, the standard deviation is assumed to be constant. In practice, therefore, it is best to establish experimental relations between the mean and minimum strengths under actual site conditions.

With respect to the variability of flexural strength, Greer[14.2] and Lane[14.3] confirmed earlier findings to the effect that both the within-test standard deviation and the between-test standard deviation are independent of the value of the flexural strength. A typical value of between-test standard deviation, when the level of control is good, is below 0.4 MPa (60 psi) (see Fig. 14.7).

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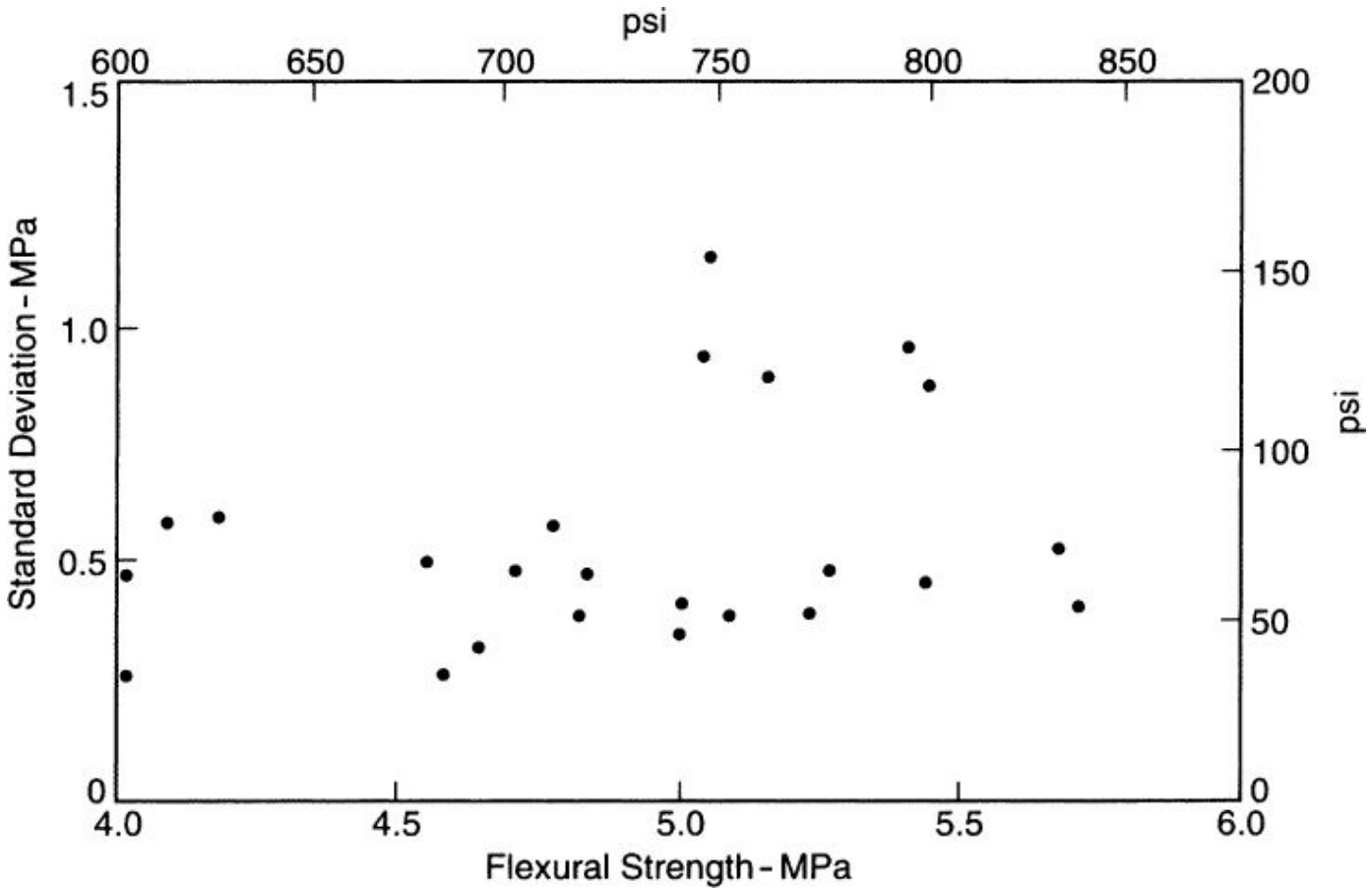


Fig. 14.7. Relation between standard deviation and 28-day flexural strength determined in pavement construction[14.3]

Quality control

It is apparent from Fig. 14.3 that the lower the difference between the minimum strength and the mean strength of the mix the lower the cement content that need be used. The factor controlling this difference for concrete of a given level of strength is the quality control. By this is meant

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the control of variation in the properties of the mix ingredients and also control of accuracy of all those operations which affect the strength or consistency of concrete: batching, mixing, transporting, placing, curing, and testing. Thus, quality control is a production tool, and one of its reflections is the standard deviation.

The variation in the strength of cement was discussed in Chapter 7. On a large project, it is possible to eliminate most of this variation by obtaining cement from one source only, when advantage can be taken of the actual strength of the cement to be used.

The influence of the variation in the grading of aggregate was stressed in Chapter 3, and this factor is particularly important when the mix is controlled by workability requirements: for the workability to be kept constant, a change in grading may require an increase in water content with a consequent drop in strength.

Variations in strength of concrete arise also from inadequate mixing, insufficient compaction,

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irregular curing, and variations in testing procedures – all discussed in the appropriate chapters. The need for control of these factors on the site is obvious.

Changes in the moisture content of aggregate, unless carefully compensated for by the amount of added water, also seriously affect the strength of concrete. To minimize these changes, stockpiles should be arranged so that the aggregate is allowed to drain before use; also, the mixer operator should be well trained in maintaining a constant workability of the mix.

A standard deviation can be ascribed to each factor separately, although in some cases the magnitude of the individual effects cannot be determined. As already mentioned the various standard deviations are additive in the root-square-form, so that if σ_1 and σ_2 are ascribed to two causes the resultant standard deviation is $\sigma = \sqrt{(\sigma_1^2 + \sigma_2^2)}$. This is important to remember as the assumption of arithmetic addition would lead to a gross over-estimate of the total

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standard deviation. The knowledge of individual contributions of various factors to the overall variation, obtained by statistical methods, is of value in deciding whether taking some measures to reduce variation is economic, or whether the reduction in variability is disproportionately small for the cost of improved control.

Quality control is sometimes taken to be synonymous with production of high-strength concrete. This is certainly not so, as low-strength concrete can be manufactured under good control, and this is indeed practised in the case of construction of massive structures where obtaining large quantities of lean concrete of low variability results in large savings. The degree of control is evaluated by the variation in test results; various statistical techniques are available.

For the sake of completeness, *quality assurance* should be mentioned. This is an administrative control system "implemented through quality assurance programmes to provide a means of controlling quality-affecting activities to pre-

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determined requirements".[14.7] Thus, quality assurance is a management tool comforting to the owner of the structure, but quality assurance *per se* does not produce concrete appropriate for the given conditions.

Factors governing the selection of mix proportions

It may be convenient at this stage to restate the basic objective: we are to determine the proportions of the most economical concrete mix that will be satisfactory both in the fresh and in the hardened state. To achieve this, we shall now consider the various factors of Fig. 14.1 and follow the sequence of decisions right down to the final choice of mix proportions. It may be recalled that water/cement ratio and strength have already been discussed.

Durability

It has been stated, on more than one occasion, that the selection of mix proportions must satisfy

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not only the strength requirements but must also ensure adequate durability. However, there does not yet exist a generally agreed and reliable approach to the selection of mix of proportions required for durability under any given conditions. One reason for this situation is the very wide range of exposure circumstances, including the extremely onerous conditions in very hot and notionally arid coastal areas. In these areas, protection of reinforcing steel from corrosion strongly affects the selection of mix proportions of the concrete in the cover zone.

The nowadays widely accepted recognition of a specific durability requirement in the mix selection is in contrast to the previous belief that reinforced concrete was inherently durable and would remain in service for a long time without repair: the maxim was ‘strong concrete is durable concrete’. For instance, British Standard Code of Practice CP 114 (1948)[14.12] stated: “No structural maintenance should be necessary for dense concrete constructed in accordance with this code.”

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Even the 1969 edition of the same code of practice[14.10] limited itself to the statement: "The greater the severity of the exposure the higher the quality of concrete required . . ."

The factors influencing durability were discussed in Chapters 10 and 11; simple means of specifying the mix proportions so as to achieve the required durability will now be considered. The word 'simple' is used in recognition of the fact that the penetrability of concrete, which plays a crucial role in its durability, cannot be directly controlled in the production of concrete. Hence, reliance is necessary on the water/cement ratio, cement content, compressive strength – indeed, any one of these, or two, or all three at the same time, can be used. It is worth reiterating that, whatever mix proportions are chosen, the concrete must be capable of full compaction using the means available, and that such compaction must be achieved in practice.

The American Concrete Institute Building Code 318-02[14.8] devotes a separate chapter to

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durability requirements. With respect to exposure to freezing and thawing, ACI 318-02 requires, for normal weight concrete, a specified maximum water/cement ratio, and for lightweight aggregate concrete, a specified minimum strength; these are shown in Table 14.3. The reason for this difference in approach in the two types of concrete is that it is not practicable to control the water/cement ratio of the lightweight aggregate concrete. In addition, all concretes require air entrainment, the total air content being specified according to the conditions of exposure and the maximum size of aggregate used (see Table 11.3). The limitations on the amounts of fly ash and ground granulated blastfurnace slag when de-icing agents are used, prescribed by ACI 318-02[14.8] are given on p. 668.

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Table 14.3. Requirements of ACI 318-02[14.8] for Concrete Exposed to Freezing and Thawing

Exposure condition	*Maximum water/cement ratio for normal weight concrete*	*Minimum compressive strength of test cylinders for lightweight aggregate concrete*	
		MPa	*psi*
Exposed to water when low permeability required	0.50	28	4000
Exposed to freezing and thawing in a moist condition or to de-icing agents	0.45	31	4500
Exposed to chlorides from de-icing agents or from sea-water splash or spray corrosion protection required	0.40	34	5000

The requirements suggested in the U.S. Strategic Highway Research Program,[14.14] are more stringent than those of ACI 318-02: the water/cement ratio is not to exceed 0.35 so as to ensure discontinuous capillaries in the cement paste after one day's curing.

British Standard BS 5328-1 : 1997 contains an elaborate classification of exposure and recommends appropriate values of the maximum wa-

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ter/cement ratio, minimum cement content, and 28-day compressive strength. These recommended values are likely to be inadequate in climates other than temperate, and even under British conditions they may be somewhat optimistic. One of these recommendations is to the effect that a maximum water/cement ratio of 0.55, a minimum cement content of 325 kg/m^3, and a 28-day characteristic strength of 40 MPa (measured on cubes) should be used for concrete occasionally exposed to sea-water spray or de-icing agents or to severe freezing conditions while wet. These recommendations are not endorsed in the present book. All parts of BS 5328 have been withdrawn and replaced by BS EN 206-1 : 2000 and BS 8500 : Parts 1 & 2 : 2002.

According to BS 5328-1 : 1997 (withdrawn), a satisfactory strength "will generally ensure that the limits on free water/cement ratio and cement content will be met without further checking". In view of the wide range of cements available world-wide, this assumption may not be valid

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and it is not recommended in this book. In particular, some cementitious materials increase the compressive strength of concrete but the higher strength does not necessarily contribute to the resistance to freezing and thawing or to carbonation.[14.9] It is very much doubted that strength alone can be used as an indicator of durability.

With respect to sulfate attack, BS 5328-1 : 1997 recommends both a maximum value of the water/cement ratio and a minimum cement content, and also specifies the type of cement to be used for various concentrations of sulfates in groundwater or in soil. It is arguable that there is some inconsistency between the approach of the same British Standard to the requirements for resistance to sulfate attack and to other conditions of exposure for which strength alone can be used as a measure of compliance. This situation may well be the consequence of the combination of our inadequate understanding of the behaviour of concrete under various forms of attack, coupled with

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practical difficulties of control of all aspects of mix ingredients and of their proportions.

For resistance to sulfate attack, ACI 225R-91[14.17] prescribes a maximum water/cement ratio between 0.45 and 0.50 for the categories of exposure given in Table 10.7. The cementitious materials to be used are also prescribed.

The cement content as such does not control durability: it does so only in so far as it influences the water/cement ratio, which, in turn, influences strength. Moreover, considering reliance on a minimum cement content, it should be remembered that, while it is expressed in kilograms per cubic metre of concrete, durability depends largely on the properties of hydrated cement paste. Thus, it is the cement content of the paste that is relevant, and the volume of the cement paste (in a unit volume of concrete) is smaller the larger the maximum size of aggregate. For this reason, BS 5328-1 : 1997 recommends an adjustment of the cement content as a function of the maximum size of aggregate in the follow-

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ing manner. The specified cement content of a mix with a maximum size of aggregate of 20 mm should be increased by 20 kg/m^3 when the maximum size of aggregate is 14 mm, and by 40 kg/m^3 when it is 10 mm. Conversely, when the maximum size of aggregate is 40 mm, the cement content can be reduced by 30 kg/m^3 as compared with concrete containing 20 mm aggregate. It may be observed that, in the French approach, the cement content is assumed to be inversely proportional to the fifth root of the maximum aggregate size; this ascribes to the maximum size of aggregate a large influence on the required cement content.

If durability requires a certain maximum water/cement ratio, but structural requirements are for a value of strength which can be readily achieved at a higher water/cement ratio, a set of incompatible values of strength and water/cement ratio should not be specified. Rather, a higher specified strength should be used so as to correspond to the water/cement ratio required for reas-

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ons of durability. In this manner, there will be no temptation on the part of the concrete producer to disregard the water/cement ratio and to rely solely on an adequate level of strength.[14.8] This higher strength should be established prior to the commencement of the structural design so that advantage can be taken of the use of a higher strength of concrete in the structural design.

It has to be stated that little is known about the variability of the water/cement ratio in in situ concrete. According to Gaynor,[14.13] on well-controlled jobs the standard deviation of the water/cement ratio is between 0.02 and 0.03. This high variability may be a reflection of the fact that the total amount of free water in a given batch is not easily ascertained. One reason for this is that, even if the moisture content of aggregate is measured accurately, the result may not be representative of the given batch.

The water/cement ratio alone does not determine the resistance of concrete to chloride penetration: the type of cementitious material used

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greatly affects the penetrability of the resulting concrete. In particular, concretes containing both ground granulated blastfurnace slag and silica fume offer particularly good resistance.[14.1] This situation exemplifies the difficulty of basing the specification for durability on strength alone. The same argument applies to the use of cement content alone.

The nature of the cementitious materials to be used is of vital importance also under other conditions of exposure. When concrete is to be subjected to chemical attack, a suitable type of cement must be used but, if resistance to freezing and thawing is the only durability requirement, the choice of the type of cement is governed by other considerations, for instance, the development of early strength or of a high heat of hydration for concreting in cold weather. Indeed, the beneficial properties of the various cementitious materials, discussed in Chapter 13, should be exploited in the selection of the cement. However, the limits on the maximum content of fly ash and

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ground granulated blastfurnace slag, imposed by ACI 318-02)[14.8] for concrete exposed to de-icing agents, should be remembered (see p. 668).

Because the type of cement affects the early development of strength, it may be necessary, with some cements, to use a low water/cement ratio to ensure a satisfactory strength at early ages. Thus, strength, type of cement, and durability determine between them the water/cement ratio required – one of the essential quantities in the calculation of mix proportions.

Workability

So far, we have considered the requirements for the concrete to be satisfactory in the hardened state but, as said before, properties when being transported, possibly pumped, and placed are equally important. One essential at this stage is a satisfactory workability. Selection of mix proportions which do not permit the achievement of appropriate workability totally defeats the purpose of rational mix proportioning.

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The workability that is considered desirable depends on two factors. The first of these is the minimum size of the section to be concreted and the amount and spacing of reinforcement; the second is the method of compaction to be used.

