Edited by **John Newman** • **Ban Seng Choo**

Advanced Concrete Technology

Concrete Properties

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Contents

Part 1 Fresh concrete

Part 2 Setting and hardening of concrete

Part 3 Properties of hardened concrete

Part 4 Durability of concrete and concrete construction

10 Concrete and fire exposure 10/1

Bob Cather

11 Freeze/thaw resistance 11/1

Michel Pigeon, Bruno Zuber and Jacques Marchand

12 Acid, soft water and sulfate attack 12/1

Steve Kelham

Preface

The book is based on the syllabus and learning objectives devised by the Institute of Concrete Technology for the Advanced Concrete Technology (ACT) course. The first ACT course was held in 1968 at the Fulmer Grange Training Centre of the Cement and Concrete Association (now the British Cement Association). Following a re-organization of the BCA the course was presented at Imperial College London from 1982 to 1986 and at Nottingham University from 1996 to 2002. With advances in computer-based communications technology the traditional residential course has now been replaced in the UK by a web-based distance learning version to focus more on self-learning rather than teaching and to allow better access for participants outside the UK. This book, as well as being a reference document in its own right, provides the core material for the new ACT course and is divided into four volumes covering the following general areas:

- constituent materials
- properties and performance of concrete
- types of concrete and the associated processes, plant and techniques for its use in construction
- testing and quality control processes.

The aim is to provide readers with an in-depth knowledge of a wide variety of topics within the field of concrete technology at an advanced level. To this end, the chapters are written by acknowledged specialists in their fields.

The book has taken a relatively long time to assemble in view of the many authors so the contents are a snapshot of the world of concrete within this timescale. It is hoped that the book will be revised at regular intervals to reflect changes in materials, techniques and standards.

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PART 1

Fresh concrete

1

Fresh concrete

P.L. Domone

1.1 Introduction

Fresh concrete is a transient material with continuously changing properties. It is, however, essential that these are such that the concrete can be handled, transported, placed, compacted and finished to form a homogenous, usually void-free, solid mass that realizes the full-potential hardened properties. A wide range of techniques and systems are available for these processes, and the concrete technologist, producer and user must ensure that the concrete is suitable for those proposed or favoured.

Fresh concrete technology has advanced at a pace similar to many other aspects of concrete technology over the past three decades, and indeed many of these advances have been inter-dependent. For example, the availability of superplasticizers has enabled workable concrete to be produced at lower water/binder ratios thus increasing the *in-situ* strength.

In this chapter, we will start by considering the property known as *workability**, including its definition and common methods of measurement. We will point out the limitations of these, and show how this leads to the need for a more fundamental scientific description of the behaviour of fresh cement pastes and concrete. We will then describe how this has been achieved by applying the principles of *rheology*, and explain the development and use of test methods which give a more complete understanding of the behaviour. We will then discuss the effect on the rheological properties of a range of constituent materials, including admixtures and cement replacement materials, and how a knowledge of these properties can be used to advantage. The factors that influence the loss of workability before setting are then briefly considered.

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*The alternative term 'consistence' is often used, particularly in specifications and standards.

We will not discuss the specific properties required for particular handling or placing techniques such as pumping, slipform construction, underwater concreting etc. These are covered in various chapters in Volume 3 of this series, but hopefully the more general description given in this chapter will be of value when reading these. We will, however, describe the principles of ensuring that the concrete is correctly placed and compacted to give a uniform, homogenous result. Finally, we will discuss the behaviour of the concrete after placing but before setting, with particular reference to segregation and bleed.

1.2 Workability

1.2.1 Terminology and definitions

Problems of terminology and definition are immediately encountered in any discussion of the fresh properties of concrete. Every experienced concrete technologist, producer and handler has an understanding of the nature and properties of the material, and can choose from a wide variety of terms and expressions to describe it; examples include harsh, cohesive, lean, stiff, rich, etc. Unfortunately, all these terms, and many others, are both subjective and qualitative, and even those that purport to be quantitative, e.g. slump, give a very limited and sometimes misleading picture, as we will see. This is not to say that such terms and values should not be used, but that they must be used with caution, particularly when trying to describe or specify the properties unambiguously.

A satisfactory definition of workability is by no means straightforward. Over 50 years ago, Glanville, *et al*. (1947), after an extensive study of fresh concrete properties, defined workability as ‘the amount of work needed to produce full compaction’, thereby relating it to the placing rather than the handling process. A more recent ACI definition has encompassed other operations; it 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’ (ACI, 1990). This makes no attempt to define how the workability can be measured or specified. A similar criticism applies to the ASTM definition of ‘that property determining the effort required to manipulate a freshly mixed quantity of concrete with minimum loss of homogeneity’ (ASTM, 1993).

Such definitions are clearly inadequate for the description, specification and quality control of fresh concrete, and many attempts have been to provide a more satisfactory definition which includes quantitative measurements. These are sometimes more restrictive, for example the ACI (1990) definition of consistency as ‘the relative mobility or ability of freshly mixed concrete to flow’, which is measured by the slump test. This difficulty illustrates that no single test or measurement can properly describe all of the required properties of the fresh concrete.

(Tattersall 1991) has proposed a division of the terminology relating to workability into three classes:

Class 1: *Qualitative*, to be used in a general descriptive way without any attempt to quantify, e.g. workability, flowability, compactability, stability, pumpability.
Class 2: *Quantitative empirical*, to be used as a simple quantitative statement of behaviour in a particular set of circumstances, e.g. slump, flow table spread.

Class 3: *Quantitative fundamental*, to be used strictly in accordance with the definitions in BS 5168: Glossary of rheological terms, e.g. viscosity, mobility, fluidity, yield value.[1]

Such a division is helpful in that it clearly exposes the limitations of many of the terms, and it will be useful to keep this in mind when reading this chapter.

1.2.2 Measurement of workability by quantitative empirical methods

Many tests have been devised and used over many years to produce quantitative empirical values in Class 2 above. They give a single measurement, and are therefore often referred to as ‘single-point’ tests, to distinguish them from the ‘two-point tests’ which give two measurements, and which we will describe later.

As long ago as 1947, twenty-nine single-point tests were described as the more important of those developed up to that time (Glanville *et al*., 1947). A recent compendium of tests has included sixteen single-point tests, and therefore at least this number are likely to be in current use (RILEM, 2002). Few, if any, of the tests described are suitable for the complete range of workabilities used in practice. Indeed, many have been developed in the past two decades in response to the use of increasingly higher workability concrete, including, most recently, self-compacting concrete.

Four tests have a current British Standard: *slump, compacting factor, Vebe* and *flow table* (or more simply, *flow*), and will now be discussed together with the slump flow test, an adaptation of the slump test for self-compacting concrete, and the degree of compactability test, which has replaced the compacting factor test in the recent European Standards. The tests are shown and described in Figures 1.1–1.6. Table 1.1 gives the principles on which they operate, and some comments on their use.

The slump test (Figure 1.1), which is simple, quick and cheap, is almost universally used for nearly all types of medium and high workability concrete. As well as the drawbacks listed in Table 1.1, there are also some differences in practice with its use in different countries, particularly with respect to the British and American standards.

First, the British and European Standards specify that the slump should be measured to the highest point of the concrete, whereas the American standard specifies measurement to the displaced original centre of the top surface of the concrete (as shown in Figure 1.1). Clearly, the same test on the same concrete can give different values depending on where it is performed.

Second, the British standard only recognizes values from a true slump as valid, and does not allow recording of values from either shear or collapsed slump (Figure 1.1); the American standard includes a similar restriction for shear slump, but allows measurements of a collapsed slump, and values of 250 mm and above are often reported. The recent European standard states that the test is sensitive to changes in consistency corresponding to slumps between 10 and 200 mm, and the test is not considered suitable beyond these extremes.

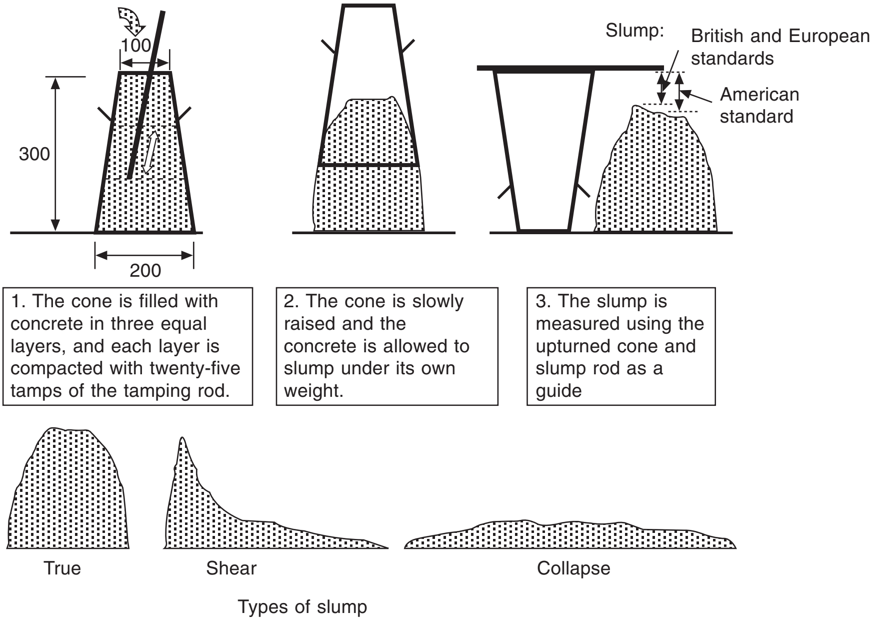
The flow table (Figure 1.4) test was introduced initially to German standards when superplasticizers and high workability flowing concrete (i.e. collapsed slump) started to

---

[1] A list of the relevant standards can be found at the end of the chapter.

Table 1.1 Common single-point workability tests

Test	Principle	Comments
Slump (Figure 1.1)	measures a flow property of concrete under self-weight after standard compaction	• suitable for medium and high workability concrete • sensitive to small changes in water content • very simple, suitable for site use • heavily operator dependent
Compacting factor (Figure 1.2)	measures the effect of a standard amount of work (height of fall) on compaction	• suitable for low, medium and high workability mixes • fairly simple, but requires scales • less operator dependent than slump
Vebe (Figure 1.3)	measures the amount of work (time at constant vibration) for full compaction	• suitable for very low and low workability mixes • greater relation to concrete placing conditions than slump • more complex than other methods, requires standard vibrating equipment • sometimes difficult to define end point
Flow table (Figure 1.4)	measures the effect of a standard amount of work (bumps) on spread	• suitable for high and very high workability mixes • gives some indication of tendency of mix to segregate • fairly simple, but, like slump, operator dependent
Slump flow (Figure 1.5)	as in the slump test, measures a flow property of concrete under self-weight, but after self-weight compaction	• developed for self-compacting concrete • very simple, suitable for site use • operator dependent, but less so than slump
Degree of compactability (Figure 1.6)	measures the effect of a standard amount of work (dropping the concrete from the edge of the container) on compaction	• an alternative to the compacting factor test • simple, suitable for site use • likely to be operator dependent

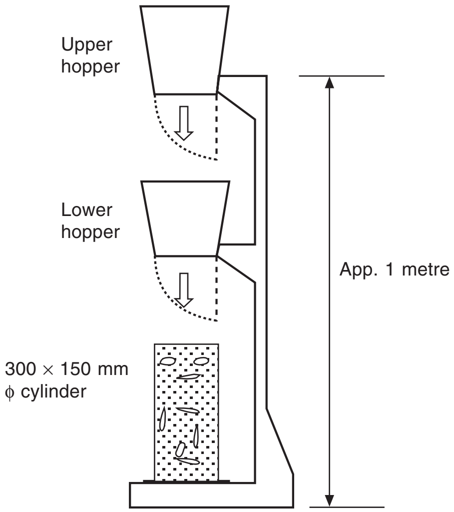


1. The cone is filled with concrete in three equal layers, and each layer is compacted with twenty-five tamps of the tamping rod.