It is clear that when the section is narrow and complicated, or when there are numerous corners or inaccessible parts, the concrete must have a high workability so that full compaction can be achieved with a reasonable amount of effort. The same applies when embedded steel sections or fixtures are present, or when the amount and spacing of reinforcement make placing and compaction difficult. Because these features of the structure are determined during its design, the necessary workability must be ensured in the selection of mix proportions. On the other hand, when no such limitations are present, workability may be chosen within fairly wide limits, but the means of transportation and compaction must be decided upon accordingly; it is important that the prescribed method of compaction is used during the

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entire progress of construction. Advice on the appropriate value of slump and of means of compaction for various types of construction is given in BS 5328-1 : 1997.

A property closely related to workability is cohesiveness. This depends largely on the proportion of fine particles in the mix and, especially in lean mixes, attention must be paid to the grading of the aggregate at the fine end of the scale. It is sometimes necessary to make several trial mixes with different proportions of fine to coarse aggregate in order to find the mix with an adequate cohesiveness.

While every mix should be cohesive so that uniform and well-compacted concrete can be obtained, the exact importance of cohesiveness varies. For instance, where concrete has to be hauled without agitation over a long distance or is handled down a chute, or has to pass through reinforcement, possibly to some inaccessible corner, it is essential that the mix be truly cohesive. In cases when the conditions leading to se-

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gregation are less likely to be encountered, cohesion is of smaller importance, but a mix which segregates easily must never be used.

Maximum size of aggregate

In reinforced concrete, the maximum size of aggregate which can be used is governed by the width of the section and the spacing of the reinforcement. With this proviso, it used to be considered desirable to use as large a maximum size of aggregate as possible. However, it now seems that the improvement in the properties of concrete with an increase in the size of aggregate does not extend beyond about 40 mm (1 in.) so that the use of larger sizes may not be advantageous (see p. 174). In particular, in high performance concrete, the use of aggregate larger than 10 to 15 mm ($\frac{3}{8}$ to $\frac{5}{8}$ in.) is counter-productive (see p. 678).

Furthermore, the use of a larger maximum size means that a greater number of stockpiles has to be maintained and the batching operations be-

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come correspondingly more complicated. This may be uneconomical on small sites but, where large quantities of concrete are to be placed, the extra handling cost may be offset by a reduction in the cement content of the mix.

The choice of the maximum size may also be governed by the availability of materials and by their cost. For instance, when various sizes are screened from a pit, it is generally preferable not to reject the largest size, provided this is acceptable on technical grounds.

Grading and type of aggregate

Most of the remarks in the preceding section apply equally to the considerations of aggregate grading because it is often more economical to use the material available locally, even though it requires a richer mix (but provided it will produce concrete free from segregation) rather than to bring in a better graded aggregate from farther afield.

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It has been stressed repeatedly that, although there are certain desiderata for a good grading curve, no ideal gradings exist, and excellent concrete can be made with a wide range of aggregate gradings.

The grading influences the mix proportions for a desired workability and the water/cement ratio: the coarser the grading the leaner the mix which can be used, but this is true within certain limits only because a very lean mix will not be cohesive without a sufficient amount of fine material.

It is possible, however, to reverse the direction of choice: if the cement content is fixed (e.g. a lean mix may be essential for massive concrete construction) then a grading must be chosen such that concrete of given water/cement/aggregate proportions and having a satisfactory workability can be made. Clearly, there are limits on grading outside which it is not possible to make good concrete.

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The influence of the type of aggregate should also be considered because its surface texture, shape and allied properties influence the aggregate/cement ratio for a desired workability and a given water/cement ratio. In selecting a mix, it is essential, therefore, to know at the outset what type of aggregate is available.

An important feature of satisfactory aggregate is the uniformity of its grading. In the case of coarse aggregate, this is achieved comparatively easily by the use of separate stockpiles for each size fraction. However, considerable care is required in maintaining the uniformity of grading of fine aggregate, and this is especially important when the water content of the mix is controlled by the mixer operator on the basis of a constant workability: a sudden change toward finer grading requires additional water for the workability to be preserved, and this means a lower strength of the batch concerned. Also, an excess of fine aggregate may make full compaction impossible and thus lead to a drop in strength.

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Thus, while narrow specification limits for aggregate grading may be unduly restrictive, it is essential that the grading of aggregate varies from batch to batch within prescribed limits only.

Cement content

All the factors considered up to now, including water/cement ratio, will determine between them the aggregage/cement ratio or the cement content of the mix. To obtain a clear picture of the various influences, Fig. 14.1 should once again be consulted.

The choice of the cement content is made either on the basis of experience or alternatively from charts and tables prepared from comprehensive laboratory tests. Such tables are no more than a guide to the mix proportions required because they apply fully only to the actual aggregates used in their derivation. Moreover, recommended proportions are usually based on aggregate gradings which have been found to be satisfactory. When a significant departure from such

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gradings is necessary, it may be useful to bear in mind some of the guidance 'rules' established as far back as 1950. One of these 'rules' is: when there is an excess of particles smaller than 600 μm (No. 30 ASTM) sieve, the quantity of material passing the 4.76 mm ($\frac{3}{16}$ in.) sieve should be reduced by an amount up to 10 per cent of the total aggregate. On the other hand, when there is an excess of particles in the 1.20 to 4.76 mm (No. 16 ASTM to $\frac{3}{16}$ in.) size range, the quantity of fine aggregate should be increased. However, fine aggregate with a large excess of particles between 1.20 mm (No. 16 ASTM) and 4.76 mm ($\frac{3}{16}$ in.) sieves produces a harsh mix and may require a higher cement content for a satisfactory workability.

In comparing various mixes, it is sometimes convenient to convert rapidly the aggregate/cement ratio into the cement content or vice versa: Fig. 14.8 makes such a conversion very easy.

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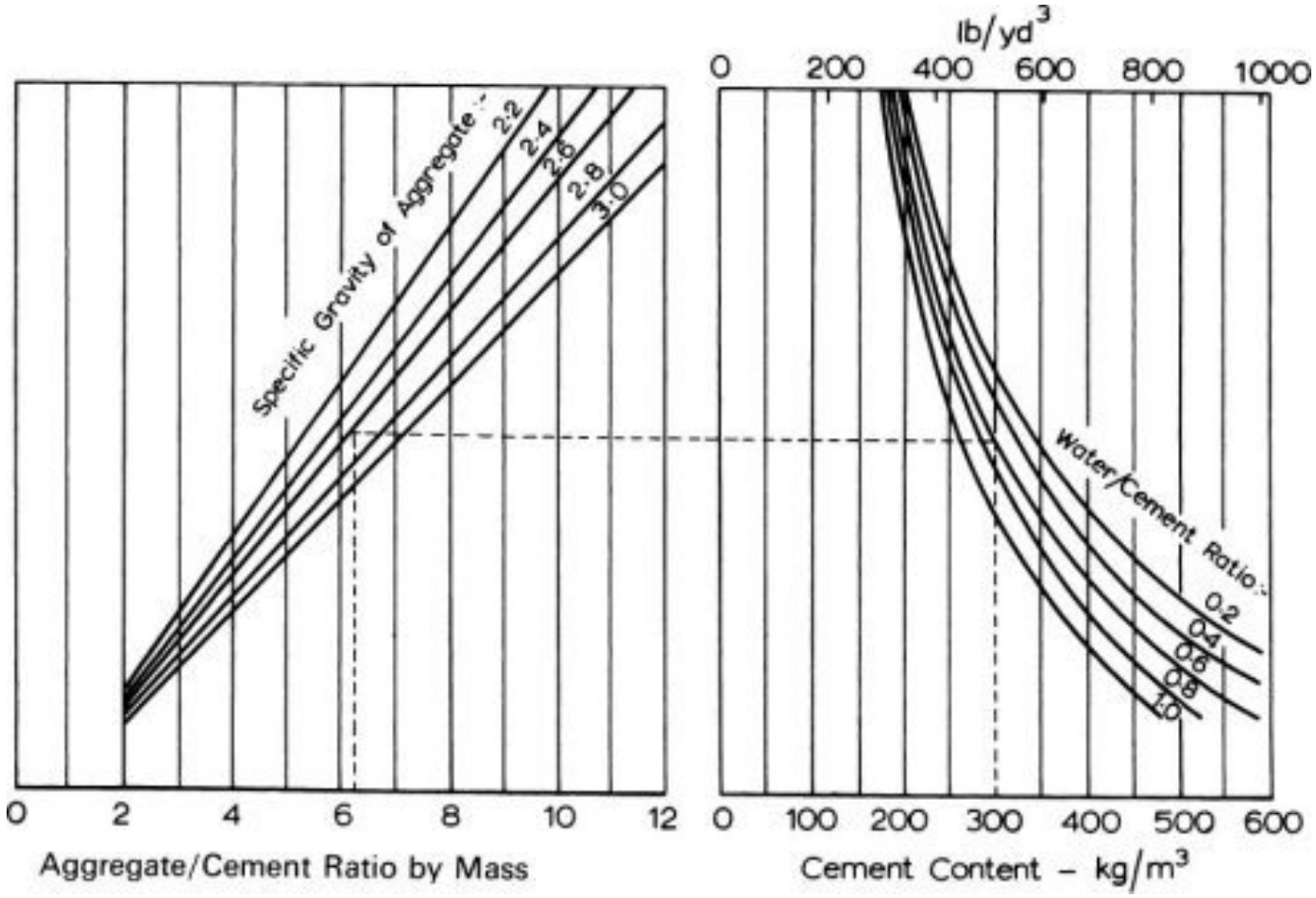


Fig. 14.8. Conversion chart for aggregate/cement ratio and cement content (courtesy of Cement and Concrete Association)

Mix proportions and quantities per batch

With the water/cement ratio and the cement content known, there is no difficulty in determining the proportions of cement, water, and aggregate. In practice, the aggregate is supplied from at least two stockpiles, and the quantities of aggregate

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of each size have to be given separately. This presents no problem because, in finding a suitable grading, we already had to calculate the proportions of the different size fractions of aggregate. The details of calculation are given in the example on p. 752.

For practical purposes, the mix quantities are given in kilograms or pounds per batch. When cement is supplied in bulk, we choose the batch quantities so that their sum is equal to the capacity of the mixer. When cement is supplied in bags, and there is no provision for weighing it, it is preferable to choose the batch quantities so that the mass of cement per batch is one bag or its multiple. The mass of cement is then known accurately. In exceptional cases, a half-bag can be used, but other fractions cannot be reliably determined and should never be used. Bag sizes are given on p. 7.

If a concrete mix of certain proportions is to be modified by the use of an admixture, some changes in the quantity of some of the ingredients

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are necessary. An important principle is to maintain the volume of the coarse aggregate in a unit volume of concrete, and to adjust only the volume of fine aggregate. This is done by changing the quantity of fine aggregate on an absolute volume basis by an amount equal and opposite to the changes in the volume of water, entrained air and cement. The liquid part of any admixture is considered to be a part of the mix water.

Calculation by absolute volume

The procedure so far described leads to the determination of values of the water/cement ratio and the cement content or the aggregate/cement ratio, and also of the relative proportions of the aggregates of various sizes, but does not give the volume of fully compacted concrete produced by these materials. This volume is obtained by a simple calculation, using the so-called absolute volume method, which assumes that the volume of compacted concrete is equal to the sum of the absolute volumes of all ingredients.

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It is usual to calculate the quantities of ingredients to produce 1 cubic metre or 1 cubic yard of concrete. Then, if W, C, A_1, and A_2 are the required quantities by mass of water, cement, fine aggregate, and coarse aggregate, respectively, we have, for the cubic metre:

$$\frac{W}{1000}+\frac{C}{1000\rho_c}+\frac{A_1}{1000\rho_1}+\frac{A_2}{1000\rho_2}=1$$

where ρ with the appropriate suffix respresents the specific gravity of each material. In the Imperial or American system of units of measurement, since the density of water (62.4) is expressed in pounds per cubic foot, the total volume of 1 cubic yard has to be expressed as 27 cubic feet. For the cubic yard, therefore the corresponding equation is:

$$\frac{W}{62.4}+\frac{C}{62.4\rho_c}+\frac{A_1}{62.4\rho_1}+\frac{A_2}{62.4\rho_2}=27.$$

The mix proportioning calculations give the values of W/C, $C/(A_1 + A_2)$ and A_1/A_2, whence the values of W, C, A_1, and A_2 can be found.

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When an additional cementitious material, possibly with a specific gravity different from that of Portland cement, is present, or when the coarse or fine aggregate is in more than one stockpile, additional terms of similar form are added to the equation. When entrained air is present, and its percentage is, say, *a* per cent of the volume of concrete, the right-hand side of the 'cubic yard' equation would read:

$$27 \times \left[1 - \frac{a}{100}\right].$$

For the 'cubic metre' equation, 27 is replaced by 1.

In the preceding equations, *C* represents the cement content in kilograms per cubic metre or pounds per cubic yard of the concrete, and *W* is the water content in the same units; the latter must not be confused with the water/cement ratio. In the United States, the cement content used to be expressed in sacks of cement per cubic yard of

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concrete and was referred to as *cement factor*; a sack weighs 94 lb.

If the aggregate contains free moisture whose mass is, say, m per cent of the mass of the dry aggregate then the values of the mass of the *added* water W and of (wet) aggregate must be adjusted. The mass of free water in A' kg (lb) of aggregate is x such that:

$$\frac{m}{100} = \frac{x}{A' - x}$$

and the mass of dry aggregate is $A = A' - x$. Hence, $x = Am/100$. This mass is added to A to give the mass of wet aggregate per batch, $A(1 + m/100)$, and is subtracted from W to give the mass of added water, $W - Am/100$.

Generally, each size fraction of aggregate has a different moisture content, and the correction should be applied to A_1, A_2, etc., with an appropriate value of m.

In the production of concrete of low strength, the determination of the moisture content of ag-

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gregate can be dispensed with if the grading of aggregate is reasonably constant and weigh-batching is used. Under those circumstances, a change in workability caused by a variation in the moisture content of aggregate can be prevented by an experienced mixer operator who can adjust the amount of added water so that the workability, as judged by eye, remains constant. The water/cement ratio remains then also sensibly constant. It should be stressed, however, that, if concrete of specified proportions is to be consistently produced, then it is essential for all ingredients, including moisture in the aggregate, to be precisely determined.

In the case of volume batching, no correction for moisture content need be made in the case of coarse aggregate, but the bulking of fine aggregate must be allowed for (see p. 134). The quantity of added water must be adjusted by the mixer operator as in the case of weigh-batching.

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Combining aggregates to obtain a type grading

While there are no ideal gradings – a point repeatedly made – it may be desirable or required to proportion the available materials in such a way that the grading of the combined aggregate is similar to a specific curve or lies between given limits. This can be done by calculation or graphically. Both procedures are best illustrated by means of examples.

In these examples, it is assumed that all aggregates have the same specific gravity. The physical composition of concrete is, however, based on volumetric proportions. It follows that, if the specific gravities of the different size fractions differ appreciably from one another, the proportions required should be adjusted accordingly. This approach is necessary in the calculation of mix proportions of lightweight aggregate mixes when lightweight coarse aggregate and normal weight fine aggregate are used.

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Suppose the gradings of the fine aggregate and the two coarse aggregate size fractions are as listed in Table 14.4, and we are to combine the materials so as to approximate to the coarsest grading of Fig. 3.15 (curve 1). On this curve, 24 per cent of the total aggregate passes the 4.75 mm ($\frac{3}{16}$ in.) sieve, and 50 per cent passes the 19.0 mm ($\frac{3}{4}$ in.) sieve.

Table 14.4. Example of Combining Aggregates to Obtain a Type Grading

Sieve size		Cumulative percentage passing for:			(1) × 1	(2) × 0.94	(3) × 2.59	(4) + (5) + (6)	Grading of combined aggregate (7) ÷ 4.53
mm or μm	in. or No.	Fine aggregate (1)	19.0 to 4.75 mm ($\frac{3}{4}$ to $\frac{3}{16}$ in.) (2)	38.1 to 19.0 mm ($1\frac{1}{2}$ to $\frac{3}{4}$ in.) (3)	(4)	(5)	(6)	(7)	(8)
38.1	$1\frac{1}{2}$	100	100	100	100	94	259	453	100
19.0	$\frac{3}{4}$	100	99	13	100	93	34	227	50
9.50	$\frac{3}{8}$	100	33	8	100	31	21	152	34
4.75	$\frac{3}{16}$	99	5	2	99	5	5	109	24
2.36	8	76	0	0	76	0	0	76	17
1.18	16	58			58			58	13
600	30	40			40			40	9
300	50	12			12			12	3
150	100	2			2			2	$\frac{1}{2}$

Let x, y, z be the proportions of fine, 19.0 to 4.75 mm ($\frac{3}{4}$ to $\frac{3}{16}$ in.), and 38.1 to 19.0 mm ($1\frac{1}{2}$ to $\frac{3}{4}$ in.) aggregates, respectively. Then, to satisfy the condition that 50 per cent of the combined

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aggregate passes the 19.0 mm ($\frac{3}{4}$ in.) sieve, we have:

$$1.0x + 0.99y + 0.13z = 0.5(x + y + z).$$

The condition that 24 per cent of the combined aggregate passes the 4.75 mm ($\frac{3}{16}$ in.) sieve can be written:

$$0.99x + 0.05y + 0.02z = 0.24(x + y + z).$$

From these two equations we find:

$$x:y:z = 1:0.94:2.59$$

that is, the three aggregates are combined in the proportions 1:0.94:2.59.

To find the grading of the combined aggregate we multiply columns (1), (2), and (3) of Table 14.4 by 1, 0.94, and 2.59, respectively, the products being shown in columns (4), (5), and (6). We now add these three columns (column 7) and divide the sum by 1 + 0.94 + 2.59 = 4.53. The

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result, given in column (8), is the grading of the combined aggregate. The grading is given to the nearest 1 per cent as, because of the variability of the materials, any higher apparent accuracy has no meaning.

Figure 14.9 shows the grading of the combined aggregate, together with the type curve which we are seeking to follow. Deviations are apparent, and indeed unavoidable, because agreement with the type curve is generally possibly only at specified points.

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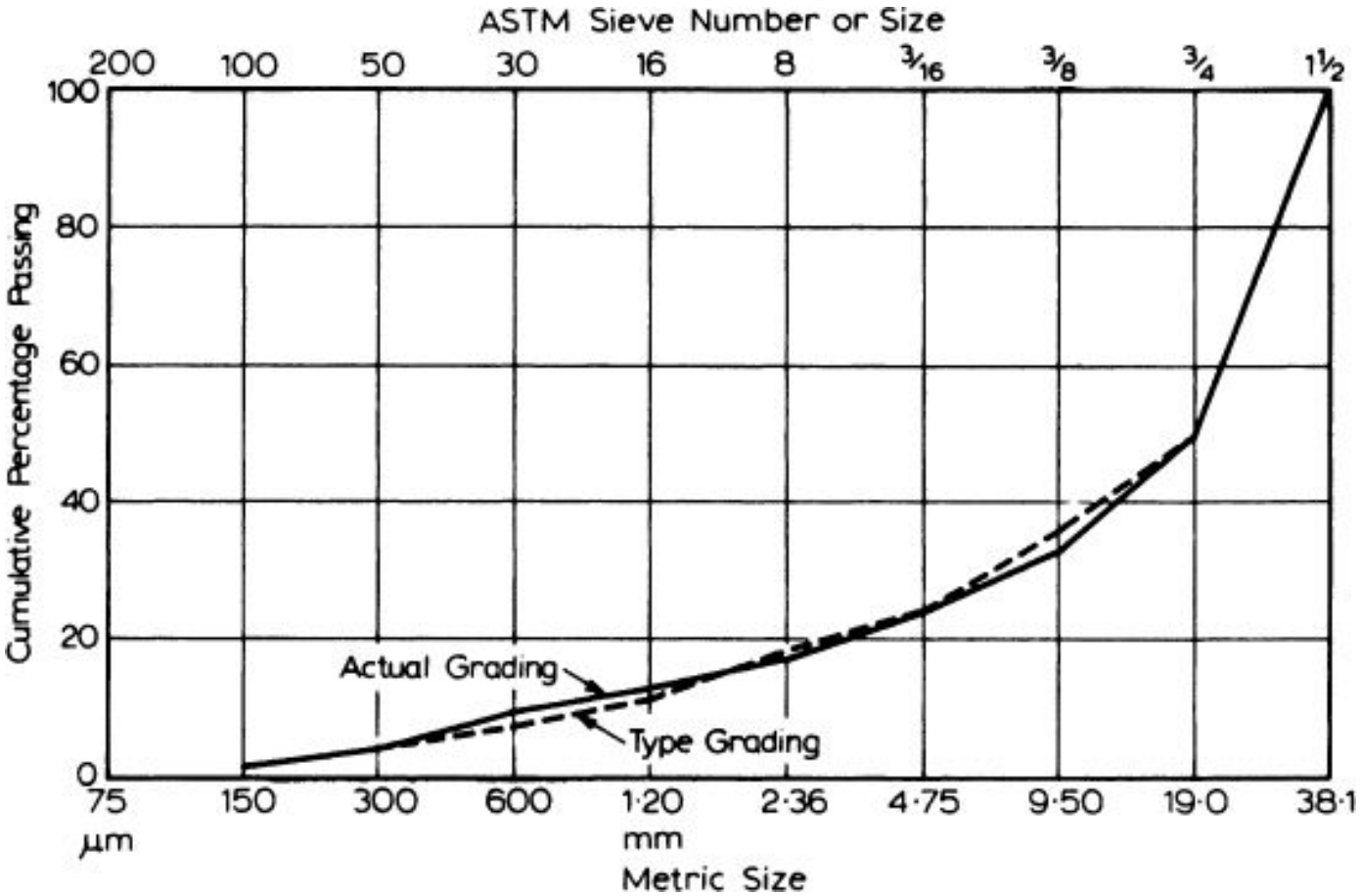


Fig. 14.9. Grading of the aggregate for the example of Table 14.4

The graphical method is shown in Fig. 14.10. The two coarse aggregates are combined first, using the percentage passing the 19.0 mm ($\frac{3}{4}$ in.) sieve as a criterion. Percentage passing is marked along three sides of a square. The values for the two coarse aggregates are entered on two opposite sides, and the points corresponding to the same sieve size are joined by straight lines. A

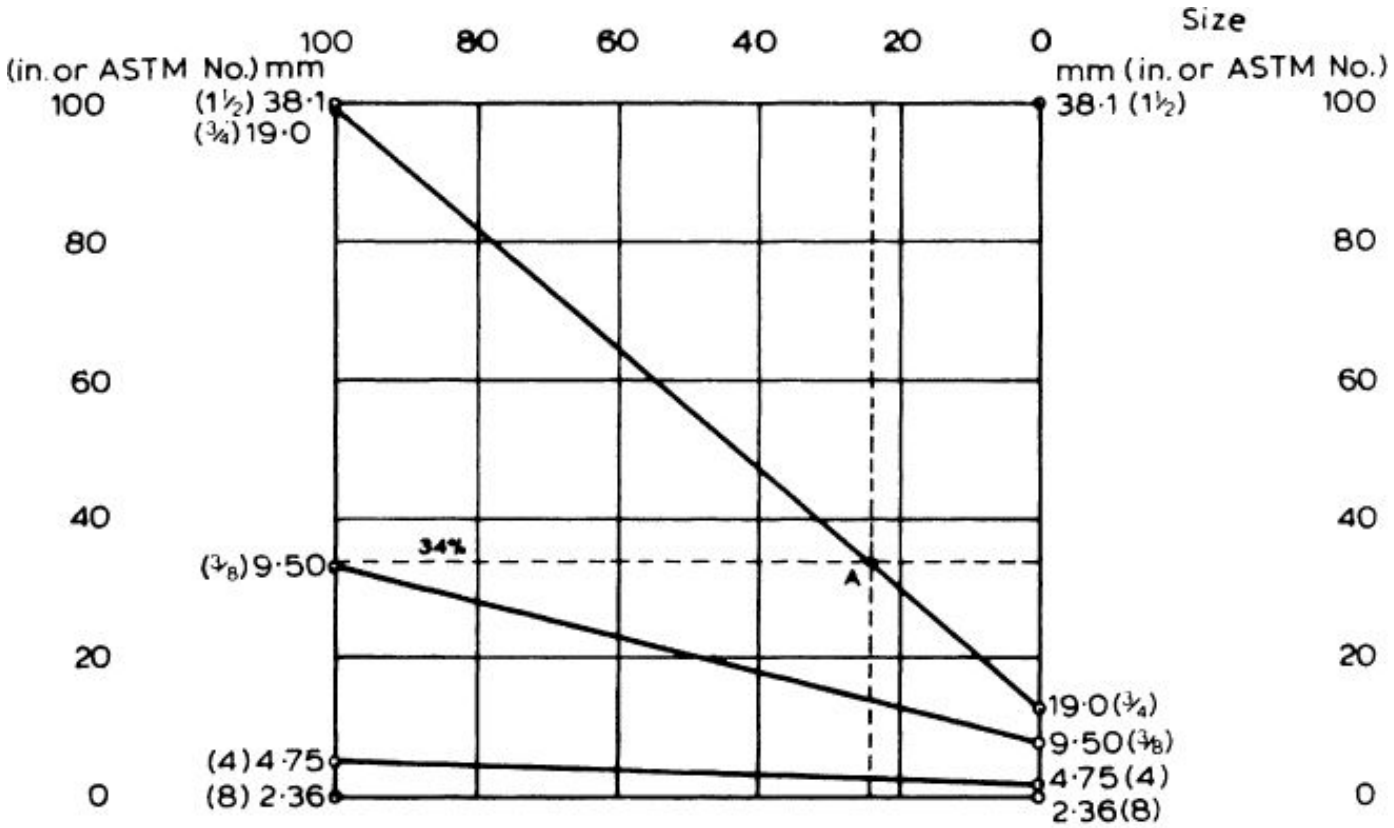
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vertical line is now drawn through the point where the line joining the 19.0 mm ($\frac{3}{4}$ in.) values intersects the horizontal line representing the correct percentage of aggregate smaller than 19.0 mm ($\frac{3}{4}$ in.). In our case, (50 – 24) = 26 parts of aggregate coarser than 9.50 mm ($\frac{3}{16}$ in.) sieve are to pass the 19.0 mm ($\frac{3}{4}$ in.) sieve while 50 parts are to be retained. The ratio is thus 26:(50 + 26), or 34 per cent of *all* coarse aggregate. A horizontal line is, therefore, drawn through the 34 per cent point to intersect the 19.0 mm ($\frac{3}{4}$ in.) line at *A*. A vertical line through *A* gives the quantity of material 19.0 to 4.75 mm ($\frac{3}{4}$ to $\frac{3}{16}$ in.) as a percentage of the total coarse aggregate. In Fig. 14.10(a) this value is 24 per cent. The vertical line gives also the grading of the combined coarse aggregate, and this is combined with the fine aggregate in a similar manner to that already described (Fig. 14.10(b)). We find that 22 parts of fine aggregate are to be combined with 78 parts of aggregate coarser than 4.75 mm ($\frac{3}{16}$ in.) sieve. The ag-

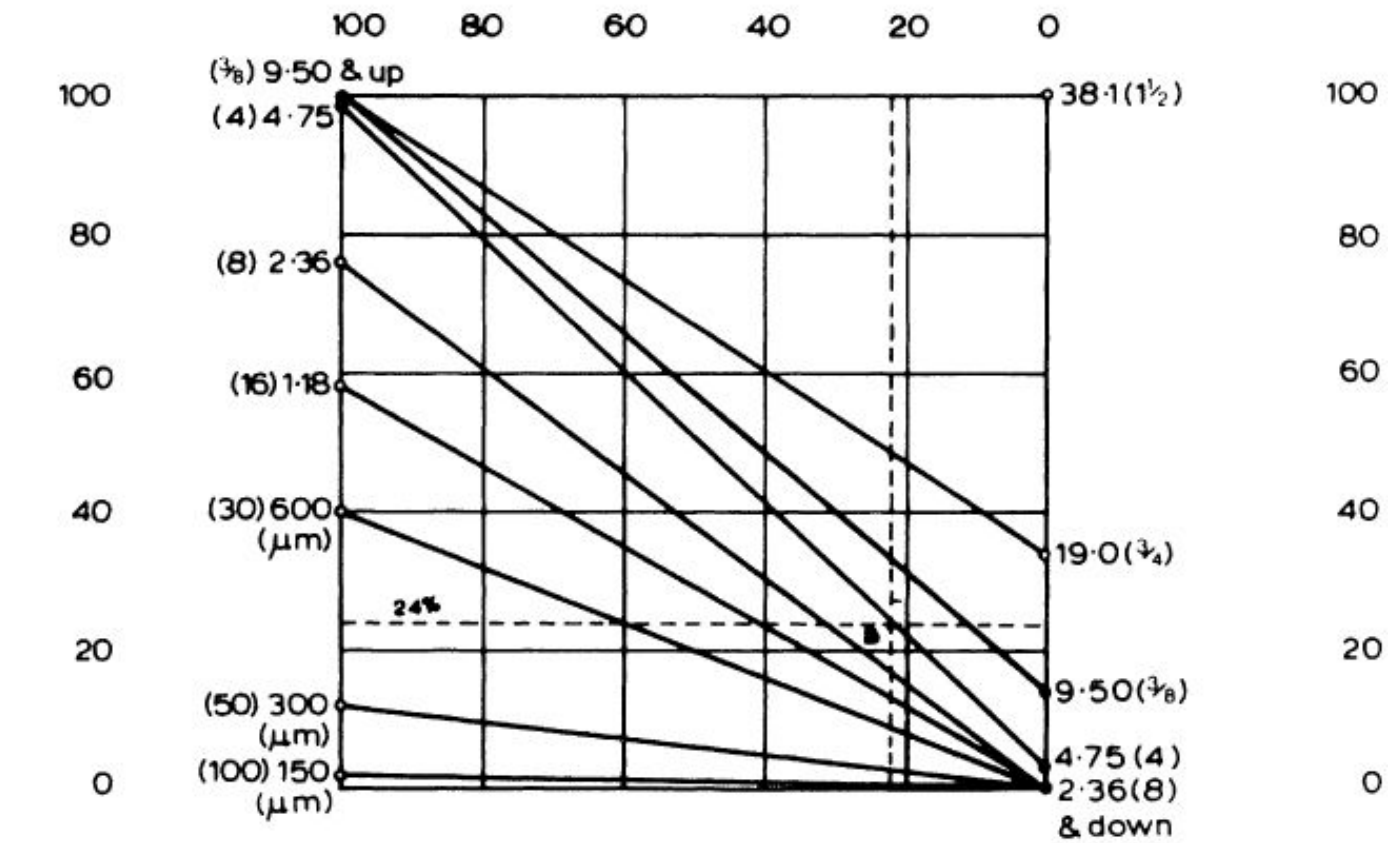
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gregate is thus to be proportioned as 22:(24/100) × 78:(76/100) × 78, or 1:0.85:1.69. The vertical line through *B* (Fig. 14.10(b)) gives the combined grading of aggregate obtained by proportioning the three aggregates in the ratio 1:0.85:2.69. This agrees with the grading obtained earlier by calculation, but both methods are approximations based on quantities passing only two specific sieve sizes.

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(a) Combining Coarse Aggregates



(b) Combining (a) with fine Aggregate

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Fig. 14.10. Graphical method of combining aggregates (example of Table 14.4)

It is possible to draw (in a figure of the type of Fig. 14.10(b)) envelopes of standard gradings: because any vertical line represents a possible grading it is immediately apparent whether or not a grading within the envelope can be obtained; the range of proportions is then given by a point similar to *B*, corresponding to any chosen vertical line.