2. The cone is slowly raised and the concrete is allowed to slump under its own weight.

3. The slump is measured using the upturned cone and slump rod as a guide

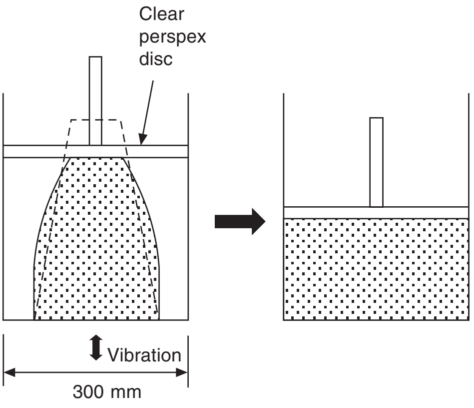
Figure 1.1 The slump test (BS 1881 Part 102: 1983; BS EN 12350-2: 2000; ASTM C 143-90a).



1. Concrete is loaded into the upper hopper
2. The trap door is opened, and the concrete falls into the lower hopper
3. The trap door is opened, and the concrete falls into the cyinder
4. The concrete is struck off level with the top of the cylinder
5. The cylinder + concrete is weighed, to give the partially compacted weight of concrete
6. The cylinder is filled with fully compacted concrete
7. The cylinder + concrete is weighed, to give the fully compacted weight of concrete

$$\text{Compacting factor} = \frac{\text{weight of partially compacted concrete}}{\text{weight of fully compacted concrete}}$$

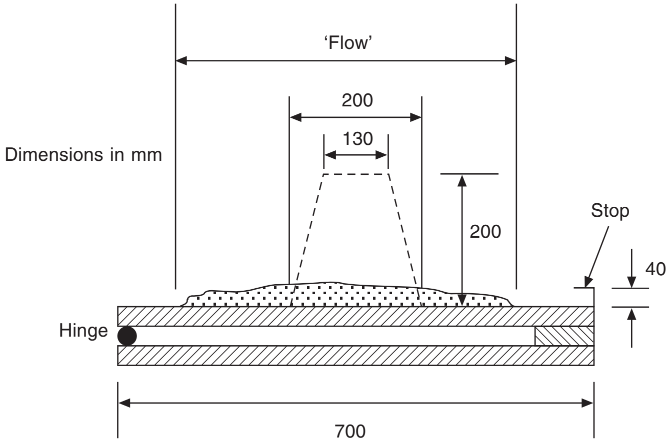
Figure 1.2 The compacting factor test (BS 1881 Part 103: 1993).



1. A slump test is performed in a rigid container
2. A clear perspex disc, free to move vertically, is lowered onto the concrete surface
3. Vibration at a standard rate is applied with a vibrating table

Vebe degrees is the time (in seconds) to complete covering of the underside of the disc with concrete

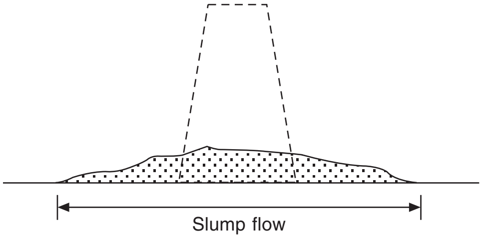
Figure 1.3 The Vebe test (BS 1881 Part 104: 1983, BS EN 12350-3: 2000).



1. A conical mould (2/3 the height of that in the slump test) is used to produce a sample of concrete in the centre of a 700 mm square board, hinged along one edge
2. The free edge of the board is lifted against the stop and dropped 15 times

Flow = final diameter of the concrete
(mean of two measurements at right angles)

Figure 1.4 The flow table test (BS 1881 Part 105: 1984, BS EN 12350-5: 2000).

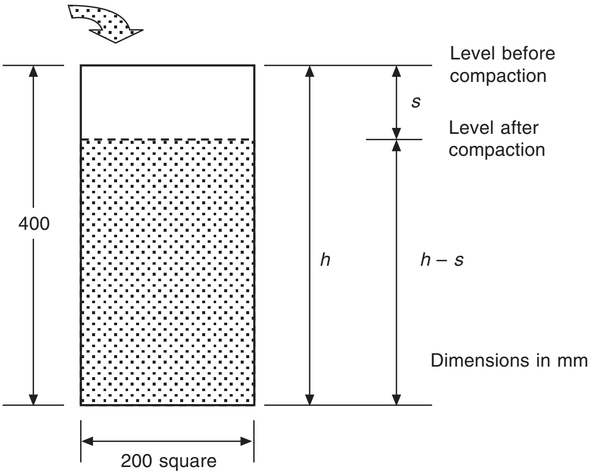


1. A slump cone (see Figure 1.1) is filled without compaction
2. The cone is lifted and the slump flow is final diameter of spread (mean of two diameters at right angles)
3. The time to reach a spread of 500 mm is sometimes also measured
4. The baseboard must be smooth, clean and level

Figure 1.5 The slump flow test.

become popular in the 1970s. However, this test was criticized (Dimond and Bloomer, 1977) even before its first inclusion in British Standards in 1983, for several reasons, including:

- The test is operator sensitive, potentially more so than the slump test;
- When the spread exceeds 510 mm, the recommended minimum for flowing concrete (Cement and Concrete Association, 1978), the concrete thickness is about the same as a 20 mm aggregate particle, and the test cannot therefore be a satisfactory measure of the bulk concrete properties;
- There is a high degree of correlation between the initial spread before jolting and the final spread after jolting, and thus no extra information is gained by the jolting.



1. The container is filled with concrete, using a trowel, from all four edges in turn
2. Excess concrete is struck off with a straight edge
3. The concrete is compacted by vibration
4. The height *s* is measured at the mid-point of each side, and the mean of the four readings calculated

Degree of compactability = $h/(h - s)$
(to two decimal places)

Figure 1.6 The degree of compactability test (BS EN 12350-4: 2000).

The relationships between slump and flow table results from three sources are shown in Figure 1.7; two of these indicate an S-shaped relationship showing the increased sensitivity of the flow table test at higher slumps, but the third is linear between slumps of 100 and 250 mm. However, the scatter is sufficiently wide to encompass both forms of the relationship.

The slump flow test (Figure 1.5) could be considered as an alternative to the flow table test, and, as already mentioned, is widely used for testing high-fluidity self-compacting

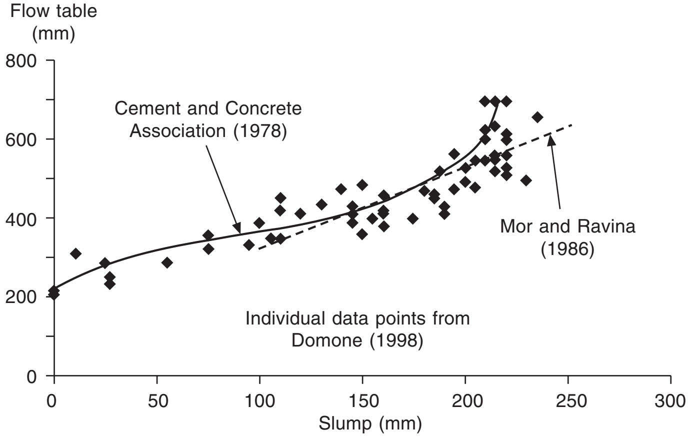


Figure 1.7 The relationship between slump and flow table measurements.

concrete (it has been standardized for this purpose in Japan). The only extra complication over the slump test is that the result is more sensitive to the surface condition of the board on which the test is performed. The relationship between slump amd slump flow from three test programmes is shown in Figure 1.8. Not surprisingly this shows that, at slumps above about 200 mm, the latter is much more sensitive to changes in the concrete fluidity. The best-fit relationships diverge at higher slumps, which may reflect differences in practice, e.g. in the measurement of slump as discussed above.

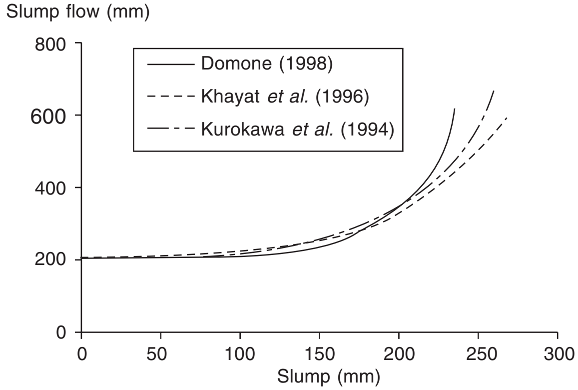


Figure 1.8 The relationship between slump and slump flow measurements.

Since the tests listed in Table 1.1 are based on several different principles, and measure different properties, it is not surprising that only a very wide degree of correlation is obtained between them, with considerable scatter. This is illustrated by the data plotted in Figure 1.9, from a single but comprehensive test series. These broad general relationships are reflected in the consistence classes given in the European standard for concrete specification, EN 206: 2000, which are listed in Table 1.2. The standard states that the classes are not directly related, but they are consistent with the relationships shown in Figures 1.7 and 1.9

Table 1.2 Consistence classes according to BS EN 206-1: 2000

Slump		Vebe		Degree of compactability		Flow	
Class	Range (mm)	Class	Range (secs)	Class	Range	Class	Range (mm)
		V0	≥ 31	C0	≥ 1.46		
S1	10–40	V1	30–21	C1	1.45–1.26	F1	≤ 340
S2	50–90	V2	20–11	C2	1.25–1.11	F2	350–410
S3	100–150	V3	10–6	C3	1.10–1.04	F3	420–480
S4	160–210	V4	5–3			F4	490–550
S5	≥ 220					F5	560–620
						F6	≥ 630

The situation is further complicated by the fact that, in some instances, if different tests are used to either rank or differentiate between mixes, conflicting results can be obtained. For example, Table 1.3 gives the slump, Vebe and compacting factor values of

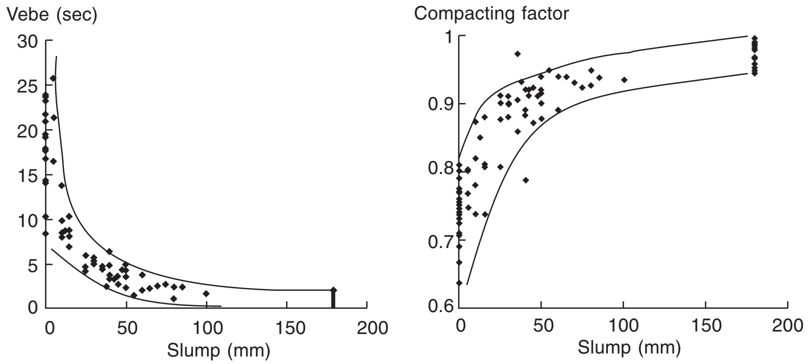


Figure 1.9 Typical spread of results from single-point workability tests (data from Ellis, 1977).

Table 1.3 Slump, Vebe and compacting factor results from four mixes (data from Ellis, 1977)

Mix	Slump (mm)	Vebe (sec)	Compacting factor
A	25	4.3	0.91
B	50	4.9	0.88
C	40	3.3	0.92
D	35	4.4	0.97

four mixes selected from the results of the test programme which gave rise to Figure 1.9. Ranking them in order of increasing workability gives:

by slump: Mix A → Mix D → Mix C → Mix B

by Vebe: Mix B → Mix D → Mix A → Mix C

by compacting factor: Mix B → Mix A → Mix C → Mix D

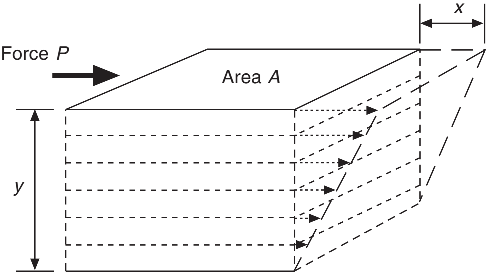
These different rankings are clearly unsatisfactory – not only do the tests have limitations, but they can also be misleading.

For a greater understanding of the behaviour in general, and an explanation of the anomalies that can arise from single point testing in particular, we need to turn to the science of rheology, and to consider the developments in the application of this to fresh concrete that have taken place over the past thirty years or so.

1.2.3 Rheology of liquids and solid suspensions

Rheology is the science of the deformation and flow of matter, and hence it is concerned with the relationships between stress, strain, rate of strain and time. We are concerned with flow and movement, and so we are interested in the relationship between stress and rate of strain.

Fluids flow by the action of shear stress causing a sliding movement between successive adjacent layers, as illustrated for laminar (non-turbulent) flow in Figure 1.10. The relationship between shear stress (τ) and rate of shear strain ($\dot{\gamma}$) is called the flow curve, and can take a variety of forms, as shown in Figure 1.11. The simplest form is a straight line passing



applied shear stress (τ) = force/area = P/A
shear strain (γ) = shear displacement/base length = x/y
rate of change of shear strain = $d\gamma/dt = \dot{\gamma}$

Figure 1.10 Shear flow in a fluid under the action of a shear force.

through the origin. This is called Newtonian behaviour, and is a characteristic of most simple liquids, such as water, white spirit, petrol, lubricating oil, etc., and of many true solutions, e.g. sugar in water. The equation of the line is

$$\tau = \eta \cdot \dot{\gamma}$$

and the single constant η (called the *coefficient of viscosity*) is sufficient to fully describe the flow behaviour.

The other forms of flow curves in Figure 1.11 all intercept the shear stress axis at some positive, non-zero value, i.e. flow will only commence when the shear stress exceeds this threshold value, which is often called the *yield stress*. This is a characteristic of solid suspensions, i.e. solid particles in a liquid phase, of which cement paste, mortar and concrete are good examples. A wide range of equations have been proposed to model the various shapes of flow curves found in practice, but for our purposes it is sufficient to consider a general equation of the form:

$$\tau = \tau_0 + a \cdot \dot{\gamma}^n$$

where τ_0 is the intercept on the shear stress axis, and a and n are constants. The three lines shown have different values of n. In *shear thinning* behaviour, the curve is convex to the

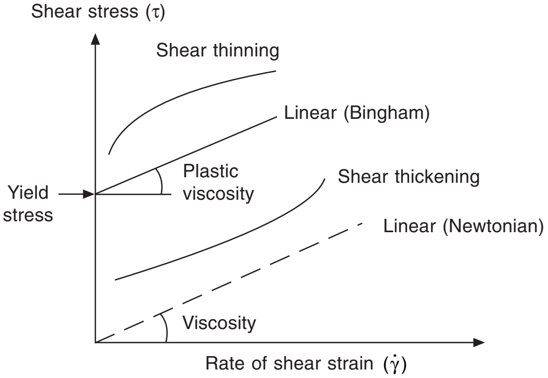


Figure 1.11 Types of flow curves.

shear stress axis and $n < 1$; in *shear thickening*, the curve is concave to the shear stress axis and $n > 1$. The particular case of a straight line relationship is called *Bingham* behaviour, for which $n = 1$. The equation for this is normally written as

$$\tau = \tau_y + \mu \cdot \dot{\gamma}$$

where τ_y is the *yield stress*, and μ is the *plastic viscosity*.

This is of particular interest as there is considerable evidence from tests over the past thirty years that the behaviour of fresh cement paste, mortar and concrete conforms well to this model. Some recent studies have shown that some types of concrete containing high amounts of binder and superplasticizers show non-linear behaviour, i.e. $n \neq 1$ (e.g. de Larrard *et al*., 1998), but the simpler Bingham model is appropriate and sufficient for most cement paste, mortar and concrete. This means, of course, that values of *two* constants, yield stress and plastic viscosity, are necessary to define the behaviour. We will now discuss methods of measuring these, and how they are influenced by the mix proportions and constituents.

1.2.4 Tests on cement paste

Instruments that measure the relationship between shear stress and strain rate are called viscometers or rheometers (the two terms are, in effect, interchangeable). There are several forms of such instruments, and a coaxial cylinder viscometer, as illustrated in Figure 1.11, is perhaps the most common. In the version shown in Figure 1.12, the inner cylinder is rotated and the torque imposed on the stationery outer cylinder is measured; other rotating and measuring arrangements are possible. For testing most liquids, the

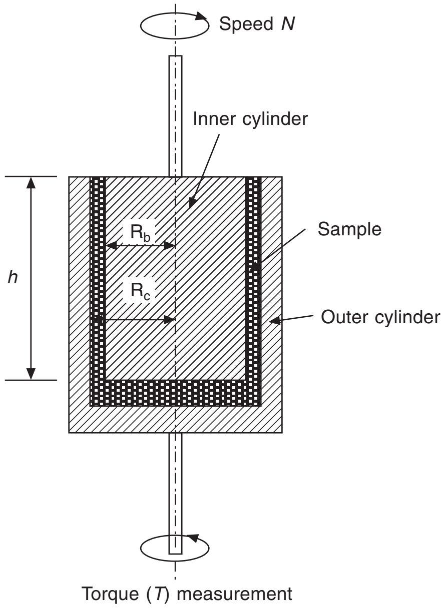


Figure 1.12 A coaxial cylinder viscometer.

annular gap between the inner and outer cylinders is typically of the order of few millimetres, and this is suitable for testing cement paste, in which the maximum particle size is about 100 μm.

For low shear rates and/or viscous liquids, the flow is uniform and laminar and can readily be analysed; the resulting linear equation relating the measured torque (*T*) to the rotational speed (*N*) for a Bingham fluid is:

$$N = \frac{T}{2\mu h}\left(\frac{1}{R_b^2} - \frac{1}{R_c^2}\right) - \frac{\tau_y}{\mu} \cdot \ln\left(\frac{R_b}{R_c}\right) \qquad \text{(symbols as in Figure 1.12)}$$

and hence τ_y and μ can be calculated from the measured flow curve of *T* versus. *N*, and the instrument geometry. (*Note*: this is known as the Reiner–Rivlin equation; a full analysis can be found in Tattersall and Banfill, 1983.) There has been considerable debate over the requirements for and methods of avoiding particle sedimentation and slippage on the cylinder surfaces. Although these have not been fully resolved, a consensus of typical behaviour can be identified.

First, taking the simplest mixture of cement and water, varying the water/cement ratio produces a fan-shaped family of flow curves such as that in Figure 1.13, which shows that both the yield stress and plastic viscosity reduce with increasing water content. Figure 1.14 shows some values for Portland cement mixes, from which it can be seen that the magnitude of the changes of both the yield stress and plastic are similar, i.e. adding or subtracting water produces similar proportional changes in both properties.

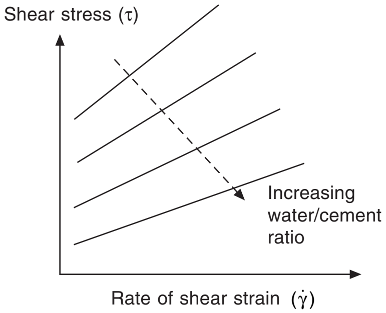


Figure 1.13 Flow curves for cement pastes with varying water/cement ratio.

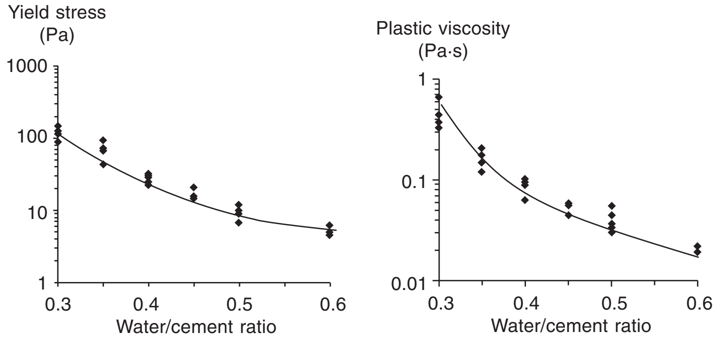


Figure 1.14 Typical effect of water/cement ratio on Bingham constants for cement paste (Domone and Thurairatnam, 1988).

The behaviour is, however, somewhat different when the fluidity of the paste is increased with a superplasticizer. Typical data for the addition of a naphthalene formaldehyde superplasticizer to pastes with three different water/cement ratios are shown in Figure 1.15. With increasing admixture dosage, the proportional reduction in the plastic viscosity is much less than that in the yield stress.

The data of Figures 1.14 and 1.15 can be combined into a single diagram of yield stress versus plastic viscosity, as in Figure 1.16. From the individual data points lines of equal water/cement ratios and superplasticizer dosages can be drawn. The latter are much steeper than the former and indeed, are near vertical over much of the range. Clearly the mechanisms of fluidity increase by water and superplasticizer must be different – both make the flow easier to initiate, i.e. they reduce the yield stress, but superplasticizers maintain the viscosity. Such diagrams are extremely useful in showing these interactive effects, and we will use them later to describe the more complicated behaviour of concrete.

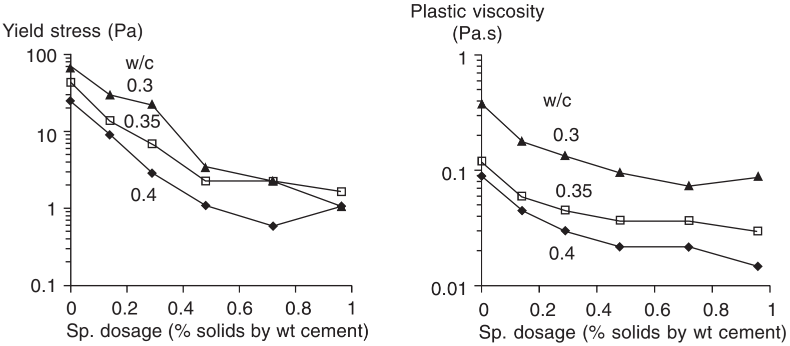


Figure 1.15 Typical effect of superplasticizer on Bingham constants for cement paste (Domone and Thurairatnam, 1988).

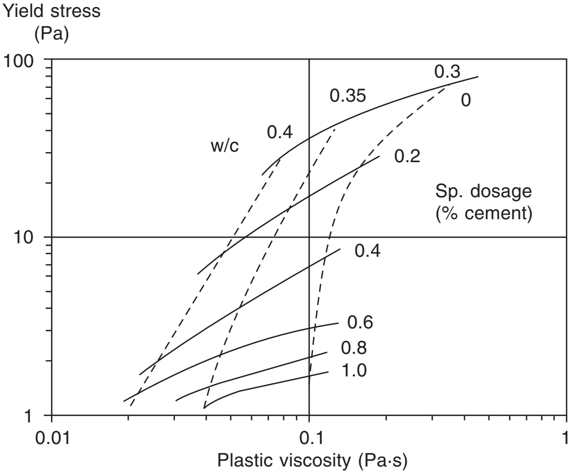


Figure 1.16 Yield stress/plastic viscosity diagram for cement paste with varying water/cement ratio and superplasticizer dosage (constructed from the data in Figures 1.14 and 1.15).