American method of selection of mix proportions

The ACI Standard Practice ACI 211.1-91[14.5] describes a method of selection of mix proportions of concrete containing Portland cement alone or together with other cementitious materials, and containing also admixtures. It should be emphasized that the method provides a first approximation of mix proportions to be used in trial mixes. In essence, the method of ACI 211.1-91 consists

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of a sequence of logical, straightforward steps which take into account the characteristics of the materials to be used. These steps will now be described.

Step 1: Choice of slump

At the time of mix proportioning, the slump will have been determined by the exigencies of construction. It should be noted that slump should be specified not only at the minimum end, but a maximum value should also be specified. This is necessary to avoid segregation when the mix, which has not been selected to have a higher slump, suddenly becomes ‘wet’.

Step 2: Choice of maximum size of aggregate

This, too, will have been decided, usually by the structural designer, bearing in mind the geometric requirements of member size and spacing of reinforcement, or alternatively for reasons of availability.

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Step 3: Estimate of water content and air content

As discussed in Chapter 4, the water content required to produce a given slump depends on several factors: the maximum size of aggregate, its shape, texture, and grading; the content of entrained air; the use of admixtures with plasticizing or water-reducing properties; and the temperature of concrete. Tables relating slump to these properties have to be used, unless direct experience is available. One such table is Table 4.1. Alternatively, the values recommended by ACI 211.1-91[14.5] can be used; a selection of these is given in Table 14.5. For use in practice, notes to this table and comments in ACI 211.1-91, which are not reproduced here, should be taken into account.

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Table 14.5. Approximate Mixing Water and Air Content Requirements for Different Slumps and Nominal Maximum Sizes of Aggregates given in ACI 211.1-91[14.5]

Slump, mm	*Water, kg/m³ of concrete for indicated nominal maximum sizes of aggregate*							
	9.5	*12.5*	*19*	*25*	*37.5*	*50*	*75*	*150*
Non-air-entrained concrete								
25 to 50	207	199	190	179	166	154	130	113
75 to 100	228	216	205	193	181	169	145	124
150 to 175	243	228	216	202	190	178	160	—
Amount of entrapped air, per cent	3	2.5	2	1.5	1	0.5	0.3	0.2
Air-entrained concrete								
25 to 50	181	175	168	160	150	142	122	107
75 to 100	202	193	184	175	165	157	133	119
150 to 175	216	205	197	184	174	166	154	—
Total air content, (per cent) for:								
Improvement of workability	4.5	4.0	3.5	3.0	2.5	2.0	1.5	1.0
Moderate exposure	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
Extreme exposure	7.5	7.0	6.0	6.0	5.5	5.0	4.5	4.0

The values of Table 14.5 are typical for well-shaped angular aggregates having what is considered to be ‘good’ grading. When the coarse aggregate is rounded, the water requirement per cubic metre of concrete can be expected to be reduced by about 18 kg in the case of non-air-entrained concrete, and by 15 kg for air-entrained

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concrete. Water-reducing admixtures, and even more so superplasticizers, will significantly reduce the values of water given in Table 14.5. It should be remembered that the liquid part of admixtures constitutes a part of the mix water.

Table 14.5 also gives the values of the amount of entrapped air which can be expected. These are useful in the calculation of density of compacted concrete and of yield.

Step 4: Selection of water/cement ratio

There are two criteria for the selection of the water/cement ratio: strength and durability. As far as the compressive strength is concerned, the average value aimed at must exceed the specified 'minimum' strength by an appropriate margin (see p. 734). The term 'cement' refers to the total mass of cementitious materials used; their choice is governed by numerous factors, such as heat development, rate of gain of strength, and resistance to various forms of attack, so that the type of blended cement to be used has to be established

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at the outset of mix proportioning. It is for the actual cement to be used that the relation between strength and the water/cement ratio has to be established over a certain range of strengths.

As far as durability is concerned, the water/cement ratio may well be specified by the structural designer or by an appropriate design code. What is vital is that the water/cement ratio chosen is the *lower* of the two values emanating from strength and durability requirements.

When different cementitious materials are used, it should be remembered that they have varying values of specific gravity: typical values are 3.15 for Portland cement, 2.90 for ground granulated blastfurnace slag, and 2.30 for fly ash.

Step 5: Calculation of cement content

The outcome of Steps 3 and 4 gives the cement content directly: it is the water content divided by the water/cement ratio. If, however, from durability considerations, there is a requirement for

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a certain minimum cement content, the *larger* of the two values must be used.

Occasionally, from heat development considerations, the specification imposes a maximum cement content. Clearly, this must be observed. Heat development is of particular concern in mass concrete, and mix proportioning for that type of concrete is specifically covered in ACI 211.1-91.[14.5]

Step 6: Estimate of coarse aggregate content

Here, the assumption is made that the optimum ratio of the bulk volume of coarse aggregate to the total volume of concrete depends only on the maximum size of aggregate and on the grading of fine aggregate. The shape of the coarse aggregate particles does not directly enter the relation because, for instance, a crushed aggregate has a greater bulk volume for the same mass (that is, a lower bulk density) than a well-rounded aggregate. Thus, the shape factor is automatically taken into account in the determination of the

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bulk density. Table 14.6 gives values of the optimum volume of coarse aggregate when used with fine aggregates of different fineness moduli (see p. 154). This volume is converted into mass of coarse aggregate per cubic metre of concrete by multiplying the value from the table by the oven-dry rodded mass of the aggregate (in kg/m^3).

Table 14.6. Bulk Volume of Coarse Aggregate per Unit Volume of Concrete[14.5]

Maximum size of aggregate		*Bulk volume of oven-dry rodded coarse aggregate per unit volume of concrete for fineness modulus of fine aggregate of:*			
mm	*in.*	*2.40*	*2.60*	*2.80*	*3.00*
9.5	$\frac{3}{8}$	0.50	0.48	0.46	0.44
12.5	$\frac{1}{2}$	0.59	0.57	0.55	0.53
20	$\frac{3}{4}$	0.66	0.64	0.62	0.60
25	1	0.71	0.69	0.67	0.65
37.5	$1\frac{1}{2}$	0.75	0.73	0.71	0.69
50	2	0.78	0.76	0.74	0.72
75	3	0.82	0.80	0.78	0.76
150	6	0.87	0.85	0.83	0.81

The values given will produce a mix with a workability suitable for reinforced concrete construction. For less workable concrete, e.g. that used in road construction, the values may be increased by about 10 per cent. For more workable concrete, such as may be required for placing by pumping, the values may be reduced by up to 10 per cent.

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Step 7: Estimate of fine aggregate content

At this stage, the mass of fine aggregate is the only remaining unknown quantity. The absolute volume of this mass can be obtained by subtracting the sum of the absolute volumes of water, cement, entrained air, and coarse aggregate from the volume of concrete, that is, 1 m^3. For each ingredient, the absolute volume is equal to the mass divided by the absolute density of the material (in kg/m^3); the absolute density is the specific gravity of the material multiplied by the density of water (1000 kg/m^3).

The absolute volume of fine aggregate is converted into mass by multiplying this volume by the specific gravity of the fine aggregate and by the density of water.

Alternatively, the mass of fine aggregate can be obtained directly by subtracting the total mass of other ingredients from the mass of a unit volume of concrete, if this can be estimated from

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experience. This approach is slightly less accurate than the absolute volume method.

Step 8: Adjustments to mix proportions

As in any process of selection of mix proportions, trial mixes have to be made. Advice, by way of some rules of thumb, for adjustments to the mix, is given in ACI 211.1-91.[14.5] In general terms, it is important to remember that, if workability is to be changed, but the strength is to remain unaffected, the water/cement ratio must remain unaltered. Changes can be made in the aggregate/cement ratio or, if suitable aggregates are available, in the grading of the aggregate; the influence of grading on workability was discussed in Chapter 3.

Conversely, changes in strength but not in workability are made by varying the water/cement ratio with the water *content* of the mix remaining unaltered. This means that a change in the water/cement ratio must be accompanied by a

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change in the aggregate/cement ratio so that the mass ratio

$$\frac{\text{water}}{\text{water} + \text{cement} + \text{aggregate}}$$

is approximately constant.

The above mix proportioning method of the American Concrete Institute can readily be programmed for computer use; an example of manual calculation is given later in this section.

Example

We require a mix with a *mean* 28-day compressive strength (measured on standard cylinders) of 35 MPa and a slump of 50 mm, ordinary Portland cement being used. The maximum size of well-shaped, angular aggregate is 20 mm, its bulk density is 1600 kg/m^3, and its specific gravity is 2.64. The available fine aggregate has a fineness modulus of 2.60 and a specific gravity of 2.58. No air entrainment is required. For the sake of

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completeness, all steps, even when obvious, will be given.

Step 1: A slump of 50 mm is specified.

Step 2: The maximum size of aggregate of 20 mm is specified.

Step 3: From Table 14.5 for a slump of 50 mm and a maximum size of aggregate of 20 mm (or 19 mm), the water requirement is approximately 190 kg per cubic metre of concrete.

Step 4: From past experience, a water/cement ratio of 0.48 is expected to result in concrete with a compressive strength, measured on cylinders, of 35 MPa. There are no special durability requirements.

Step 5: The cement content is 190/0.48 = 395 kg/m^3.

Step 6: From Table 14.6, when used with a fine aggregate having a fineness modulus of 2.60, the bulk volume

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of oven-dry rodded coarse aggregate with a maximum size of 20 mm is 0.64. Given that the bulk density of the coarse aggregate is 1600 kg/m^3, the mass of coarse aggregate is $0.64 \times 1600 = 1020$ kg/m^3.

Step 7: To calculate the mass of fine aggregate, we need first to calculate the volume of all the other ingredients. The required values are as follows.

Volume of water is 190/1000	$= 0.190$ m^3
Solid volume of cement, assuming usual specific gravity of 3.15, is $395/(3.15 \times 1000)$	$= 0.126$ m^3
Solid volume of coarse aggregate is $1020/(2.64 \times 1000)$	$= 0.396$ m^3
Volume of entrapped air, given in Table 14.5, is 0.02×1000	$= 0.020$ m^3
Hence, total volume of all ingredients except fine aggregate	$= 0.732$ m^3
Therefore, the required volume of fine aggregate is $1.000 - 0.732$	$= 0.268$ m^3
Hence, the mass of fine aggregate is $0.268 \times 2.58 \times 1000$	$= 690$ kg/m^3.

From the various steps, we can list the estimated mass of each of the ingredients in kilograms per cubic metre of concrete as follows:

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Water	190
Cement	395
Coarse aggregate, dry	1020
Fine aggregate, dry	690
Therefore, the density of concrete is	2295 kg/m^3.

Mix selection for no-slump concrete

The ACI 211.1-91[14.5] method of selection of mix proportions is intended for use with concretes having a slump of at least 25 mm (1 in.). For no-slump concrete, some modifications are required; these are given in ACI 211.3-75 (Revised 1987) (Reapproved 1992).[14.4]

The primary modification is applied to the water requirement given in Table 14.5. The values in that table for concrete with a slump of 75 to 100 mm (3 to 4 in.) are taken as reference values. Assigning a relative value of 100 per cent to those reference values of water requirement (in kg/m^3), the water requirement at other values of workability can be taken as a percentage given in Table 14.7. Three categories of no-slump concrete are

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recognized: extremely dry, very stiff, and stiff. The same table also gives relative values of water requirement at higher workabilities.

Table 14.7. Relative Mixing Water Requirements for Concretes with Different Workabilities[14.4]

Description	*Workability*				*Relative value of water content per cent*
	Slump		*Vebe time s*	*Compacting factor*	
	mm	*in.*			
Extremely dry	—	—	32–18	—	78
Very stiff	—	—	18–10	0.70	83
Stiff	0–25	0–1	10–5	0.75	88
Stiff plastic	25–75	1–3	5–3	0.85	92
Plastic (reference)	75–125	3–5	3–0	0.90	100
Flowing	125–175	5–7	—	0.95	106

The second modification to the procedure of ACI 211.1-91 for the purpose of mix selection for no-slump concrete is in the values of the bulk volume of coarse aggregate per unit volume of concrete: those given in Table 14.6 have to be multiplied by factors listed in Table 14.8. Further details are given in ACI 211.1-91. Otherwise, the

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procedure for no-slump concrete is similar to the procedure of 211.1-91, described earlier.

Table 14.8. Factors to be Applied to the Volume of Coarse Aggregate Calculated on the Basis of Table 14.6 for Mixes of Different Workabilities[14.4]

Consistency	*Factor for maximum size of aggregate of:*				
	10 mm ($\frac{3}{8}$ in.)	*12.5 mm ($\frac{1}{2}$ in.)*	*20 mm ($\frac{3}{4}$ in.)*	*25 mm (1 in.)*	*40 mm ($1\frac{1}{2}$ in.)*
Extremely dry	1.90	1.70	1.45	1.40	1.30
Very stiff	1.60	1.45	1.30	1.25	1.25
Stiff	1.35	1.30	1.15	1.15	1.20
Stiff plastic	1.08	1.06	1.04	1.06	1.09
Plastic (reference)	1.00	1.00	1.00	1.00	1.00
Flowing	0.97	0.98	1.00	1.00	1.00

Mix selection for flowing concrete

Some special comments with respect to flowing concrete should now be made. First of all, flowing concrete is described by ASTM C 1017-07 as concrete with a slump greater than 190 mm ($7\frac{1}{2}$ in.) which has a cohesive nature. Commonly, flowing concrete has a slump of 200 mm (8 in.) or

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a flow of 510 to 620 mm or a compacting factor of 0.96 to 0.98. In the process of mix selection, it is convenient first to do so for concrete with a slump of 75 mm (3 in.), the higher slump being then achieved by the addition of a superplasticizer. Properly proportioned, flowing concrete exhibits little bleeding or segregation and no abnormal segregation. To ensure these properties, highly angular, flaky or elongated coarse aggregate should be avoided. As far as fine aggregate is concerned, increasing its content by 5 percentage points above the usual content (with a corresponding reduction in coarse aggregate) contributes to cohesion. When the fine aggregate is very coarse, an even greater increase in its content may be required. The reduction in water content has to be taken into account in the calculation of the yield.

An alternative approach[14.6] to ensuring cohesion of flowing concrete is to select the fine aggregate content so that the total mass of particles smaller than 300 μm in the aggregate, together

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with the mass of cementitious material, exceeds 450 kg per cubic metre of concrete when the maximum size of aggregate is 20 mm; for aggregate whose maximum size is 40 mm, the 'ultra-fines' content should be 400 kg/m^3. Instead of prescribing the content of 'ultra-fines' in relation to the cement content of the mix, their content can be prescribed as a function of the maximum aggregate size. The Italian Standard for ready-mixed concrete UNI 7163-1979 is cited[14.34] as specifying 450 kg (per cubic metre of concrete) of all material smaller than 250 μm when the maximum size of aggregate is 15 mm; for 20 mm aggregate, the value is 430 kg/m^3.

It can be recalled that flowing concrete is well suited to pumping because it offers less resistance than normal slump concrete so that the rate of pumping can be increased and pumping over greater distances is possible. Flowing concrete is desirable for use in large pours because, using a superplasticizer, a low cement content can be combined with a low water content so that both

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the heat development and shrinkage can be kept low. A retarding variety of superplasticizer (Type G according to ASTM C 494-10) can be beneficial.

Mix selection for high performance concrete

In Table 13.5 details of several high performance concrete mixes were given. However, a generalized systematic approach to the selection of mix proportions of high performance concretes has not yet been developed. The reasons for this include the fact that very few structures made of high performance concrete have, as yet, been constructed, and each structure involved specific and specially selected materials. For a future use of high performance concrete, the discussion, in Chapter 13, of the cement–superplasticizer compatibility and of the influence of the various cementitious materials, especially silica fume, on the properties of the resultant concrete should be of considerable value.