1.2.5 Tests on concrete

For concrete, the presence of coarse aggregate means that a much larger sample needs to be tested. Three main test systems have been developed:

1. A concentric cylinder apparatus with ribbed cylinders to prevent slippage at the cylinder surfaces, called the BML viscometer (Figure 1.17).

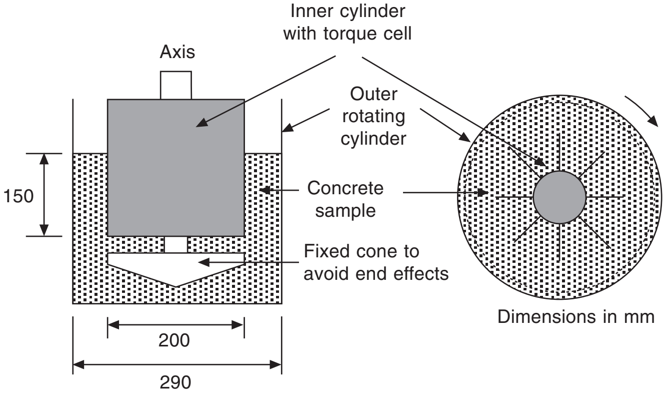


Figure 1.17 The BML viscometer (Wallevik and Gjorv, 1990; RILEM, 2002) (the dimensions are those of the most commonly used system).

2. A parallel plate system in which a cylindrical sample of concrete is sheared between two circular parallel plates, again with ribs to prevent slippage, called the BT RHEOM (Figure 1.18).

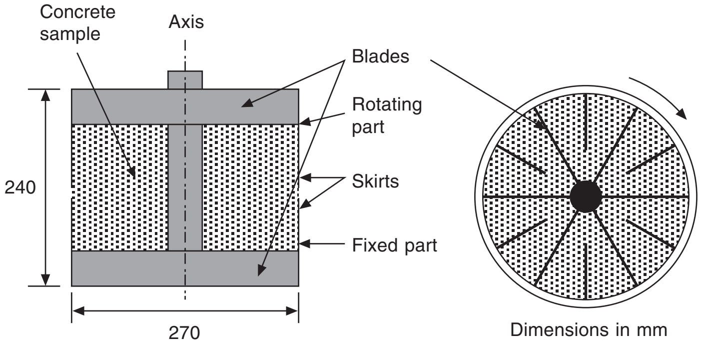
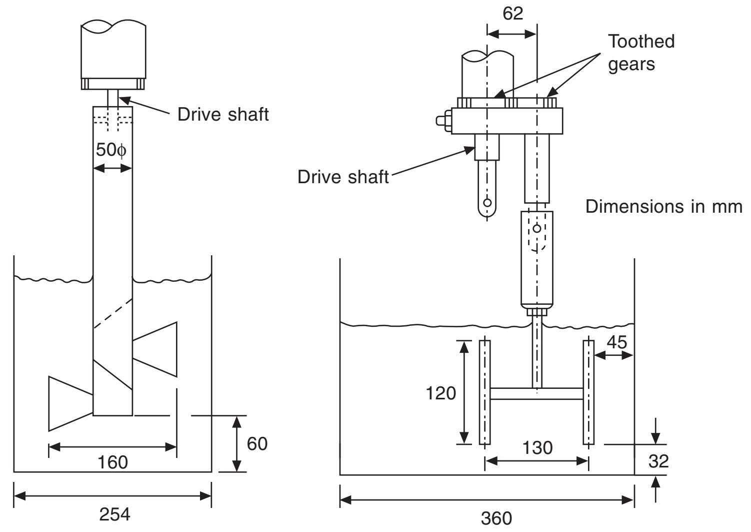


Figure 1.18 The BT RHEOM rheometer (de Larrard *et al.*, 1997; RILEM 2002).

3. A system based on a mixing action in which an impeller is rotated in a bowl of concrete, known as the Tattersall (after the leader of its development team), or two-point workability test. Two alternative impeller types can be used:
    - an interrupted helix for medium- and high-workability mixes (the MH system) (Figure 1.19(a))
    - an H-shaped blade with a planetary motion within the concrete for medium-to-low-workability mixes (the LM system) (Figure 1.19(b)).



(a) Interrupted helix impeller (b) Offset H impeller

Figure 1.19 The two impeller systems for the two-point workability test (Domone, Xu and Banfill, 1999; RILEM, 2002).

All these tests give a flow curve in the form of the relationship between applied torque (*T*) and speed of rotation of the moving part (*N*). For the great majority of concrete mixes, a straight-line relationship of the form

$$T = g + h \cdot N$$

fits the data well. This is, of course, Bingham behaviour in which *g* is a yield term and *h* a viscosity term.

The relationships of *g* to yield stress (τ_y) and *h* to plastic viscosity (μ) depend on the flow pattern generated by the test and the apparatus size and geometry, which are all clearly different for each apparatus. Analytical relationships have been obtained for the BML viscometer and the BT RHEOM by assuming laminar uniform flow, but the flow pattern in the two-point test is far too complex for this, and the relationship has to be obtained by testing calibration fluids of known properties.

Since yield stress and plastic viscosity are fundamental properties of a Bingham fluid, any test should give the same values of these for the same concrete. For several years, rigorous comparison of data was impossible since different workers in different countries favoured one instrument or another, but it seemed that different values were being obtained for at least similar concrete. To quantify and try to resolve these differences, a series of comparative tests was carried out in 2000, in which all three instruments were taken to the same laboratory and simultaneously tested a series of fresh concrete mixes with a wide range of rheology (Banfill *et al.*, 2001). Two other instruments were also used in the test programme: an IBB rheometer which was essentially the two-point workability test with the offset H impeller (Figure 1.19), but which did not give results in fundamental units, and a large concentric cylinder viscometer previously used for measuring the flow of mountain debris, and which it was hoped would provide a rigorous control data.

The results confirmed that all the instruments did indeed give differing values of yield stress and plastic viscosity for the same mix, but that

- they each ranked all the mixes in approximately similar order for both yield stress and plastic viscosity;
- pairwise comparison of the results gave highly significant correlations.

In both cases the yield stress values were somewhat more consistent than those of plastic viscosity. Although the reasons for the differences between the instruments were not resolved, the results were very encouraging and at least enabled data from the different instruments in different places at different times to be compared.

However, we should also recognize that irrespective of their absolute value, it is equally important to know how τ_y and μ (or indeed g and h) *vary* with the concrete's component materials, mix proportions etc., and there is a considerable amount of published information on this. Figure 1.20 is a schematic summary of typical effects of varying a number of factors individually, compiled from several sources. This shows that:

- The effects of water content and (super)plasticizers are similar to those found in cement paste as discussed above. Increasing or decreasing the water content changes both yield stress and plastic viscosity, whereas the admixtures reduce the yield stress at largely constant plastic viscosity; large doses of plasticizers and superplasticizers can have diverging effects.
- Partial replacement of cement by either pulverized-fuel ash (pfa) or ground granulated blast furnace slag (ggbs) primarily reduces the yield stress, with a reduction in viscosity in the case of pfa, and an increase with ggbs.
- More paste leads to a higher viscosity but a lower yield stress, i.e. the mix tends to flow more readily, but is more cohesive, a property often qualitatively called 'rich' or 'fatty'. Mixes with less paste, although tending to flow less readily, are less viscous – 'harsh' or 'bony'.
- Air-entraining agents tend to reduce the viscosity at near-constant yield stress.

All these effects, although typical, will not necessarily occur with all mixes, and the behaviour can vary according to the type and source of component materials (particularly admixtures) and the properties of the initial mix, i.e. the starting point in Figure 1.20.

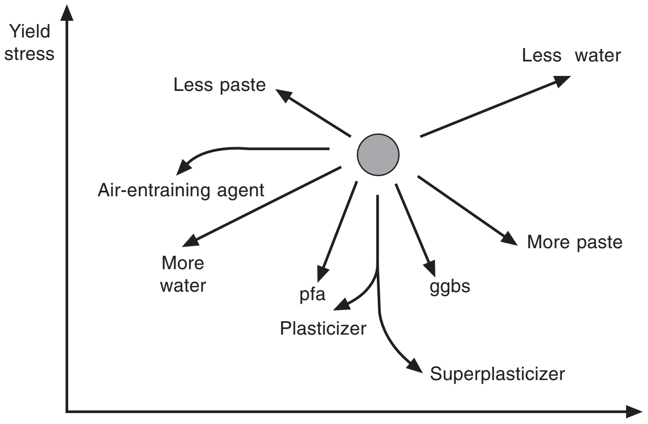


Figure 1.20 Summary of the effect of varying the proportions of concrete constituents on the Bingham constants.

Also, it is difficult to predict the interactive effects of two or more variables; an example of this is shown in Figure 1.21 for mixes containing varying cement and microsilica contents. Small amounts of microsilica reduce the plastic viscosity, with almost no effect on the yield stress; however, above a threshold level of microsilica, which depends on the cement content, there is a substantial increase in the yield stress, followed by an increase in the plastic viscosity.

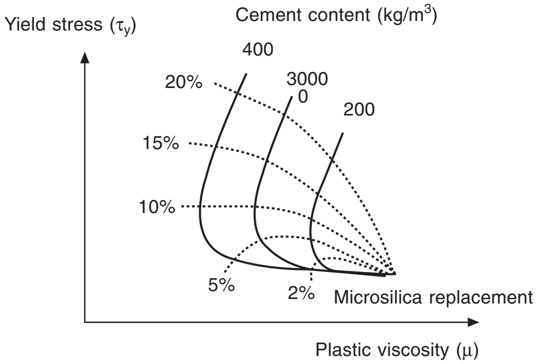


Figure 1.21 Variation of the Bingham constants of mixes containing microsilica (Gjorv, 1997).

1.2.6 Relation of single-point test measurements to Bingham constants

As we have discussed above, with the 'conventional' single-point tests (slump, Vebe, etc.) only one value is measured. In each test, the concrete is moving, but at a different shear rate in each case. Each test will have an associated average shear rate (albeit difficult to define in most cases), and is therefore equivalent to determining only one point on the T versus N (or τ versus $\dot{\gamma}$) graph.

In the slump test, the rate of movement is small and the concrete is at rest when the slump is measured, i.e. the shear rate is zero or near zero throughout, and therefore a relationship between slump and yield stress might be expected. This has indeed been found to be the case in many test programmes, starting with some of the earliest published work (Tattersall and Banfill, 1983). Results from two recent experimental programmes are shown in Figure 1.22. These are for a range of mixes with and without superplasticizers and cement replacement materials. Both sets of data considered individually show a good correlation between slump and yield stress (with some 'outliers'), confirming the earlier findings with a more limited range of mix variables. Ferraris and de Larrard obtained their data in Figure 1.22 with the BTRHEOM, and Domone *et al.* used the two-point workability test. Although the two sets of data overlap, they increasingly digress at lower workabilities, which is consistent with the results of the comparative test programme described in the previous section.

It also follows that no relationship between slump and plastic viscosity should necessarily exist. This is confirmed in Figure 1.23, which shows the companion data obtained by Ferraris and de Larrard to that in Figure 1.22.

The fact that different single-point tests operate at different equivalent shear rates provides an explanation for the confusing and sometimes misleading conclusions that can

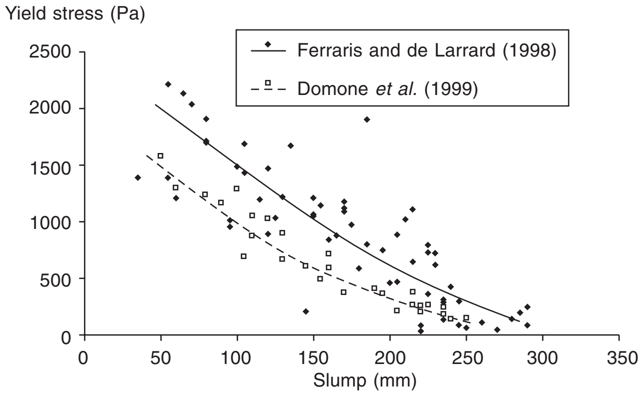


Figure 1.22 Slump and yield stress results for a wide range of mixes.