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Despite the absence of an accepted method of mix selection for high performance concrete, some specific comments can be made. First, because workability can be controlled by an appropriate dosage of superplasticizer, the water content should be chosen on the basis of the water/cement ratio required from strength considerations. Excessive content of cementitious material should be avoided so as to control shrinkage: a value of 500 to 550 kg/m^3 (850 to 930 lb/yd^3), of which about 10 per cent is silica fume, is a desirable maximum. Portland cement with a high fineness is preferred. The absolute need for compatibility between Portland cement and the superplasticizer has already been emphasized. If air entrainment is to be used, the mix proportions have to be modified by trial and error.[14.15]

Some help in mix selection for high performance concrete can be obtained from ACI 211.4R-93,[14.16] which is intended to apply to concretes with compressive strengths (measured on cylinders) between 40 and 80 MPa (or 6000 to 12

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000 psi). In this book, even the latter value is considered to be below what is deemed to be high performance concrete. Nevertheless, some points are worth noting.

First, with high performance concrete, the specified strength is sometimes required at ages well beyond 28 days; this should be clearly taken into account in considering the strength criterion. Second, in some cases, the particular requirement of high performance concrete is a high modulus of elasticity. To achieve this, the use of coarse aggregate with a high modulus of elasticity is essential, but it is also important to select the cementitious materials which lead to a particularly good bond between the coarse aggregate particles and the matrix.

With respect to the content of coarse aggregate, ACI 211.4R-93[14.16] recommends that the bulk volume of oven-dry rodded coarse aggregate per unit volume of concrete should be between 0.65 when the maximum aggregate size is 10 mm

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($\frac{3}{8}$ in.) and 0.68 when the maximum aggregate size is 12 mm ($\frac{1}{2}$ in.) (cf. Table 14.6). It seems that, unlike ordinary concrete, the value of the bulk volume of coarse aggregate is unaffected by the fineness modulus of fine aggregate, at least in the range 2.5 to 3.2.

While the broad guidance of ACI 211.4R-93[14.16] is useful, it has to be repeated that an experimental approach to the mix selection to high performance concrete is unavoidable.

Mix selection for lightweight aggregate concrete

The relation between compressive strength and water/cement ratio applies to concrete made with lightweight aggregate in the same way as to normal aggregate concrete, and it is possible to follow the usual procedure of mix selection when lightweight aggregate is employed. It is very difficult, however, to determine how much of the total water in the mix is absorbed by the aggreg-

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ate and how much actually occupies space within the concrete, that is, forms part of the cement paste. This difficulty is caused, not only by the very high value of the water absorption of lightweight aggregates, but also by the fact that the absorption varies widely in rate and, with some aggregates, may continue at an appreciable rate for several days. A reliable determination of the specific gravity on a saturated and surface-dry basis is therefore difficult. This topic is discussed more fully in Chapter 13.

Thus the free water/cement ratio depends on the rate of absorption at the time of mixing, and not only on the moisture content of the aggregate. Hence, the use of the water/cement ratio in the calculation of mix proportions is rather difficult. For this reason, proportioning on the basis of the cement content is prefereable, although, in the case of rounded lightweight aggregate with a coated or sealed surface and a relatively low absorption, the use of the standard method of mix selection is practicable.

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Manufactured lightweight aggregate is usually bone-dry, and is rather prone to segregation. If the aggregate is saturated before mixing, the strength of the resulting concrete is about 5 to 10 per cent lower than when dry aggregate is used, for the same cement content and workability. This is due to the fact that, in the latter case, some of the mixing water is absorbed prior to setting, this water having contributed to the workability at the time of placing; this behaviour is somewhat similar to that of vacuum-dewatered concrete. Furthermore, the density of concrete made with a saturated aggregate is higher, and the resistance of such concrete to freezing and thawing is impaired. On the other hand, when aggregate with a high absorption is used, it is difficult to obtain a sufficiently workable and yet cohesive mix, and generally aggregates with absorption of over 10 per cent should be pre-soaked.

It is interesting to note that initially damp lightweight aggregate usually contains more total absorbed water after a short immersion in water

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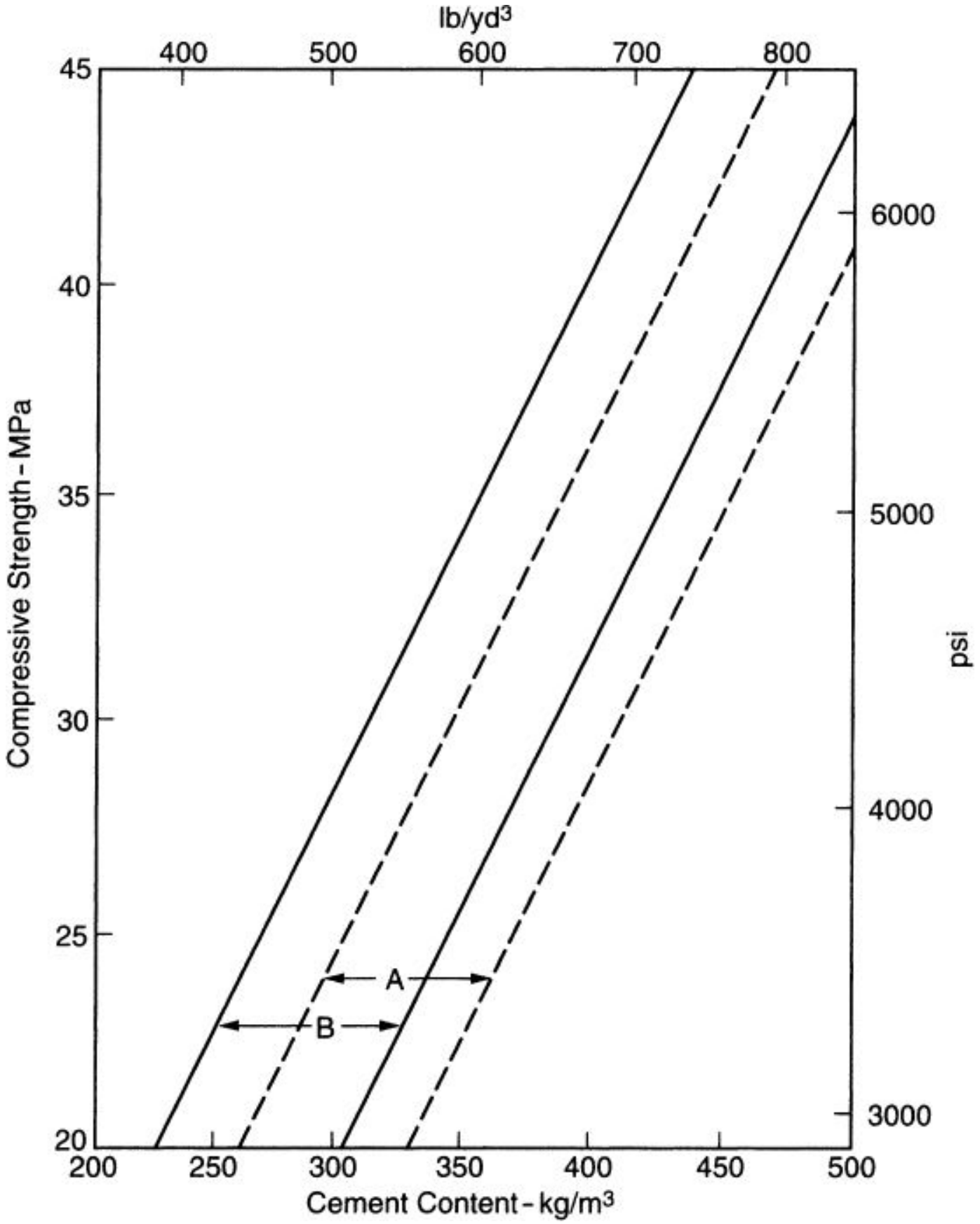
than initially dry aggregate immersed for the same length of time. The reason for this is probably that a small amount of water just moistening an aggregate particle does not remain in the surface pores but diffuses inward and fills the small pores inside. According to Hanson[14.33] this clears the larger surface pores of water so that, upon immersion, these are open to ingress of water almost as large as when the aggregate contains no initially absorbed water.

The preceding discussion explains why mix selection for lightweight aggregate concrete is best based on the premise that, for a given aggregate, together with a given air content, and slump, the compressive strength is directly related to the cement content of the mix. However, this relation can vary widely for lightweight aggregates from different sources. Figure 14.11 shows examples of this relation for an all-lightweight aggregate concrete and also for a lightweight aggregate concrete with normal weight fine aggregate. The practical approach is greatly

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facilitated by the fact that, because lightweight aggregate is a manufactured product with properties which vary very little, the recommendations of the aggregate manufacturer with respect to mix proportioning make a good starting point in the selection of mix proportions for a particular purpose.

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Fig. 14.11. General relation between compressive strength (measured on standard cylinders) and cement content for concrete made with: (*A*) all lightweight aggregate; (*B*) normal weight fine aggregate and lightweight coarse aggregate (based on ref. 14.19)

In the absence of appropriate recommendations or of closely relevant experience, use can be made of ACI Standard Practice 211.2-91.[14.19] The preferred method of ACI 211.2-91 is the so-called volumetric method, which can be used both with all-lightweight aggregate concrete and with lightweight aggregate concrete containing normal weight fine aggregate. In this method, the conversion to mass is based on the damp, loose volume of the aggregates. The total volume of the aggregate is the sum of the volumes of the separate size fractions: the total loose volume of aggregate, relative to the volume of concrete, is usually between 1.05 and 1.25. Of this total volume of aggregate, the loose volume of fine aggregate represents between 40 and 60 per cent, de-

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pending on the specific properties of the aggregate used and on the desired properties of concrete. When the maximum size of aggregate is 20 mm ($\frac{3}{4}$ in.), it is convenient to make the first trial mix using equal volumes of fine and coarse aggregate, and using the cement content corresponding to the desired strength. The water content used is that which gives the required workability. Because of the uncertainties involved, it is usual to make three trial mixes, each with a somewhat different cement content, but all of the required workability. Hence, a relation between cement content and strength, for the given workability, can be obtained over a narrow range.

Example

Data similar to those of ACI 211.2-91 will be used. A lightweight aggregate concrete containing normal weight fine aggregate is required to have a compressive strength (measured on standard cylinders) of 30 MPa and a maximum air-dry density of 1700 kg/m^3. Compliance with the

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density requirement is determined using ASTM C 567-05a. The required slump is 100 mm. The damp, loose density of the coarse and the fine lightweight aggregates is 750 and 880 kg/m^3, respectively. The normal weight fine aggregate has a density in a saturated and surface-dry condition of 1630 kg/m^3.

From past experience, for instance as shown in Fig. 14.11, the required cement content for the trial mix can be taken as 350 kg/m^3. The volumes of aggregate to be used, in cubic metres per cubic metre of concrete, also chosen on the basis of experience, are: 0.60, 0.19, and 0.34, respectively, for the lightweight coarse, lightweight fine, and normal weight fine aggregate. Hence, the required quantities for the first trial batch of 1 m^3 are as follows:

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Cement			350 kg
Lightweight coarse aggregate	= 0.63 ×	750 =	473 kg
Lightweight fine aggregate	= 0.19 ×	880 =	168 kg
Normal weight fine aggregate	= 0.34 ×	1630 =	550 kg
Water found to be required for the desired slump		=	180 kg
Total mass			1676 kg/m^3

The actual density of fresh concrete, which contains some entrapped air, is now determined using the method of ASTM C 138-09. Supposing the actual density has been found to be 1660 kg/m^3, the *yield* is 1676/1660 = 1.01. This means that 1 per cent excess concrete would be produced if the quantities above were used. To remedy this, all the quantities per cubic metre should be divided by 1.01; for example, the cement content becomes 350/1.01 = 346 kg/m^3.

The density of concrete of 1660 kg/m^3 is below the specified maximum and is close enough to it, but tests to determine the actual strength are necessary.

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When adjustments to the mix proportions are necessary, some of the rule-of-thumb values given in ACI 211.2-91[14.19] can be useful. For example, if the mass of fine aggregate, expressed as a percentage of the total mass of aggregate, is increased by 1 percentage point, the water content necessary to maintain a constant slump should be increased by 2 kg/m^3. In order to maintain a constant strength, the cement content should be increased by about 1 per cent. The mass of coarse aggregate needs to be reduced so as to maintain the yield.

As another example of the 'rules' of ACI 211.2-91, if an increase in slump of 25 mm is required, the water content should be increased by 6 kg/m^3. In order to maintain a constant strength, a concomitant increase in cement content of 3 per cent is necessary. The mass of fine aggregate needs to be reduced to maintain yield.

Advice on the selection of mix proportions for moderate-strength lightweight aggregate con-

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crete is given in ACI 523.3R-93.[14.20] The same guide gives advice on mix selection for cellular concrete.

It is worth repeating that the various data on the mix proportions of the lightweight aggregate concrete are no more than typical figures, different aggregates having varying values of density and water demand. On the other hand, lightweight aggregate from a single source possesses a high uniformity. For this reason, mix selection over a small range of desired properties can be made with considerable confidence.

British method of mix selection (mix design)

The current British method is that of the Department of the Environment revised in 1997.[14.11] Similarly to the ACI approach, the British method explicitly recognizes the durability requirements in the mix selection. The method is applicable to normal weight concrete made with Portland cement only or also incorporating ground

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granulated blastfurnace slag or fly ash, but it does not cover flowing concrete or pumped concrete; nor does it deal with lightweight aggregate concrete. Three maximum sizes of aggregate are recognized: 40, 20, and 10 mm.

In essence, the British method consists of 5 steps, as follows.

Step 1. This deals with compressive strength for the purpose of determining the water/cement ratio. The concept of *target mean strength* is introduced, this being equal to the specified characteristic strength plus a margin to allow for variability. The target mean strength is thus similar in concept to the mean compressive strength of ACI 318R-02[14.8] (see p. 733).

The relation between strength of concrete and the water/cement ratio is dealt with rather ingeniously. Certain strengths are assumed at a water/cement ratio of 0.5 for different cements and

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types of aggregate (Table 14.9). The latter factor recognizes the significant influence of aggregate on strength. The data of Table 14.9 apply to a hypothetical concrete of medium richness cured in water at 20 °C (68 °F); richer mixes would have a relatively higher early strength because they gain strength more rapidly.

Table 14.9. Approximate Compressive Strengths of Concretes Made with a Free Water/*Cement Ratio of 0.5 According to the 1997 British Method*[14.11]

Type of cement	*Type of coarse aggregate*	*Compressive strength* (MPa (psi)) at the age of (days):*			
		3	*7*	*28*	*91*
Ordinary Portland (Type I)	Uncrushed	22 (3200)	30 (4400)	42 (6100)	49 (7100)
Sulfate-resisting Portland (Type V)	Crushed	27 (3900)	36 (5200)	49 (7100)	56 (8100)
Rapid-hardening Portland (Type III)	Uncrushed	29 (4200)	37 (5400)	48 (7000)	54 (7800)
	Crushed	34 (4900)	43 (6200)	55 (8000)	61 (8900)

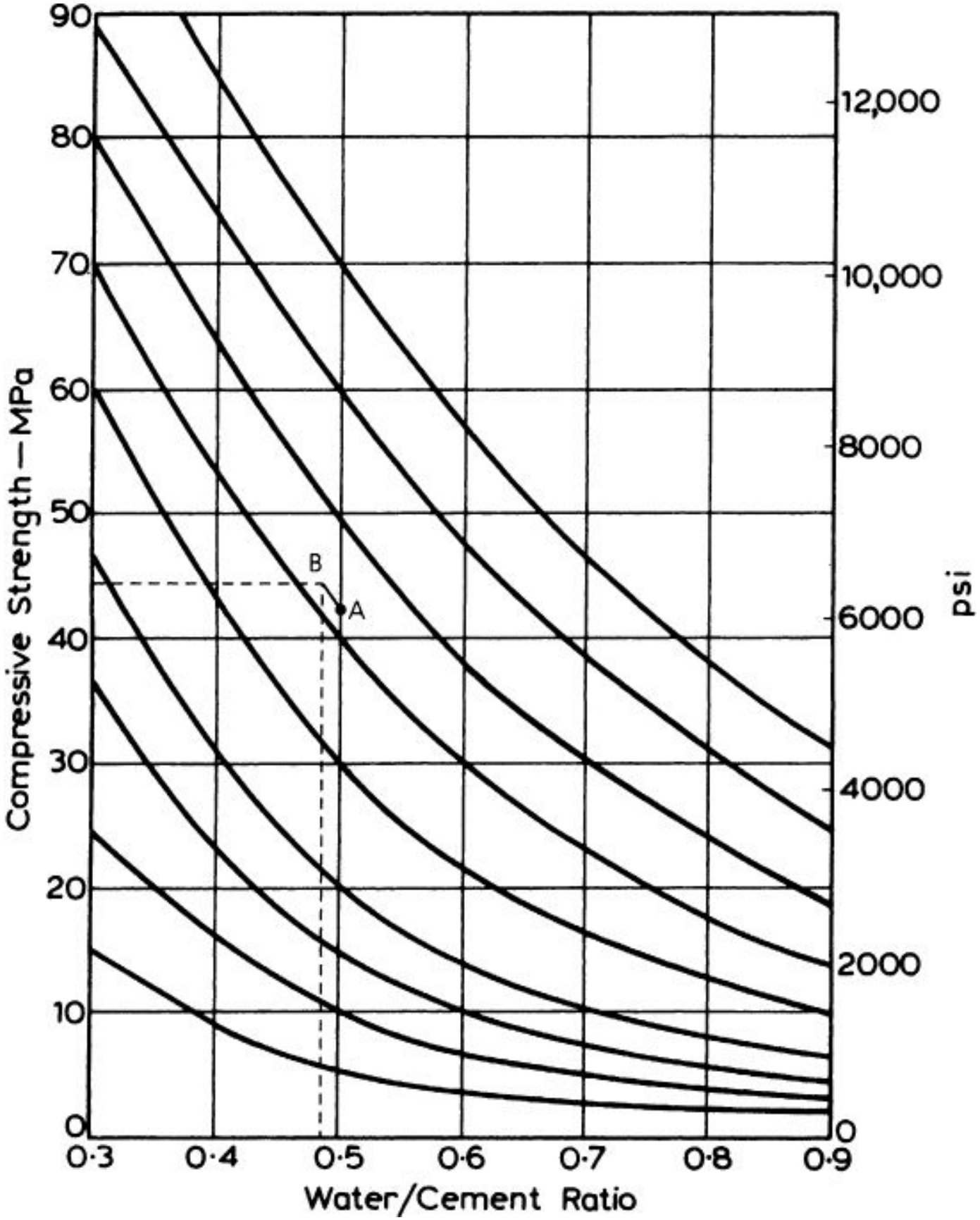
*Measured on cubes.

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From Table 14.9, we find the appropriate value of strength (at a water/cement ratio of 0.5) corresponding to the type of cement, type of aggregate, and age which are to be used. Turning to Fig. 14.12, we mark a point corresponding to this strength at a water/cement ratio of 0.5. Through this point, we now draw a curve 'parallel' (or, strictly speaking, affine) to the neighbouring curves. Using this new curve, we read off (as abscissa) the water/cement ratio corresponding to the specified target mean strength (as the ordinate). A possible need for a lower water/cement ratio for reasons of durability must not be forgotten.

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Fig. 14.12. Relation between compressive strength and free water/cement ratio for use in the British mix selection method[14.11] (see Table 14.9) (Crown copyright)

Step 2. This deals with the determination of the water content for the required workability, expressed either as slump or as Vebe time, recognizing the influence of the maximum size of aggregate and its type, namely crushed or uncrushed. The relevant data are given in Table 14.10. It can be noted that the compacting factor is not used in mix selection, although it can be used for control purposes.

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Table 14.10. Approximate Free Water Contents Required to Give Various Levels of Workability According to the 1997 British Method[14.11] (Crown copyright)

Aggregate		*Water content, kg/m³ (lb/yd³) for:*				
Max size mm (in.)	*Type*	*Slump, mm (in.)*	*0–10 (0–½)*	*10–30 (½–1)*	*30–60 (1–2½)*	*60–180 (2½–7)*
		Vebe time, s	*>12*	*6–12*	*3–6*	*0–3*
10 (⅜)	Uncrushed		150 (255)	180 (305)	205 (345)	225 (380)
	Crushed		180 (305)	205 (345)	230 (390)	250 (420)
20 (¾)	Uncrushed		135 (230)	160 (270)	180 (305)	195 (330)
	Crushed		170 (285)	190 (320)	210 (355)	225 (380)
40 (1½)	Uncrushed		115 (195)	140 (235)	160 (270)	175 (295)
	Crushed		155 (260)	175 (295)	190 (320)	205 (345)

Step 3. This determines the cement content, which is simply the water content divided by the water/cement ratio. This cement content must not conflict with any minimum value specified for reasons of durability or a maximum value specified for reasons of heat development.

Step 4. This deals with the determination of the total aggregate content. This re-

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quires an estimate of the fresh density of fully compacted concrete, which can be read off Fig. 14.13 for the appropriate water content (from Step 2) and specific gravity of the aggregate. If this is unknown, the value of 2.6 for uncrushed aggregate and 2.7 for crushed aggregate can be assumed. The aggregate content is obtained by subtracting from the fresh density the value of the cement content and of the water content.

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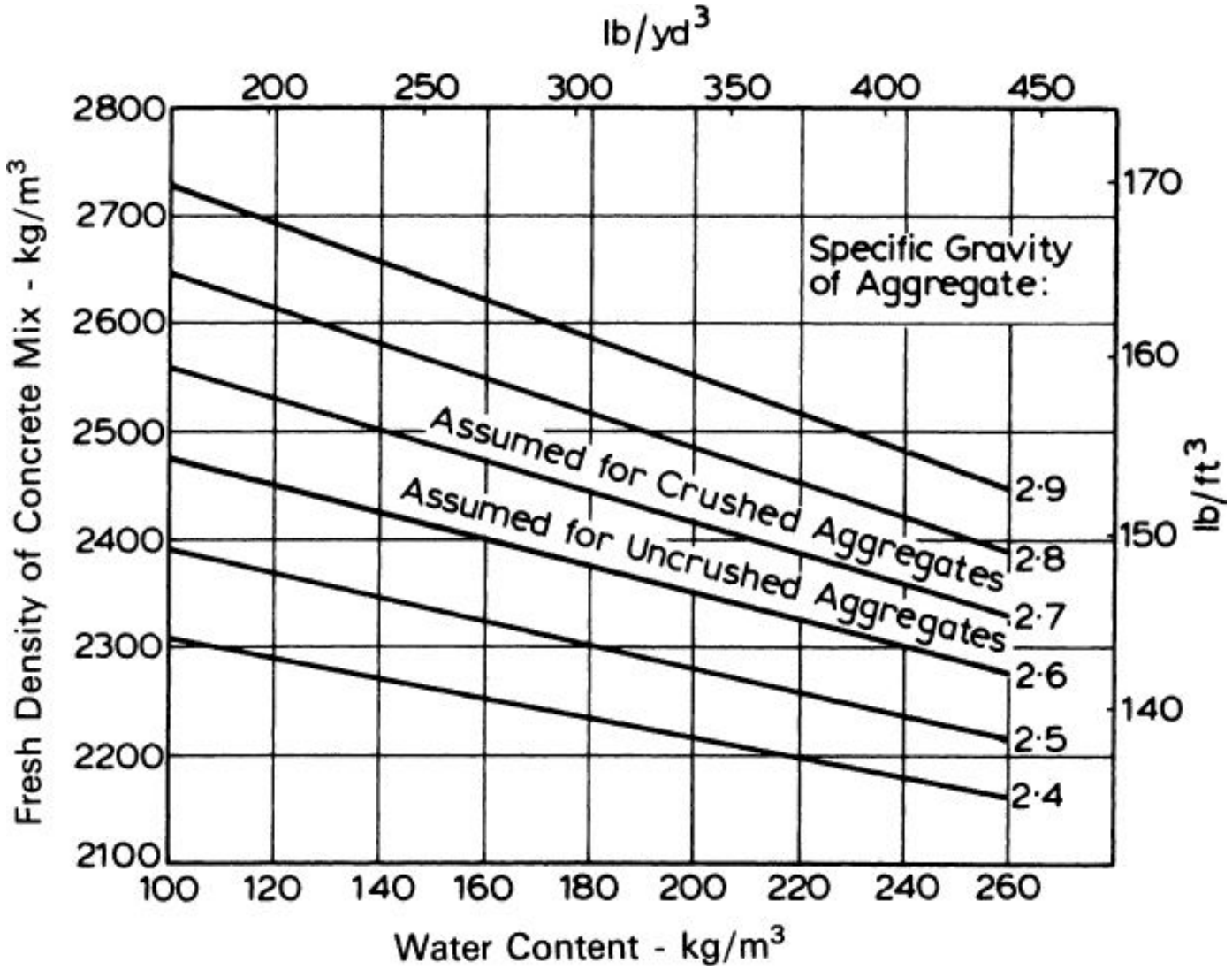


Fig. 14.13. Estimated wet density for fully compacted concrete[14.11] (specific gravity is given for saturated and surface-dry aggregate) (Crown copyright)

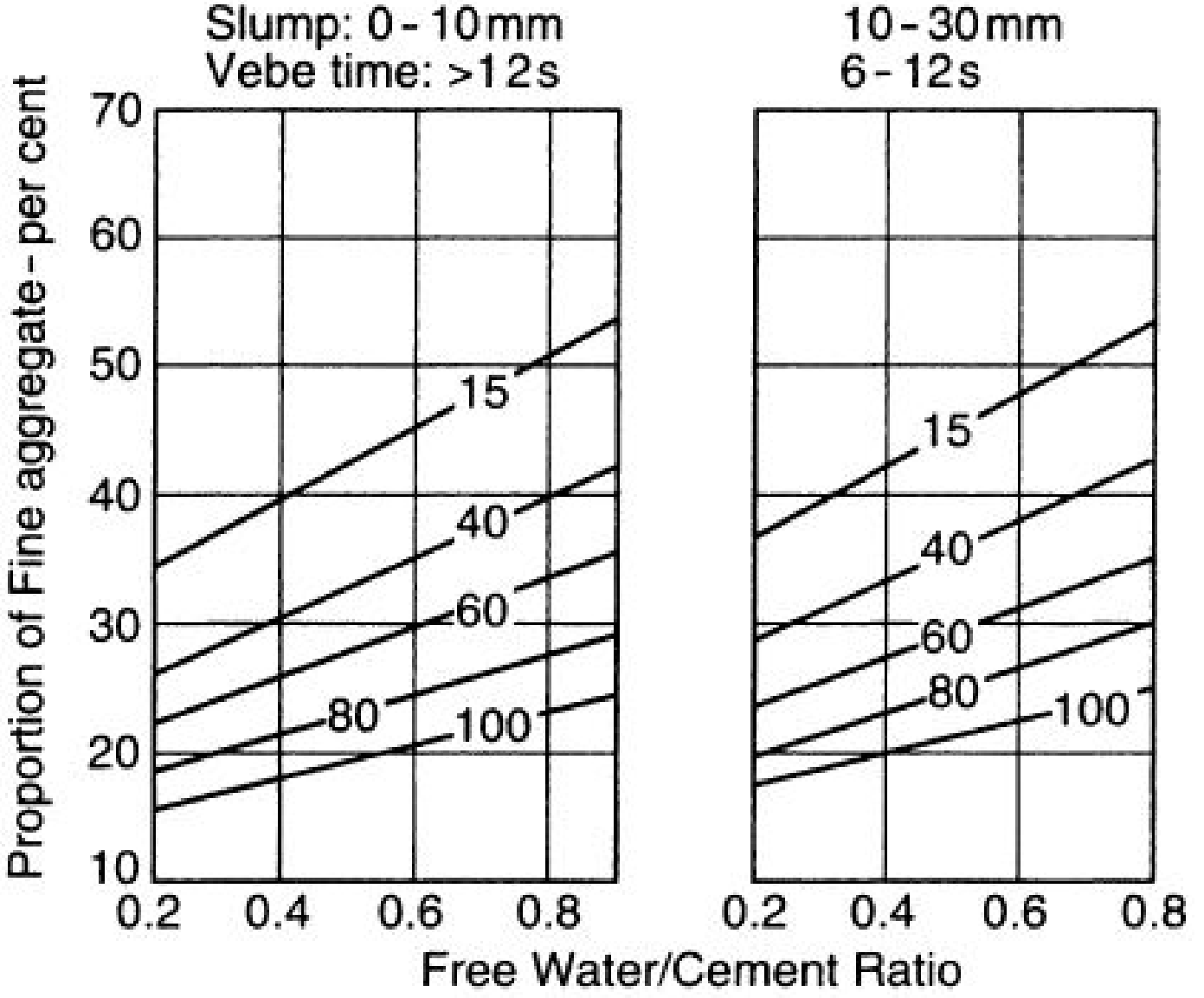
Step 5. This determines the proportion of fine aggregate in the total aggregate, using the recommended values of Fig. 14.14; only data for 20 and

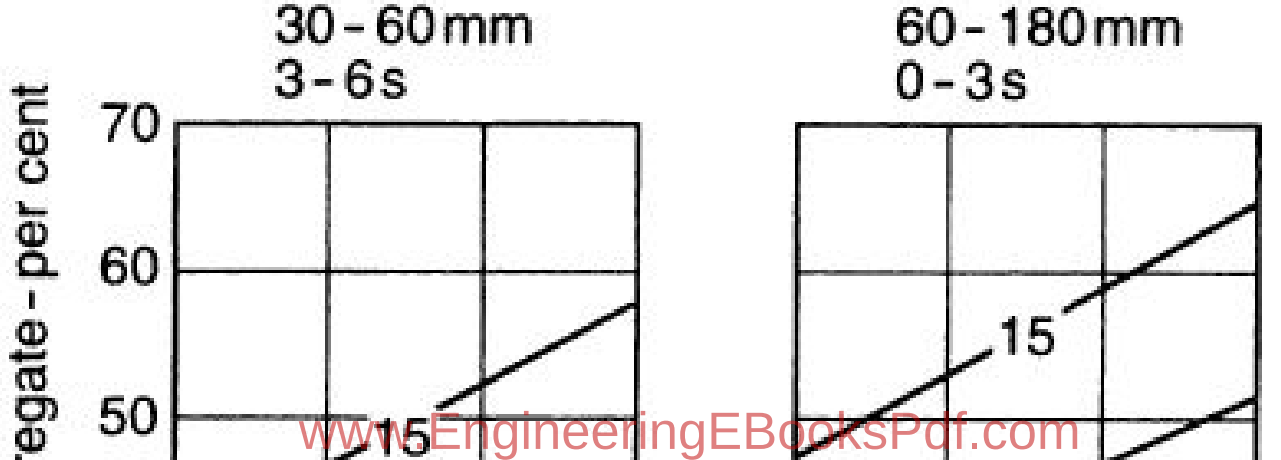
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40 mm aggregates are shown. The governing factors are: the maximum size of aggregate, the level of workability, the water/cement ratio, and the percentage of fine aggregate passing the 600 μm sieve. Other aspects of the grading of the fine aggregate are ignored and so is the grading of the coarse aggregate. Once the proportion of fine aggregate has been obtained, multiplying it by the total aggregate content gives the content of fine aggregate.