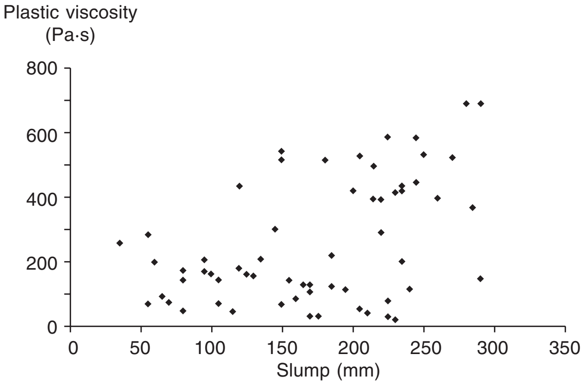


Figure 1.23 Slump and plastic viscosity results for a range of mixes (Ferraris and de Larrard, 1998).

be obtained by using two tests on the same mix that we discussed at the end of Section 1.2.2. Figure 1.24 shows flow curves of two mixes which intercept within the range of equivalent shear rates of two single-point tests – for example, obtained with mixes with varying water content and superplasticizer dosage. Test 1, with a low equivalent shear

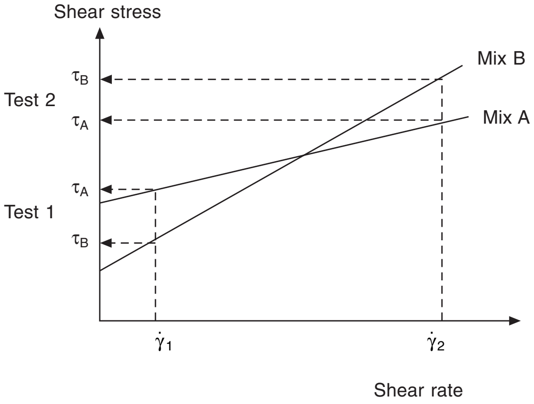


Figure 1.24 Intersecting flow curves for two mixes which give conflicting results with single-point tests.

rate of $\dot{\gamma}_1$, will rank mix A as less workable than mix B ($\tau_A > \tau_B$); test 2, however, operating with a higher equivalent shear rate $\dot{\gamma}_2$, will rank mix A as more workable than mix B ($\tau_A < \tau_B$). The inherent limitations of single-point tests are clear. No systematic studies have been done on the relationship between two-point test results and those of other single-point tests, e.g. compacting factor.

1.2.7 Cohesion, segregation and stability

A trained and experienced observer can readily estimate the cohesion or 'stickiness' of a mix. This is an important property, but a suitable test has not yet been developed; a recent report (Masterston and Wilson, 1997) has commented on the need for one. Some indication of the cohesiveness can, however, be obtained during slump, slump flow or flow table tests. For concrete with a true slump (Figure 1.1), if the concrete is tapped gently after measuring the slump, a cohesive mix will slump further, but a non-cohesive mix will fall apart. For high-workability mixes tested by slump flow or flow table, a ring of cement paste extending for several millimetres beyond the coarse aggregate at the end of the test indicates poor cohesion and instability.

It can be argued that plastic viscosity is a measure of cohesion. For example, the maintenance and perhaps increase in plastic viscosity with superplasticizer dose shown in Figure 1.20 explains how high slump (i.e. low yield stress) yet stable concrete, the so-called *flowing concrete*, can be produced with appropriate use of these admixtures.

1.2.8 Quality control with rheological tests

The extra information about mixes that can be obtained with rheological tests can be used to advantage in quality control. This can be illustrated with the following hypothetical example.

Tests on successive truckloads of nominally the same concrete gave the results shown in Table 1.4 (the g and h values were obtained with the two-point workability test, and have not been converted to τ_y and μ). The mix contained Portland cement and a superplasticizer. The specified slump was 75 mm, and so on arrival at site loads 2 and 4 could have been rejected on the basis of the slump value. However, there were two possible reasons for the excessive slump – too much water or too much superplasticizer. Examination of the g and h values shows that for mix 2, both g and h are much lower than those of the satisfactory mixes 1, 3, 4 and 5; however, with mix 6, g is lower but h is

Table 1.4 Results of quality control tests on successive loads of the same concrete mix

Load no.	Slump (mm)	g (Nm)	h (Nm)
1	85	4.5	3.5
2	150	2.8	1.9
3	75	5.0	4.1
4	80	4.8	3.9
5	75	5.2	4.4
6	140	2.9	4.1

within the range of mixes 1, 3, 4, and 5. A look at Figure 1.20 will show that it is most likely that mix 2 was over watered, and hence should be rejected. However, mix 6 will have had an overdose of superplasticizer, and provided it was stable and there were no other problems such as an unacceptable increase in setting time, the long-term strength will not be affected, and so it need not be rejected.

1.2.9 Rheology of high-performance concrete

The last ten to fifteen years have seen the development and increasing use of several types of high-performance concrete, such as high-strength concrete, high-durability concrete, fibre-reinforced concrete, underwater concrete and self-compacting concrete. Most of these contain a combination of admixtures, cement replacement materials etc. and will therefore have very different rheological properties to those of 'normal' mixes. Describing the workability of such concretes with a single-point test (such as slump) has even more perils than with normal performance mixes, and using the Bingham constants is therefore extremely useful in producing mixes which can be satisfactorily handled and placed.

Figure 1.25 shows the regions of the yield stress/plastic viscosity diagram for four types of concrete. In 'normal' concrete, in which the workability is controlled mainly by water content, the yield stress and plastic viscosity will vary together, as already discussed. Flowing concrete, produced by adding superplasticizer to a normal mix (with perhaps a higher fines content to ensure stability), has a yield stress lower than that of normal concrete, and hence a high slump, but a relatively high viscosity for stability. High-strength concrete mixes, which have a high paste content commonly containing microsilica, can be viscous and sticky, making them difficult to handle despite including superplasticizers to produce a high slump/low yield stress. Self-compacting concrete, which needs to flow under self-weight through and around closely spaced reinforcement without segregating or entrapping air is perhaps the best example of a rheologically controlled mix (Okamura, 1996). The yield stress must be very low to assist flow, but the viscosity must be high enough to ensure stability, but not so high for flow to be prohibitively slow. All these types of concrete are discussed in more detail elsewhere in these volumes.

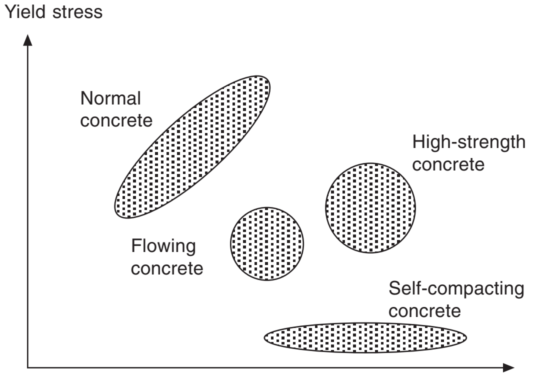


Figure 1.25 Rheology of several types of concrete.

It is appropriate here to quote de Larrard (1999), who concluded that knowledge of the rheological behaviour of fresh concrete allows the user to perform rapid, successful placement of high-quality concrete, saving time and money, and producing structures of long service life.

1.3 Loss of workability

Fresh concrete loses workability due to

- mix water being absorbed by the aggregate if this not in a saturated state before mixing
- evaporation of the mix water
- early hydration reactions (but this should not be confused with cement setting)
- interactions between admixtures (particularly plasticizers and superplasticizers) and the cementitious constituents of the mix.

Absorption of water by the aggregate can be avoided by ensuring that saturated aggregate is used, for example by spraying aggregate stockpiles with water and keeping them covered in hot/dry weather, although this may be difficult in some regions. It is also difficult, and perhaps undesirable, with lightweight aggregates. Evaporation of mix water can be reduced by keeping the concrete covered during transport and handling as far as possible. These two subjects are discussed in greater detail elsewhere in these volumes.

Most available data relates to loss of slump, which increases with

- higher temperatures
- higher initial slump
- higher cement content
- high alkali and low sulfate content of the cement

Figure 1.26 shows data from two mixes differing in water content only which illustrate the first two factors.

The rate of loss of workability can be reduced by continued agitation of the concrete, e.g. in a readymix truck, or modified by admixtures, again as discussed elsewhere. In principle, retempering, i.e. adding water to compensate for slump loss, should not have

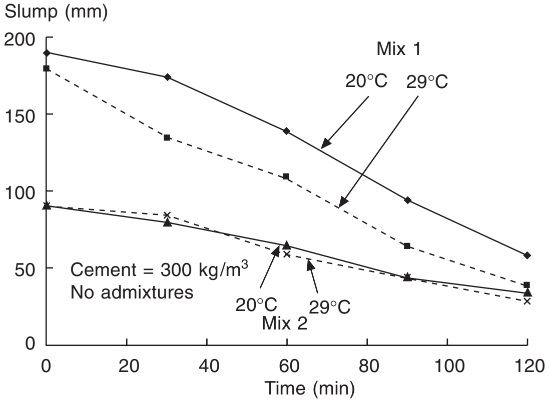


Figure 1.26 Typical slump loss behaviour of mixes without admixtures (Previte, 1977).

a significant effect on strength if only that water which has been lost by evaporation is replaced. Also, studies have shown that water can be added during retempering to increase the initial water/cement ratio by up to 5 per cent without any loss in 28-day strength (Cheong and Lee, 1993). However, except in very controlled circumstances, retempering can lead to unacceptably increased water/cement ratio and hence lower strength, and is therefore best avoided.

1.4 Placing and compaction

The methods chosen for placing and compacting the concrete will depend on the type of construction, the total volume to be placed, the required rate of placing and the preferences and expertise of the construction companies involved. There are, however, several basic rules which should be followed to ensure that the concrete is properly placed and compacted into a uniform, void free mass once it has been delivered to the formwork in a satisfactory state:

- The concrete should be discharged as close as possible to its final position, preferably straight into the formwork;
- A substantial free-fall distance will encourage segregation and should therefore be avoided;
- With deep pours, the rate of placing should be such that the layer of concrete below that being placed should not have set; this will ensure full continuity between layers, and avoid cold joints and planes of weakness in the hardened concrete;
- Once the concrete is in place, vibration, either internal or external, should be used to mould the concrete around embedments e.g. reinforcement, and to eliminate pockets of entrapped air, but the vibration should not be used to move the concrete into place;
- High-workability mixes should not be overvibrated – this may cause segregation.

The behaviour of concrete during vibration has two stages:

1. initial settlement in which the coarse aggregate particles are moved into a more stable position;
2. entrapped air bubbles rising to the surface.

The stages can be quite distinct with low-workability concrete, but stage 1 is less apparent in high-workability concrete.

In terms of concrete rheology, vibration has the effect of reducing or overcoming the yield stress, allowing the concrete to behave like a Newtonian liquid, and there is evidence that the plastic viscosity is not substantially affected by the vibration, as illustrated in Figure 1.27. Hence mixes with a low plastic viscosity will be easier to compact even though they have a high yield stress (or low slump), provided the vibration energy is sufficient to overcome this. Figure 1.20 shows that air entrainment has the effect of reducing plastic viscosity at near constant yield stress, which explains why such mixes are relatively easy to compact.