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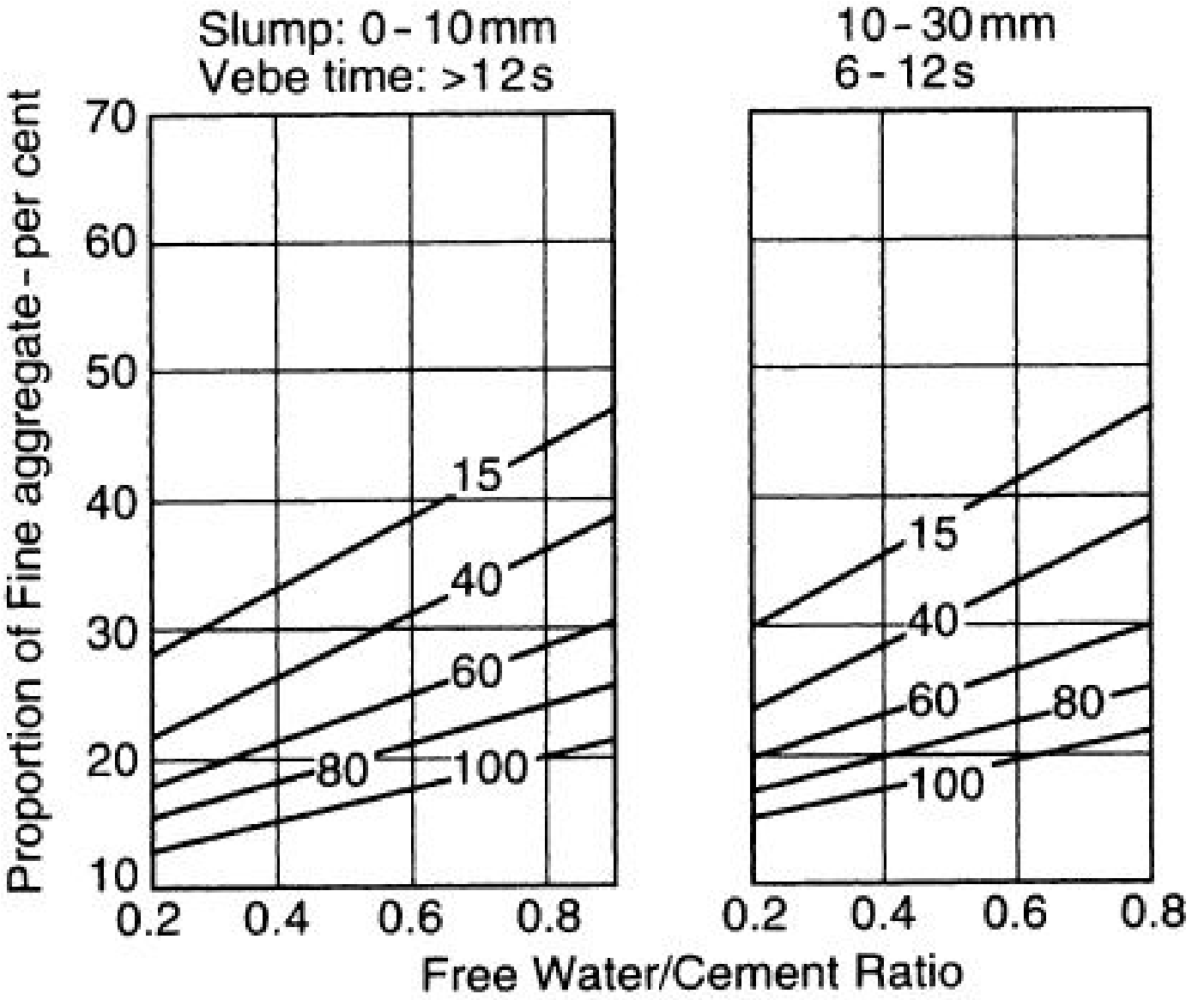
Maximum aggregate size: 20 mm

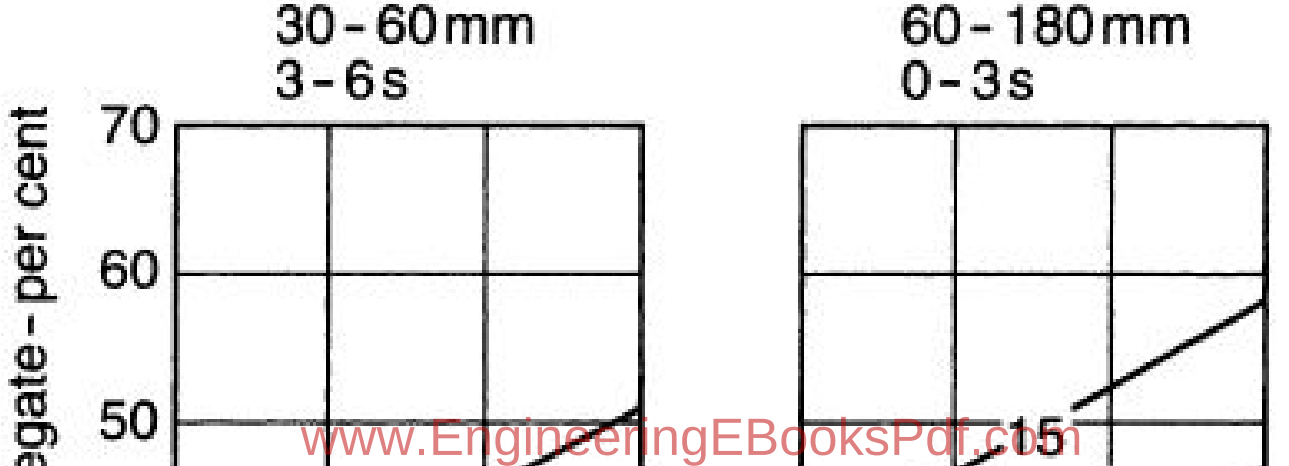




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Maximum aggregate size: 40 mm





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Fig. 14.14. Recommended proportion of fine aggregate (expressed as percentage of total aggregate) as a function of free water/cement ratio for various workabilities and maximum sizes[14.11] (numbers refer to percentage of fine aggregate passing 600 μm sieve) (Building Research Establishment; Crown copyright)

The content of coarse aggregate is then the difference between the total aggregate content and the content of fine aggregate. The coarse aggregate, in turn, should be divided into size fractions depending on the aggregate shape. As a general guide, the percentages of Table 14.11 can be used.

Table 14.11. Proportion of Coarse Aggregate Fractions According to the 1997 British Method[14.11]

Total coarse aggregate	*5–10 mm*	*10–20 mm*	*20–40 mm*
100	33	67	—
100	18	27	55

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Following the above calculations, trial mixes must be made. It should also be remembered that the British method is based on the experience of British materials so that the various values given in the tables and figures may not be applicable in other parts of the world.

The selection of mix proportions to achieve a desired splitting tension strength, which was previously included in the British method, is no longer recommended. More generally, in British practice, although flexural strength may be the correct design criterion for some structures, e.g. highway pavements, the selection of mix proportions on the basis of a direct determination of the flexural strength is rarely practised. The reason for this lies in the difficulty of the use of the modulus of rupture as a control test (see p. 599). Thus, mix proportions are selected in the usual manner, and both the compressive and tensile strengths are determined. Provided the latter is adequate, control and the attendant mix adjustments are based on the compressive strength.

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The British method of mix selection may become modified when the European Standard BS EN 206-1 : 2000 becomes widely used. Furthermore, the European Standards have not yet 'shaken down' and are likely to be amended.

Example

We wish to select a mix to satisfy requirements similar to those used in the example of the American method of mix selection (p. 757). These are: a *mean* 28-day compressive strength (measured on standard cubes) of 44 MPa (which is equivalent to a cylinder strength of 35 MPa); a slump of 50 mm; uncrushed aggregate with a maximum size of 20 mm; specific gravity of aggregate of 2.64; 60 per cent of fine aggregate passes the 600 μm sieve; no air entrainment required; ordinary Portland cement to be used.

Step 1. From Table 14.9, for ordinary Portland cement and uncrushed aggregate, we find the 28-day strength to be 42 MPa. We enter this value on

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the ordinate corresponding to a water/cement ratio of 0.5 in Fig. 14.12; this point is marked *A*. Through *A*, we draw a line 'parallel' to the nearest curve until it intersects the ordinate corresponding to the specified strength of 44 MPa; this is point *B*. The ordinate through this point gives the water/cement ratio of 0.48.

Step 2. From Table 14.10, for 20 mm uncrushed aggregate and a slump of 50 mm, we find the water requirement to be 180 kg/m^3.

Step 3. The cement content is 180/0.48 = 375 kg/m^3.

Step 4. From Fig. 14.13, for a water content of 180 kg/m^3 and aggregate with a specific gravity of 2.64, we read off the fresh density of con-

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crete of 2400 kg/m^3. The total aggregate content is thus:

$$2400 - 375 - 180 = 1845 \text{ kg/m}^3.$$

Step 5. In Fig. 14.14, we find the particular diagram for the maximum size of aggregate of 20 mm and a slump encompassing the value of 50 mm. On the line representing fine aggregate with 60 per cent passing the 600 μm sieve, at a water/cement ratio of 0.48, the proportion of fine aggregate is 32 per cent (by mass of total aggregate). Hence, the fine aggregate content is:

$$32 \times 1845 = 590 \text{ kg/m}^3$$

and the coarse aggregate content is $1845 - 590 = 1255$ kg/m^3.

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Other methods of mix selection

It is not suggested that, on each occasion, the selection of mix proportions should necessarily follow any of the procedures described earlier. Indeed, various people have their own methods which work well. What these 'methods' have in common is that they use shortcuts or rule-of-thumb steps in the procedure which are based on an individual's experience. As long as these 'methods' are used by the same individual and as long as the materials involved are not fundamentally different from those used in the past, all is well. If, however, a person has to select mix proportions using unfamiliar materials, the procedures described in this chapter are very helpful. But, even so, selecting mix proportions is not just a rule-based process.

Over the years, numerous attempts have been made to develop mix-proportioning equations based on observed influences of various factors. Such relations, or models, inevitably represent averages of behaviour. And yet, in every partic-

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ular case, the behaviour of concrete is affected by properties of ingredients which cannot be, or cannot yet be, expressed mathematically. These properties include aggregate shape and texture which, at present, are described only in broad terms such as ‘angular shape’ or ‘smooth texture’. Likewise, grading of aggregate is measured only at several sieve apertures, between any two of which there may be variation in the actual size of the particles. There is little prospect of a proper quantification of these properties in the near future. The possibility of determining these properties of aggregate during batching, so that the quantity of water added can be instantly adjusted, is even more remote.

Many properties of cement are also not properly included in the various models because the actual properties of the cement used in a given mix (as distinct from average properties) are not known or not determined.

These average relations may be valid ‘on average’, but trying to use them with a particular

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set of materials must perforce be subject to large errors. It is, therefore, futile to use elegant computer-based calculations of mix proportions. This is not to say that such an approach may not be feasible in the future when it becomes possible to describe mathematically the properties of all the materials to be used, and also to control, or measure, these properties at the batcher.[14.35]

One other note of caution may be appropriate. A statistically derived model can, at best, be valid within the range of variables used in deriving it. If this range is not clearly stated, unwitting extrapolation may be strongly misleading. It is also worth adding that some of the more elaborate methods involve numerous interacting terms, but there is little value in including factors which are subject to unpredictable variation during construction. Thus, the promise of a 'hands-off' computer-controlled *definitive* selection of mix proportions is unrealistic. In the meantime, the selection of mix proportions must be based on preliminary calculations of the kind described in

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this chapter, followed by trial mixes. The selection of mix proportions is an art as much as a science.

Concluding remarks

The various methods of mix selection may seem simple and, indeed, they do not involve any complex calculations. However, a successful implementation of the selection requires *experience*, coupled with the *knowledge* of the influence of various factors upon the properties of concrete; this knowledge must be based on an *understanding* of the behaviour of concrete. When these three desiderata – experience, knowledge, and understanding – are *all* present, the first trial mix is likely to be approximately satisfactory, and can be rapidly and successfully adjusted so as to achieve a mix with the desired properties.

It is not enough to select a suitable concrete mix; it is also necessary to ensure a proper execution of all the operations involved in concreting. Such execution requires skill backed by ap-

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propriate knowledge at the execution level. The belief, once held, that any fool can make concrete has, alas, sometimes led to a situation where he did. The consequences of such execution manifest themselves before long. It cannot be stated too strongly that, competently used, concrete is a very successful construction material but, in the literal sense of the word, concrete is not foolproof.

The first edition, and the subsequent two editions of this book, ended with a 'tongue-in-cheek' note saying, "If the reader is unable to design a satisfactory mix he should seriously consider the alternative of construction in steel." The situation has changed. First, the reader may as well be a she as a he. Then, for many modern structures, steel is not a simple alternative and may not be appropriate. And, lastly, in this fifth and truly final edition well into the third millennium, perhaps such a ubiquitous and weighty material as concrete should not be treated too flippantly. The aim of this book has been to try to provide an

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understanding of the behaviour of concrete – an excellent construction material for many years to come. If this aim has been achieved, the reader will not, in despair and frustration, need to "seriously consider the alternative of construction in steel".

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Appendix I. Relevant ASTM Standards

The two digits after the dash denote the year of publication: *a* denotes a revision in the year of publication; a date in brackets indicates the year in which the standard was last reapproved without change.

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C 29-09	Test Method for Bulk Density ('Unit Weight') and Voids in Aggregate
C 31-09	Practice for Making and Curing Concrete Test Specimens in the Field
C 33-08	Specification for Concrete Aggregates
C 39-09a	Test Method for Compressive Strength of Cylindrical Concrete Specimens
C 40-04	Test Method for Organic Impurities in Fine Aggregates for Concrete
C 42-04	Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 70-06	Test Method for Surface Moisture in Fine Aggregate
C 78-09	Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
C 87-05	Test Method for Effect of Organic Impurities in Fine Aggregate on Strength of Mortar
C 88-05	Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate
C 91-05	Specification for Masonry Cement
C 94-09a	Specification for Ready-Mixed Concrete
C 109-08	Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [5O-mm] Cube Specimens)
C 117-04	Test Method for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing
C 123-04	Test Method for Lightweight Particles in Aggregate
C 125-09a	Terminology Relating to Concrete and Concrete Aggregates
C 138-09	Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete
C 143-10	Test Method for Slump of Hydraulic-Cement Concrete
C 150-09	Specification for Portland Cement

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C 151-09	Test Method for Autoclave Expansion of Hydraulic Cement
C 156-09a	Test Method for Water Loss [from a Mortar Specimen] Through Liquid Membrane-Forming Curing Compounds for Concrete
C 171-07	Specification for Sheet Materials for Curing Concrete
C 173-10	Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
C 186-05	Test Method for Heat of Hydration of Hydraulic Cement
C 191-08	Test Methods for Time of Setting of Hydraulic Cement by Vicat Needle
C 192-07	Practice for Making and Curing Concrete Test Specimens in the Laboratory
C 204-07	Test Methods for Fineness of Hydraulic Cement by Air-Permeability Apparatus
C 215-08	Test Method for Fundamental Transverse, Longitudinal, and Torsional Resonant Frequencies of Concrete Specimens
C 227-10	Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar- Bar Method)
C 230-08	Specification for Flow Table for Use in Tests of Hydraulic Cement
C 231-09b	Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method
C 232-09	Test Methods for Bleeding of Concrete
C 260-06	Specification for Air-Entraining Admixtures for Concrete
C 266-08	Test Method for Time of Setting of Hydraulic Cement Paste by Gillmore Needles
C 289-07	Test Method for Potential Alkali-Silica Reactivity of Aggregates (Chemical Method)
C 293-08	Test Method for Flexural Strength of Concrete (Using Simple Beam With Center-Point Loading)
C 294-05	Descriptive Nomenclature for Constituents of Concrete Aggregates
C 295-08	Guide for Petrographic Examination of Aggregates for Concrete