Vibration frequencies in common use vary from 50 to 200 Hz, with table and surface vibrators operating at the lower end of this range, and formwork and internal poker vibrators at the upper end. There is evidence that the most important parameter governing

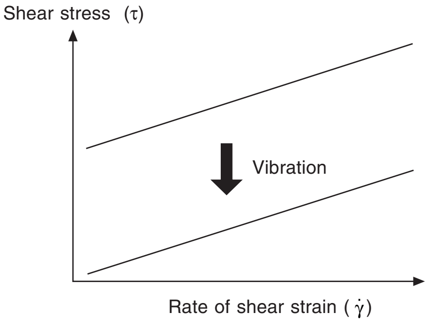


Figure 1.27 The effect of vibration on the flow curve of fresh concrete.

the effectiveness of the vibration is the peak velocity (V_{max}), rather than the frequency or amplitude separately (Banfill *et al.*, 1999). This is given by

$$V_{max} = 2\pi fA$$

where f = frequency and A = amplitude.

1.5 Segregation and bleed after placing

Fresh concrete is a mixture of solid particles with specific gravities ranging from about 2.6 (most aggregates) to 3.15 (Portland cement). After the concrete has been placed, the particles tend to settle and the water to rise (Figure 1.28). This can lead to segregation, in which the larger aggregate particles fall to the lower parts of the pour, and/or bleeding, in which water or water-rich grout rises to the surface of the concrete to produce laitance, a weak surface layer, or becomes trapped under the aggregate particles thus enhancing interface transition zone effects. These processes are hindered by the interlocking of the particles and for the smaller particles, the surface forces of attraction. It follows that the major causes of segregation and bleed are poorly graded aggregates and excessive water contents. Bleed also decreases with increasing fineness of the cement, cement content of

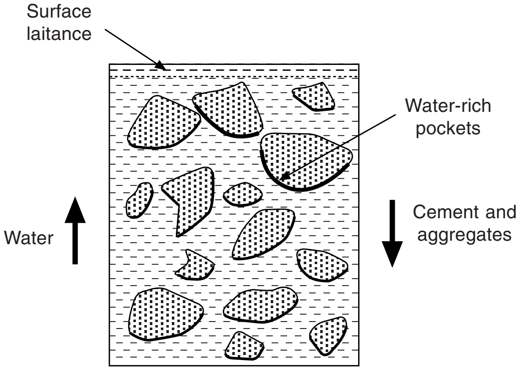


Figure 1.28 Segregation and bleed in freshly placed concrete.

the concrete, and the incorporation of cement replacement materials. It is not possible to generalize about the effect of admixtures.

Some bleed is unavoidable, and may not be harmful. For example, if the concrete is placed in hot or windy conditions, the loss of bleed water from the surface may not cause any distress, and the water/cement ratio of the remaining concrete may be reduced. However, if the rate of evaporation of the water is greater than the rate of bleed, plastic shrinkage, which can lead to surface cracking, will occur.

The combined effects of bleed and particle settlement are that, after hardening, the concrete in the lower part of a pour of any significant depth can be stronger than that in the upper part. This effect is illustrated in Figure 1.29, which shows test data from trial columns. Even though these are of a modest height of 500 mm, the strength differences between the top and bottom are of the order of 10 per cent.

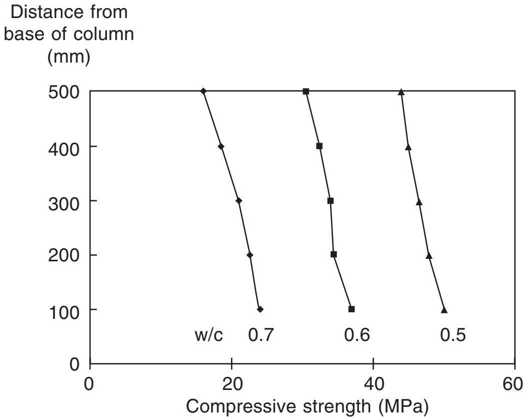


Figure 1.29 Variation of concrete strength in a column after full compaction (Hoshino, 1989).

Bleed can be measured in two ways:

- the reduction in height (i.e. settlement) of a sample of undisturbed concrete;
- the amount of bleed water rising to the surface of an undisturbed sample, which is measured after drawing off with a pipette (e.g. as in ASTM C 232–92).

In both types of test, the rate as well as the total bleed can be measured.

Excessive bleed and segregation can lead to problems of plastic shrinkage and plastic settlement cracking on the top surface of pours. These are discussed in Chapter 2.

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Contains all the relevant theory, background and details of application of rheology to fresh cement and concrete, and summarizes all the pioneering studies of the 1960s and 1970s. An excellent reference text.

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Relevant standards

British standards

(British Standards Institution)
BS 5168: 1975 Glossary of rheological terms
BS 1881: Part 102: 1983 Method for determination of slump
BS 1881: Part 103: 1993 Method for determination of compacting factor
BS 1881: Part 104: 1983 Method for determination of Vebe time
BS 1881: Part 105: 1984 Method for determination of flow

American standards

(American Society for Testing and Materials)
ASTM Specification C 143-90a Test for slump of hydraulic cement concrete. 1990.
ASTM Specification C 232-92 Test for bleeding of concrete. 1992.
ASTM Specification C 125-93 Standard definitions and terms relating to concrete and concrete aggregates.

European standards

(published by British Standards Institution)
BS EN 12350-2: 2000 Testing fresh concrete. Slump test
BS EN 12350-3: 2000 Testing of fresh concrete. Vebe test
BS EN 12350-4: 2000 Testing of fresh concrete. Degree of compactability
BS EN 12350-5: 2000 Testing of fresh concrete. Flow table test
BS EN 206-1: 2000 Concrete – Part 1: Specification, performance, production and conformity

Japanese standards

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PART 2

Setting and hardening of concrete

2

Plastic and thermal cracking

Richard Day and John Clarke

2.1 Introduction

Reinforced concrete is a composite material where load-bearing and deformation properties are determined by the behaviour between the elements – steel and concrete – as well as the individual constituents of these elements, particularly those of the concrete.

Concrete, at all ages, has a low tensile strength compared to the compressive strength. Under load, the tensile strain builds in the tensile zone. This tensile strain is taken up by the reinforcement but it is inevitable that regular but controlled cracking will occur. This is accounted for as part of the structural design process, where the crack widths are limited by an appropriate area of reinforcement suitable to the working environment.

Tensile strain and the possibility of cracking (flexural, shear, torsion, anchorage failure etc.) do not only occur due to structural loading. Micro-cracks will develop in the concrete at the interfaces between the steel and cement paste and aggregate, although only visible through a microscope, due to internal shrinkage etc. This type of cracking is not covered here. Cracks can develop at the unformed surface of immature concrete due to a rapid reduction of volume at the surface (plastic shrinkage). If a concrete bleeds excessively, the denser particles tend to settle over embedded materials, e.g. the reinforcement, causing the near surface to tear (plastic settlement). In these cases the reinforcement does not generally take up any tensile strain, although may affect the crack pattern developed. Cracks can also be associated with temperature cycles, either from the hydrating concrete at early ages or solar gain (thermal), where restraint against movement prevents expansion or contraction so that tensile strain is induced. Here the reinforcement accommodates the tensile forces and influences the crack pattern. Other forms of cracks from chemical actions (ASR, freeze–thaw, reinforcement corrosion) also occur.

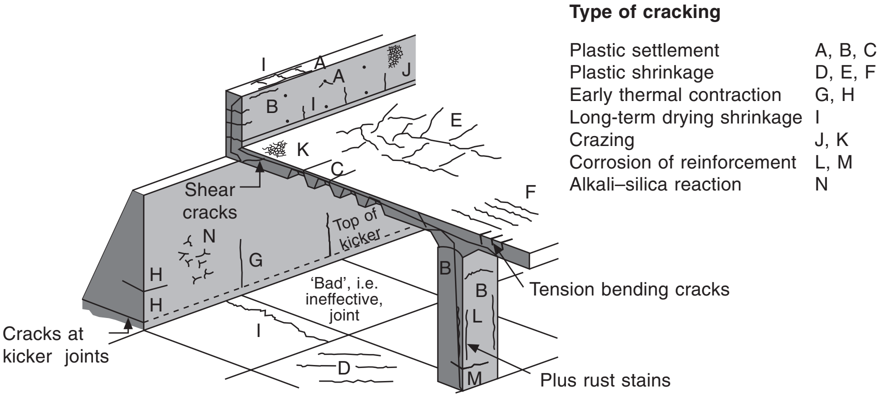
The presence of cracks can influence the behaviour and durability of a concrete member. They can reduce the shear capacity of a section or provide a path by which moisture, oxygen, carbon dioxide, chlorides etc. can penetrate into the concrete surrounding the reinforcement which in time may result in reinforcement corrosion. These aspects are covered in more detail in later chapters. Cracks and crack patterns have different characteristics depending on the underlying cause.

Different types of crack occur at different times in the life of a concrete element (see Table 2.1). So as well as a recognition of a crack pattern, a knowledge of the time of the first appearance of cracks is helpful in diagnosing the underlying cause.

Table 2.1 Typical times for appearance of defects (from Concrete Society Technical Report 54)

Type of defect	Typical time of appearance
Plastic settlement cracks	Ten minutes to three hours
Plastic shrinkage cracks	Thirty minutes to six hours
Crazing	One to seven days – sometimes much longer
Early thermal contraction cracks	One day to two or three weeks
Long-term drying shrinkage cracks	Several weeks or months

The Concrete Society (1992) provides information on the most common forms of ‘intrinsic’ cracks in concrete. Figure 2.1 (taken from the Technical Report) illustrates most of the types of crack that are likely to be experienced in the lifetime of a concrete structure.



Type of cracking

Plastic settlement	A, B, C
Plastic shrinkage	D, E, F
Early thermal contraction	G, H
Long-term drying shrinkage	I
Crazing	J, K
Corrosion of reinforcement	L, M
Alkali–silica reaction	N

Figure 2.1 Examples of intrinsic cracks in hypothetical structure (from Concrete Society, 1992).

The following sections are chiefly concerned with early-age movements but also discuss the longer-term effects of drying shrinkage.

2.2 Plastic cracking

Plastic cracking occurs in the first few hours of the concrete being laid, before it has gained sufficient tensile strength to resist internal tensile stresses. Because they form in the unhardened concrete they are fundamentally different from thermal or other cracks.

Plastic settlement cracks typically occur in deeper sections such as walls, columns and deep beams. Plastic shrinkage cracks are more prevalent on exposed flat slabs. The key to understanding the mechanism for both types of plastic cracking is bleeding.

Bleeding may be described as the relative upward movement of water within fresh concrete accompanied by the downward movement of the heavier particles that are suspended in the concrete matrix. This is caused by the inability of the solid constituents to prevent water movement as they settle under gravity. Bleeding is effectively a form of sedimentation, which is arrested as the particles form bridges, interrupting further downward movement, and as the cement paste hydrates and stiffens. It therefore depends not only on the mix constituents and section dimensions but also the ambient conditions. A major factor in the capacity of a mix to bleed is the grading and consistency of the mix. Mixes that bleed excessively are generally harsh and not cohesive, i.e. contain insufficient fine material. (This subject is covered in Chapter 1.) It must be noted that all concrete experiences some bleeding but it is not a sign of incomplete compaction.

When bleed water is seen it appears as clean water on the surface, but on warm or windy days this may evaporate. It is the combination of the capacity of concrete to bleed and surface evaporation that causes both forms of plastic cracking. The mechanism is discussed in the following sections.

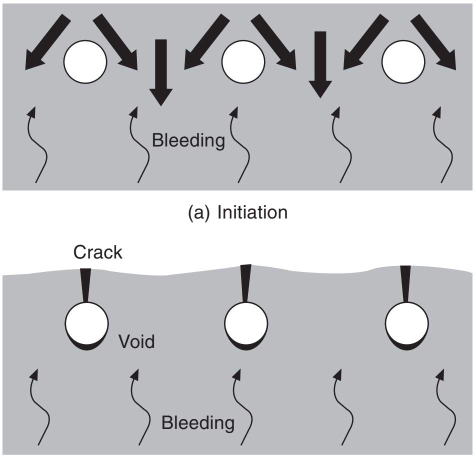
2.3 Plastic settlement cracks

Plastic settlement cracks form within 30 minutes to 6 hours of casting the concrete, dependent on the prevailing conditions and mix characteristics.

2.3.1 The mechanism of plastic settlement

If the settlement of solids in the concrete can freely take place without hindrance there will be a reduction in depth and volume of the cast concrete but no cracking. However, any restraint to this movement, e.g. reinforcement, can result in plastic settlement cracks. Where the solids continue to settle in comparison to those which are prevented from further downward movement, the concrete will ‘break its back’ and a tear appears in the surface as it is forced into tension. Cracks may develop at regular spacing reflecting the reinforcement layout. They often occur in conjunction with voids under the bars as shown in Figure 2.2. Figure 2.2(a) shows initiation and Figure 2.2(b) the condition after a few hours. These crescent-shaped voids may initially be filled with bleed water. The region of bond between the bar and concrete is thus reduced.

The nearer to the surface the restraint occurs, the more likely the formation of cracks, i.e. the less the cover, the greater the chance of cracks. A settlement crack is unlikely to occur if the depth of cover to the reinforcement is greater than one third of the section



(a) Initiation

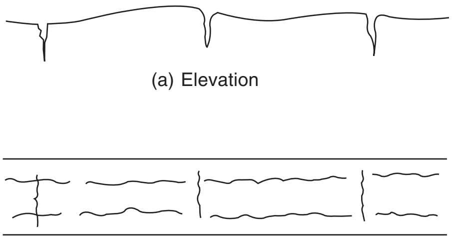
(b) After a few hours

Figure 2.2 Formation of plastic settlement crack (initial and final state).

depth (Turton, 1981). The wind speed (rate of evaporation) and mix proportions (tendency to bleed) would be expected to affect the severity of the cracking. The number of cracks is influenced by the occurrence of the restraint. However, the reinforcement diameter and concrete workability have little influence.

2.3.2 Visual appearance

The most common restraint in slabs is from the reinforcement. The cracks occur on the top surface and usually follow the line of the uppermost bars, giving a series of parallel cracks; there may also be shorter cracks at right angles over the bars running in the opposite direction. Cracks are typically 1 mm wide and usually run from the surface to the bars (see Figures 2.3 and 2.4). The settlement may also result in visible undulations on the concrete surface, with the high points over the top reinforcing bars.



(a) Elevation

(b) Plan

Figure 2.3 General plan view of cracks following bar pattern.

In some cases where the bars in the top layer of reinforcement are close together, the whole surface layer of the concrete may be 'suspended' on the reinforcement while the concrete below settles. This can lead to a horizontal discontinuity beneath the bars,

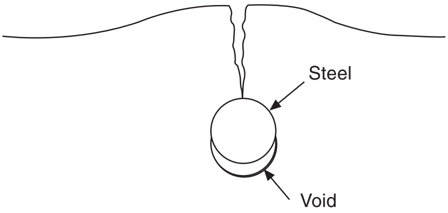


Figure 2.4 Section showing undulations.

resulting in a loss of bond and with time delamination of concrete cover that protects the reinforcing steel against corrosion.

Unlike cracks in hardened concrete, due to overloading for instance, these cracks form at a very early age and pass through the cement paste and do not pass through aggregate particle pieces. The path is therefore more tortuous. This form of crack can be potentially serious as it passes longitudinal with the reinforcement and extends to the steel, negating the resistance to corrosion provided by the concrete.

Fine cracks can occur in relatively narrow formed surfaces such as columns. The concrete may arch between the containing form faces. Settlement below the restrained concrete results in a crack being formed, generally coinciding with the links (see Figure 2.5). It is sometimes possible for plastic settlement cracks to form on a vertical face where reinforcement has restricted the free flow of concrete within the formwork. In such cases it is possible that the cracks are formed between the lines of the reinforcement.

The concrete can also be supported by the formwork face. This causes restraint to the concrete between connected members and is especially evident where changes in section cause differential settlement, the concrete in the deeper section settling more than the shallower section resulting in a crack. This is noticeable in the transition between a flared column head (mushroom) and the plain column, and in trough and waffle slabs where more settlement takes place in the web than the comparatively thin flange (see Figures 2.6 and 2.7). The cracks may pass through the flange and appear similar to shrinkage cracks. It can also occur at other locations, such as under spacer blocks. Cracks at mushroom heads of columns are generally horizontal. They are also typically 1 mm wide and can cross the full section.

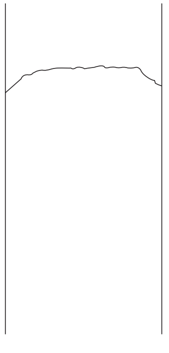


Figure 2.5 Arching near top of column, cracking coinciding with links.

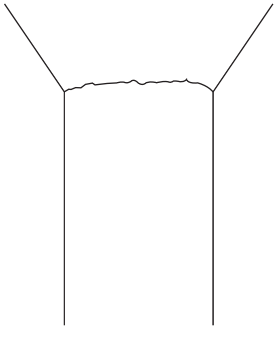


Figure 2.6 Cracks at change of section in mushroom head column.

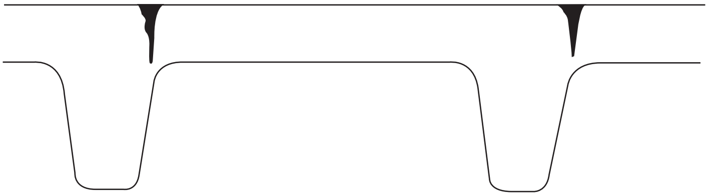


Figure 2.7 Cracks at change of section in trough and waffle slabs.

If the sub-base or other material against which the concrete is placed has a high absorbency (dry soil, permanent forms) the settlement can be exaggerated, again the cracking following the reinforcement layout.

2.3.3 Prevention of plastic settlement cracking

The restraints that cause plastic settlement cracking are inherent in the construction and generally cannot be avoided. Abrupt changes in section depth could be avoided at the design detailing stage but the main reduction of risk is through mix design and suitable cohesion of the concrete to reduce bleeding. In simple terms this can be achieved by increasing the sand content. However, there is a limit to this at which the bleeding will increase. Very clean (marine-dredged) sand tend to assist water movement, so blending with a ‘dirtier’ sand with a higher fines (<150 micron) content can be beneficial.

A tendency to bleed largely depends on the properties of the cement. Fineness is a controlling factor, possibly because the finer particles hydrate more quickly thereby reducing the rate of sedimentation. Rich mixes are less prone than lean mixes. Pozzolanic additions may also help as they reduce water content and add to the fines. High GGBS contents should be avoided as slower setting times allow for the bleeding to continue longer. Air entrainment admixtures can be used, the entrained air stabilizing the matrix and reducing the water movement. A similar effect is claimed for polypropylene fibres within concrete (British Board of Agrément, 1995).

2.3.4 Remedial measures

Plastic settlement cracks rarely pass through the full section, except in cases similar to trough and waffle slabs, mainly because they stop at the reinforcement that causes the restraint. Structural integrity is therefore not compromised, however the cracks need to be sealed especially on slabs, to reduce the risk of reinforcement corrosion.

Where plastic settlement cracking is apparent in the newly placed concrete, the most effective way of eliminating their occurrence is to revibrate the concrete after the cracks have formed but before initial set. Assessing the most appropriate time is the responsibility of the operative. This will vary depending on the mix characteristics (cement type) and ambient conditions. The concrete must be capable of being re-fluidized by a poker vibrator. In general, timely and proper revibration can serve only to improve the situation, if not completely restore the bond beneath the bar. Tamping the surface may close the surface but is unlikely to remove the voids beneath the bar. If noticed early enough, and the cracks remain clean and not filled with detritus, cement can be brushed into the openings and allowed to set. This however does not resolve any bond reduction caused by voids beneath the reinforcement.

Treating cracks in older hardened concrete will depend on the service conditions, i.e. exposure class, and the severity of the cracking. They can be treated with resin injection although in some instances full-depth breaking out and reinstatement may be necessary.

2.4 Plastic shrinkage cracks

Plastic shrinkage cracking should not be confused with drying shrinkage cracking which may occur at much later ages (see Table 2.1). These cracks generally form between 1 and 6 hours after casting although are not generally noticed until the next day. Unlike settlement cracks, they are not directly affected by the proximity of reinforcement to the concrete surface or the layout.

2.4.1 The mechanism of plastic shrinkage

Fresh concrete just after it has been placed has little strength. Water can move relatively freely in what is still a fluid suspension. Water, the least dense component of the mixture, tends to move upwards towards the surface as heavier materials move down during compaction. The upward movement of water is known as bleeding.

Water can be lost at the plastic concrete surface by evaporation, which results in contraction known as plastic shrinkage. Plastic shrinkage is mainly a physical action and is caused by surface tension forces. As the surface dries, menisci are formed between the solid particles and therefore capillary tension forces act (Turton, 1981). The magnitude of the shrinkage is affected by the amount of water lost from the surface which is governed by the temperature, ambient relative humidity, and wind velocity. The rate of loss does not necessarily predict the occurrence of plastic shrinkage, rather the stability of the mix. Basically, if the amount of water lost to evaporation is greater than the rate of bleed there is a net reduction in volume. The surface layer of concrete tries to shrink but is restrained by underlying layers that are not subject to the same reduction in volume. Restraint can also be partly provided by the reinforcement and friction at the surface of the formwork or sub-base. The result of the restraint is that tensile stresses develop in the surface layer. As the concrete is still in a plastic state and has very little strength, cracks develop at the surface. The phenomenon is analogous to the drying shrinkage of clays. The process is illustrated in Figure 2.8, the upper part showing initiation and the lower the condition after a few hours.

Admixtures profoundly alter the chemical forces acting between the cementitious and fine particles within the concrete. The rheological behaviour as well as setting times can be significantly altered. It is therefore important to assess the mix design for its bleeding capacity and set time and plan accordingly. Mixes using water reducers for instance may bleed less and therefore be prone to plastic shrinkage (see Chapter 4 in Volume 1).

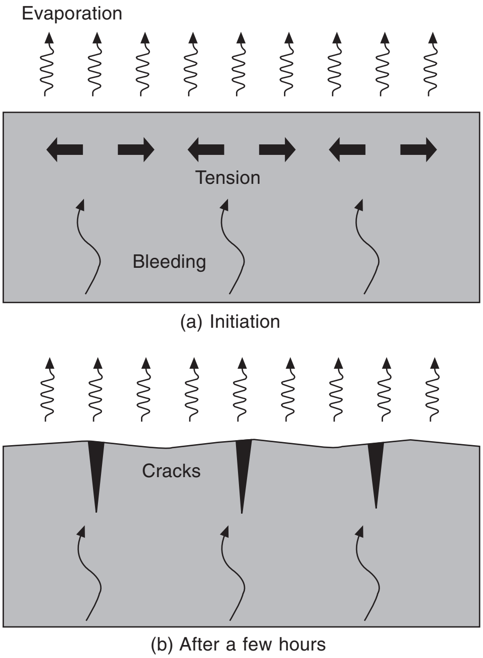


Figure 2.8 Process of plastic shrinkage cracking (initiation and final state).