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C 309-07	Specification for Liquid Membrane-Forming Compounds for Curing Concrete
C 311-07	Test Methods for Sampling and Testing Fly Ash or Natural Pozzolans for Use in Portland-Cement Concrete
C 330-09	Specification for Lightweight Aggregates for Structural Concrete
C 331-05	Specification for Lightweight Aggregates for Concrete Masonry Units
C 332-09	Specification for Lightweight Aggregates for Insulating Concrete
C 403-08	Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance
C 418-05	Test Method for Abrasion Resistance of Concrete by Sandblasting
C 441-05	Test Method for Effectiveness of Pozzolans or Ground Blast-Furnace Slag in Preventing Excessive Expansion of Concrete Due to the Alkali-Silica Reaction
C 452-06	Test Method for Potential Expansion of Portland-Cement Mortars Exposed to Sulfate
C 457-10a	Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete
C 469-02	Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C 470-09	Specification for Molds for Forming Concrete Test Cylinders Vertically
C 494-10	Specification for Chemical Admixtures for Concrete
C 496-04	Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
C 512-02	Test Method for Creep of Concrete in Compression
C 531-00 (2005)	Test for Linear Shrinkage and Coefficient of Thermal Expansion of Chemical-Resistant Mortars, Grouts, Monolithic Surfacings, and Polymer Concretes
C 566-97 (2004)	Test Method for Total Evaporable Moisture Content of Aggregate by Drying

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C 567-05a	Test Method for Determining Density of Structural Lightweight Concrete
C 586-05	Test Method for Potential Alkali Reactivity of Carbonate Rocks as Concrete Aggregates (Rock-Cylinder Method)
C 595-10	Specification for Blended Hydraulic Cements
C 597-09	Test Method for Pulse Velocity Through Concrete
C 617-09a	Practice for Capping Cylindrical Concrete Specimens
C 618-08a	Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
C 642-06	Test Method for Density, Absorption, and Voids in Hardened Concrete
C 666-03 (2008)	Test Method for Resistance of Concrete to Rapid Freezing and Thawing
C 672-03	Test Method for Scaling Resistance of Concrete Surfaces Exposed to De-icing Chemicals
C 684-99 (2003)	Test Method for Making, Accelerated Curing, and Testing Concrete Compression Test Specimens
C 685-10	Specification for Concrete Made by Volumetric Batching and Continuous Mixing
C 779-05	Test Method for Abrasion Resistance of Horizontal Concrete Surfaces
C 803-03	Test Method for Penetration Resistance of Hardened Concrete
C 805-08	Test Method for Rebound Number of Hardened Concrete
C 845-04	Specification for Expansive Hydraulic Cement
C 856-04	Practice for Petrographic Examination of Hardened Concrete
C 873-04	Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds
C 878-09	Test Method for Restrained Expansion of Shrinkage-Compensating Concrete

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C 900-06	Test Method for Pullout Strength of Hardened Concrete
C 917-05	Test Method for Evaluation of Cement Strength Uniformity From a Single Source
C 918-07	Test Method for Measuring Early-Age Compressive Strength and Projecting Later-Age Strength
C 944-99 (2005)	Test Method for Abrasion Resistance of Concrete or Mortar Surfaces by the Rotating-Cutter Method
C 979-05	Specification for Pigments for Integrally Colored Concrete
C 989-09a	Specification for Slag Cement for Use in Concrete and Mortars
C 1012-09	Test Method for Length Change of Hydraulic-Cement Mortars Exposed to a Sulfate Solution
C 1017-07	Specification for Chemical Admixtures for Use in Producing Flowing Concrete
C 1038-04	Test Method for Expansion of Hydraulic Cement Mortar Bars Stored in Water
C 1074-04	Practice for Estimating Concrete Strength by the Maturity Method
C 1084-10	Test Method for Portland-Cement Content of Hardened Hydraulic-Cement Concrete
C 1105-08a	Test Method for Length Change of Concrete Due to Alkali-Carbonate Rock Reaction
C 1138-05	Test Method for Abrasion Resistance of Concrete (Underwater Method)
C 1152-04	Test Method for Acid-Soluble Chloride in Mortar and Concrete
C 1157-10	Performance Specification for Hydraulic Cement
C 1202-10	Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
C 1240-05	Specification for Silica Fume Used in Cementitious Mixtures
C 1602-06	Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete
C 1712-09	Test Method for Rapid Assessment of Static Segregation Resistance of Self-Consolidating Concrete Using Penetration Test
E 11-2009	Specification for Wire-Cloth Sieves for Testing Purposes

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Appendix II. Relevant British and European Standards

BS denotes an ‘old style’ British Standard prepared in the United Kingdom.

BS EN denotes a European Standard adopted as a British Standard and identical to a national standard in all countries of the European Union as well as Iceland, Norway and Switzerland.

PD is a Published Document that is not a British Standard but a (guidance) document published by the British Standards Institution, which gives guidance on the use of a BS EN or amplifies it.

w/d indicates withdrawn.

A number in brackets is the year when the standard was reconfirmed.

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BS 12: 1996 (w/d)	Portland cement.
BS CP 110-1: 1972	The structural use of concrete.
BS 146-1: 1996 (w/d)	Portland blastfurnace cement.
BS EN 196-1: 2005	Methods of testing cement. Determination of strength.
BS EN 196-2: 2005	Methods of testing cement. Chemical analysis of cement.
BS EN 196-3: 2005	Determination of setting time and soundness.
BS EN 196-5: 2005	Pozzolanicity test for pozzolanic cements.
BS EN 196-6: 2010	Methods of testing cement. Determination of fineness.
BS EN 197-1: 2000	Cement. Composition, specifications and conformity criteria for common cements.
BS EN 197-2: 2000	Cement. Conformity evaluation.
BS EN 197-4: 2004	Cement. Composition, specifications and conformity criteria for low early strength blastfurnace cements.
BS EN 206-1: 2000	Concrete. Specification, performance, production and conformity.
BS EN 206-9: 2010	Concrete. Additional rules for self-compacting concrete (SCC).
BS 410-1: 2000	Test sieves. Technical requirements and testing. Test sieves of metal wire cloth.
BS 410-2: 2000	Test sieves. Technical requirements and testing. Test sieves of perforated metal plate.
BS EN 450-1: 2005	Fly ash for concrete. Definition, specification and conformity criteria.
BS EN 480-1: 2006	Admixtures for concrete, mortar and grout. Test methods. Reference concrete and reference mortar for testing.
BS ISO 565 – 1990	Test sieves – metal wire cloth, perforated metal plate and electroformed sheet – nominal side openings.
BS EN 678: 1994	Determination of the dry density of autoclaved aerated concrete.
BS EN 679: 2009	Determination of the compressive strength of autoclaved aerated concrete.

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BS EN 680: 2005	Determination of the drying shrinkage of autoclaved aerated concrete.
BS 812-1: 1975(w/d)	Testing aggregates. Methods for determination of particle size and shape.
BS 812-2: 1995	Testing aggregates. Methods of determination of density.
BS 812-102: 1989(w/d)	Testing aggregates. Methods for sampling.
BS 812-103.1: 1985 (2000)	Testing aggregates. Method for determination of particle size distribution. Sieve tests.
BS 812-103.2: 1989 (2006)	Testing aggregates. Method of determination of particle size distribution. Sedimentation test.
BS 812-104: 1994 (2000)	Testing aggregates. Method for qualitative and quantitative petrographic examination of coarse aggregate.
BS 812-105.2: 1990	Testing aggregates. Methods for determination of shape. Elongation index of coarse aggregate (obsolescent).
BS 812-106: 1985	Testing aggregates. Method for determination of shell content in coarse aggregate.
BS 812-109: 1990	Testing aggregates. Methods for determination of moisture content.
BS 812-110: 1990	Testing aggregates. Methods for determination of aggregate crushing value (ACV).
BS 812-111: 1990	Testing aggregates. Methods for determination of ten per cent fines value (TFV).
BS 812-112: 1990 (2000)	Testing aggregates. Method for determination of aggregate impact value (AIV).
BS 812-113: 1990	Testing aggregates. Method for determination of aggregate abrasion value (AAV).
BS 812-117: 1988 (2000)	Testing aggregates. Method for determination of water-soluble chloride salts.
BS 812-118: 1988 (2000)	Testing aggregates. Method for determination of sulfate content.

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BS 812-121: 1989 (2000)	Testing aggregates. Method for determination of soundness.
BS 812-123: 1999	Testing aggregates. Method for determination of alkali-silica reactivity. Concrete prism method.
BS 812-124: 2009	Testing aggregates. Method for determination of frost-heave.
BS 882-1992 (w/d)	Specification for aggregates from natural sources for concrete.
BS EN 932-1: 1997	Tests for general properties of aggregates. Methods for sampling.
BS EN 932-2: 1999	Tests for general properties of aggregates. Methods for reducing laboratory samples.
BS EN-932-3: 1997	Tests for general properties of aggregates. Procedure and terminology for simplified petrographic description.
BS EN 932-5: 2000	Tests for general properties of aggregates. Common equipment and calibration.
BS EN 932-6: 1999	Tests for general properties of aggregates. Definitions of repeatability and reproducibility.
BS EN 933-1: 1997	Tests for geometrical properties of aggregates. Determination of particle size distribution. Sieving method.
BS EN 933-2: 1996	Tests for geometrical properties of aggregates. Determination of particle size distribution. Test sieves, nominal size of apertures.
BS EN 933-3: 1997	Tests for geometrical properties of aggregates. Determination of particle shape. Flakiness index.
BS EN 933-4: 2008	Tests for geometrical properties of aggregates. Determination of particle shape. Shape index.
BS EN 933-5: 1998	Tests for geometrical properties of aggregates. Determination of percentage of crushed and broken surfaces in coarse aggregate particles.

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BS EN 933-6: 2001	Tests for geometrical properties of aggregates. Assessment of surface characteristics. Flow coefficient of aggregates.
BS EN 933-7: 1998	Tests for geometrical properties of aggregates. Determination of shell content. Percentage of shells in coarse aggregates.
BS EN 933-8: 1999	Tests for geometrical properties of aggregates. Assessment of fines. Sand equivalent test.
BS EN 933-9: 2009	Tests for geometrical properties of aggregates. Assessment of fines. Methylene blue test.
BS EN 933-10: 2009	Tests for geometrical properties of aggregates. Assessment of fines. Grading of filler aggregates (air jet sieving).
BS EN 933-11: 2009	Tests for geometrical properties of aggregates. Classification test for the constituents of coarse recycled aggregate particle size distribution. Sieving method.
BS EN 934-2: 2009	Admixtures for concrete, mortar and grout. Concrete admixtures. Definitions, requirements, conformity, marking and labelling.
BS EN 934-5: 2007	Admixtures for concrete, mortar and grout. Admixtures for sprayed concrete. Definitions, requirements, conformity, marketing and labelling.
BS EN 934-6: 2001	Admixtures for concrete, mortar and grout. Sampling, conformity control and evaluation of conformity.
BS EN 1008: 2002	Mixing water for concrete. Specification for sampling, testing and assessing the suitability of water, including water recovered from processes in the concrete industry, as mixing water for concrete.
BS 1305: 1974	Specification for batch type concrete mixers.
BS 1370: 1979	Specification for low heat Portland cement.
BS EN 1744-1: 2009	Tests for chemical properties of aggregates. Chemical analysis.

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BS 1881-5: 1984 (w/d)	Testing concrete. Methods of testing hardened concrete for other than strength.
BS 1881-103: 1993	Testing concrete. Method of determination of compacting concrete.
BS 1881-105: 1984 (w/d)	Testing concrete. Method for determination of flow.
BS 1881-108: 1993 (w/d)	Testing concrete. Method for making test cubes from fresh concrete.
BS 1881-110: 1983 (w/d)	Testing concrete. Method for making test cylinders from fresh concrete.
BS 1881-112: 1983	Testing concrete. Methods of accelerated curing of test cubes.
BS 1881-113: 1983	Testing of concrete. Method for making and curing no-fines test cubes.
BS 1881-115: 1986 (w/d)	Testing concrete. Specification for compression testing machines for concrete.
BS 1881-119: 1983	Testing concrete. Method for determination of compressive strength using portions of beam broken in flexure (equivalent cube method).
BS 1881-120: 1983 (w/d)	Testing concrete. Method for determination of the compressive strength of concrete cores.
BS 1881-121: 1983	Testing concrete. Method for determination of static modulus of elasticity in compression.
BS 1881-122: 1983	Testing concrete. Method for determination of water absorption.
BS 1881-124: 1988	Testing concrete. Methods of analysis of hardened concrete.
BS 1881-131: 1998	Testing concrete. Methods for testing cement in a reference concrete.
BS 1881-201: 1986	Testing concrete. Guide to the use of non-destructive methods of test for hardened concrete.

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BS 1881-207: 1992	Testing concrete. Recommendations for the assessment of concrete strength by near-to-surface tests.
BS 1881-209: 1990	Testing concrete. Recommendations for the measurement of dynamic modulus of elasticity.
BS ISO 1920-8: 2009	Testing of concrete – Part 8: Determination of drying shrinkage of concrete for samples prepared in the field or laboratory.
BS EN: 1992-1-1: 2004	Eurocode 2: Design of concrete structures. General rules and rules for buildings.
BS EN 1992-3: 2006	Eurocode 2: Design of concrete structures. Liquid retaining and containing structures.
BS 3148: 1986	Methods of test for water for making concrete.
BS 3892-1: 1997	Pulverized-fuel ash. Specification for pulverized fuel ash for use with Portland cement.
BS 3892-2: 1996	Pulverized-fuel ash to be used as a Type I addition.
BS 3963: 1974 (1980)	Method for testing the mixing performance of concrete mixers.
BS 4550-3: 1978	Methods of testing cement. Physical tests. Introduction.
BS 5075-2: 1982 (w/d)	Concrete admixtures. Specification for air-entraining admixtures.
BS 5328-1: 1997 (w/d)	Concrete. Guide to specifying concrete.
BS 5328-2: 2002 (w/d)	Concrete. Methods for specifying concrete mixes.
BS 5328-4: 1990 (w/d)	Concrete. Specification for the procedures to be used in sampling, testing and assessing compliance of concrete.
BS ISO 5725-1: 1994	Accuracy (trueness and precision) of measurement methods and results.
BS 6089: 2010	Assessment of in situ compressive strength in structures and precast concrete components. Complementary guidance to that is given in BS EN 1379.

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PD 6682-1: 2009	Aggregates. Aggregates for concrete. Guidance on the use of BS EN 12620.
BS 7542: 1992	Method of test for curing compounds for concrete.
BS 8110-1: 1997	Structural use of concrete. Code of practice for design and construction.
BS 8500-1: 2006	Concrete. Complementary British Standard to BS EN 206-1. Method of specifying and guidance for the specifier.
BS 8500-2: 2006	Concrete. Complementary British Standard to BS EN 206-1. Specification for constituent materials and concrete.
BS EN 12350-1,2,3,4,5,6,7: 2009	Testing fresh concrete. Various tests.
BS EN 12350-8,9,10,11,12: 2010	Testing fresh concrete. Self-compacting.
BS EN 12390-1: 2000	Testing hardened concrete. Shape, dimensions and other requirements for specimens and moulds.
BS EN 12390-2: 2009	Testing hardened concrete. Making and curing specimens for strength tests.
BS EN 12390-3: 2009	Testing hardened concrete. Compressive strength of test specimens.
BS EN 12390-4: 2000	Testing hardened concrete. Compressive strength. Specification for testing machines.
BS EN 12390-5: 2009	Testing hardened concrete. Flexural strength of test specimens.
BS EN 12390-6: 2009	Testing hardened concrete. Tensile splitting strength of test specimens.
BS EN 12390-7: 2009	Testing hardened concrete. Density of hardened concrete.
BS EN 12504-1: 2009	Testing concrete in structures. Cored specimens. Taking, examining and testing in compression.
BS EN 12504-4: 2004	Testing concrete. Determination of ultrasonic pulse velocity.
BS EN 12620: 2002	Aggregates for concrete.
BS EN 12878: 2005	Pigments for the colouring of building materials based on cement and/or lime. Specifications and methods of test.
BS EN 13055-1: 2002	Lightweight aggregates. Lightweight aggregates for concrete mortar and grout.
BS EN 13670: 2009	Execution of concrete structures.
BS EN 14487-2: 2006	Sprayed concrete. Execution.
BS EN 14647: 2005	Calcium aluminate cement. Composition, specifications and conformity criteria.
BS 15743: 2010	Supersulfated cement – Composition, specifications and conformity criteria.

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