2.4.2 Visual appearance

Plastic shrinkage cracks tend to be up to 3 mm wide, and vary in length from 50 mm up to 3 m. They are generally 20–50 mm deep rapidly tapering down within the concrete. In some circumstances they may extend through the full depth of a member. The pattern of cracks generally appear at an approximate angle of 45° to the direction of casting and run parallel to one another (see Figure 2.9). The distance between cracks is variable but could be 1 to 2 m. Cracks may also form randomly as a large map pattern (see Figure 2.10). These different patterns may be influenced by the direction in which finishing operations have been carried out or by physical features such as deep tamping marks. A notable characteristic of plastic shrinkage cracks is that they do not normally extend to the edge of a slab as this is able to shrink without restraint. These cracks can form in both unreinforced and reinforced concrete.

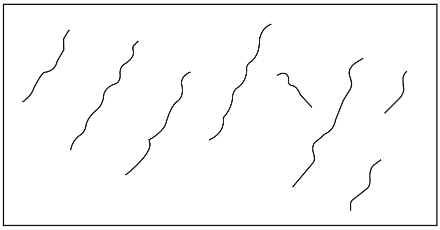


Figure 2.9 Plan of diagonal cracking.

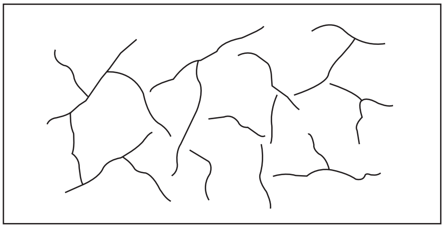


Figure 2.10 Plan of map cracking.

2.4.3 Prevention of plastic shrinkage

The control of the rate of evaporation is key. This is a function of the dryness (relative humidity) of the air and the wind speed. Relative humidity is affected by the temperature therefore in general in the UK there will be a higher rate of evaporation on a hot day than a cold day. Wind speed must not be underestimated. An increase of say, 8 km/h may have an equivalent drying effect as an increase in temperature of 10°C. Precautions to avoid rapid drying are as important in summer as on a cold winter’s day. To prevent plastic shrinkage, efficient curing measures are required, thus reducing evaporation. This should be effected as soon as possible, say within the first hour of casting. (Methods of curing are covered in Chapter 3.) The use of polypropylene fibres (British Board of Agrément, 1995), within concrete is claimed to improve the matrix to reduce bleed and provide localized ‘tensile reinforcement’ in the plastic concrete thus reducing the risk of the surface tearing. However, proper early curing must not be forgotten.

2.4.4 Remedial measures

Plastic shrinkage cracks rarely pass through the full section, the notable exception being trough and waffle slabs or similar elements. As the cracks form in concrete when the paste is still in a plastic state, they run through the paste and around the pieces of aggregate and are not generally wider than 0.5 mm. The two faces of the crack still remain interlocked through the aggregate, restricting vertical movements, unlike a crack that passes through the aggregate. Normally structural integrity is not compromised, except where close to local shear forces, e.g. at a column head. These can be restored with a low-viscosity resin injection. In general cracks need to be sealed, especially on slabs, to reduce the risk of reinforcement corrosion. Often the best remedy is to brush dry cement (damped later) or a wet grout into the cracks before detritus blocks the openings. This should be carried out as soon as possible, to encourage autogenous healing. In pavement slabs where the crack opening exceeds 0.5 mm a low-viscosity resin can be injected before the surface is trafficked, otherwise (and especially for unreinforced slabs) a full depth repair or demolish and replace the entire slab (Burkes Green, 2001) depending on the severity of cracking.

Treating cracks in older hardened concrete will depend on the service conditions, i.e. exposure class, and the severity of the cracking. They can be treated with resin injection although in some instances full-depth breaking out and reinstatement may be necessary.

2.5 Other cracks in plastic concrete

Cracks can be formed in plastic concrete by totally external means, the most common being the accidental movement of formwork. If the formwork moves once the concrete has set but not gained any significant tensile strength to resist disturbance, the paste may tear due to some of the concrete preferentially adhering to the form. If caught early enough the plastic concrete can be revibrated to close the cracks. Depending on the severity the region of the crack may need to be removed and the concrete recast.

Plastic cracking rarely leads to full structural repair and is generally aesthetic in nature. If treated the next day with cement or grout brushed into the surface, before detritus can block the opening, natural healing will be encouraged. However, the structural engineer should always be consulted for approval.

2.6 Early thermal contraction cracks

2.6.1 The mechanism of thermal contraction

The hydration reaction between cement and water that takes place in concrete produces heat. The amount of heat generated and the rate at which it is generated depend on the type of cement and its fineness. The peak temperature is dependent upon the cement type and content, the initial temperature, the ambient conditions, the geometry of the member and the type of formwork. High ambient temperatures speed up the reaction resulting in a more rapid temperature gain. Slabs have a large exposed surface area through which the concrete can lose heat. Members with large cross-sectional areas can develop higher internal temperatures than those with smaller section, as the loss of heat through the top and side surfaces has greater effect in the latter case. Timber formwork provides more insulation than steel and hence higher peak temperatures may be reached when timber formwork is used.

As the concrete heats up it expands. If there is any restraint to this expansion, for example from previous pours, compressive stresses will be generated in the young concrete. These stresses are low, due to the low elastic modulus of the young concrete, and are generally relieved by creep. Once the peak temperature has been reached, at say 12 to 18 hours after placing (though much later for very thick members), the concrete starts to cool and reduce in volume. Restraint to this thermal contraction will result in the development of tensile stresses. At this stage, the concrete is more mature and has less capacity to relieve the strain by creep. Young's modulus is greater and hence the stresses generated are higher. The concrete is still relatively weak in tension and the stresses caused by restraint to temperature-related contraction can cause cracking.

2.6.2 Limiting temperatures

If early thermal contraction is restrained, cracking will occur if the restrained strain (or induced stress) exceeds the capacity of the concrete, i.e. if:

$$T_1 \alpha k R > \varepsilon_{ult}$$

where

T_1 = drop between peak temperature after casting and ambient temperature (°C)
α = coefficient of thermal expansion (per °C)
k = modification factor
R = restraint factor
ε_{ult} = ultimate tensile strain capacity of concrete

BS 8110 takes a value of 0.8 for k and recommends values for R for various sequences of construction as given in Table 2.2.

Table 2.2 Values of restraint (taken from BS 8110)

Pour configuration	Restraint factor R
Thin wall cast onto massive concrete base	0.6 to 0.8 at base 0.1 to 0.2 at top
Massive pour cast onto blinding	0.1 to 0.2
Massive pour cast onto existing mass concrete	0.3 to 0.4 at base 0.1 to 0.2 at top
Suspended slabs	0.2 to 0.4
Infill bays, i.e. rigid restraint	0.8 to 1.0

Two approaches to cracking due to early thermal contraction are possible.

1 Steps can be taken to *avoid* cracks, by limiting the temperature change.
2 If the limiting temperature change is likely to be exceeded, sufficient reinforcement needs to be provided to *control* cracks.

In terms of avoiding cracks, BS 8110 says that ‘Experience has shown that by limiting temperature differentials to 20°C in gravel aggregate concrete, cracking can be avoided’ and gives the limiting differential temperatures shown in Table 2.3.

Table 2.3 Suggested limiting temperature changes to avoid cracking (from BS 8110)

Aggregate type	Limiting differential temperature (°C)
Gravel	20.0
Granite	27.7
Limestone	39.0
Sintered pfa (lightweight aggregate)	54.6

Information on estimating the temperature rise may be found in Harrison (1993). In extreme cases, such as with very large sections or with a high ambient temperature, it may be necessary to consider cooling the fresh concrete (see CIRIA Publication C577, 2002).

2.6.3 Control of cracking

The basic principle for the control of cracking is to provide a minimum amount of reinforcement such that its tensile capacity exceeds that of the concrete when it cracks. Thus:

$$\rho_{crit} = f_{ct}/f_y$$

where

ρ_{crit} = ratio of area of steel reinforcement to area of concrete
f_{ct} = tensile strength of the immature concrete
f_y = characteristic strength of the reinforcement.

To control maximum crack widths, the steel ratio required is:

$$\rho = (kRT_1\alpha\varphi)/(3w_{max})$$

where

φ = bar diameter

w_{max} = maximum crack width.

Further guidance is given in BS 8007 and Reinforced Concrete Council (1993).

Differential temperatures in thick members may also cause cracks. When the surface layer cools and contracts the core of the member is still at a higher temperature. This provides restraint and hence cracks may form in the surface. When the temperature through the member eventually becomes uniform, the surface cracks usually close. In large members, there will tend to be a series of cracks across the short direction in plan and elevation and possibly a series of complimentary cracks in the long direction. Cracks will tend to be wider near corners because heat is lost from two faces at this location.

2.6.4 Visual appearance

A classic case of early thermal contraction cracking is that of walls poured on strip footings that have been cast several days earlier. The stiff strip footings restrain the thermal movements in the wall. Cracks form in the wall, starting at the base and running approximately vertically. They will usually pass right through the wall section. Cracks near the end of bays may be inclined at an angle of approximately 45°. Crack spacing and width will depend on the amount of reinforcement provided. As these cracks form after hardening but before full strength is achieved, they generally run entirely through the paste and not through the aggregate.

2.7 Curling

Curling is the result of differential drying shrinkage between the top and bottom faces of a member. For a ground-supported slab moisture is primarily lost in one direction, towards the top surface. This results in a moisture gradient through the slab that causes the slab to curl. The stress induced on the top, f_{cur}, may be expressed as:

$$f_{cur} = 0.5E\,[\Delta\varepsilon/(1 - \nu)]$$

where

E = modulus of elasticity

ν = Poisson's ratio

$\Delta\varepsilon$ = differential strain between the top and bottom of the slab.

Typically $\Delta\varepsilon$ is taken as $(1.5 - 2.0) \times 10^{-6}$ per mm of slab thickness.

2.8 Crazing

2.8.1 The mechanism of crazing

Crazing can occur both on exposed surfaces and on surfaces in contact with formwork either where there is a change in properties close to the surface or a high moisture content gradient. The type of formwork is also important, as it can affect the permeability of the formed concrete surface. Steel and plastic formwork faces that are smooth and of low permeability appear to increase the incidence of crazing. An example of a surface layer with different properties is the top surface of a slab that has been excessively floated or trowelled to produce a layer of laitance, i.e. a layer consisting mainly of cement paste that may have a locally higher water content than the mass. The surface dries out more quickly than the inner mass and goes into tension.

Crazing can be apparent within a few days of casting but can occur at any time under appropriate climatic conditions, e.g. a period of low humidity leading to drying conditions. It may be that crazing has occurred early in the life of the member but does not become noticed until the cracks are accentuated by deposits of dirt. Because the cracks are so narrow and shallow, crazing is not generally detrimental to durability.

2.8.2 Visual appearance

Crazing is a close pattern of narrow (say 0.1 mm) shallow interconnected cracks usually forming closed polygonal loops. The polygonal areas are typically 10–75 mm across and the cracks are usually only a few millimetres deep.

2.9 Long-term drying shrinkage cracks

2.9.1 The mechanism of long-term drying shrinkage

To aid workability and compaction, the amount of water included in a concrete mix is greater than that required to take part in the hydration reaction with cement. The uncombined water is held within the capillary pores that form within the cement paste. If the concrete in service is exposed to conditions of low relative humidity, moisture will be lost from the surface.

The loss of moisture results in a reduction in volume, known as drying shrinkage. If the shrinkage movement is opposed by some external or internal restraint, stresses will develop in the concrete. When these restraint stresses exceed the tensile capacity of the concrete, cracks will develop. Thin members with a large surface area such as slabs are particularly vulnerable. The time at which shrinkage cracks occur will depend on the rate of drying caused by the environment but, at a maximum rate of development, is usually several months to three or four years after casting. The drying out which leads to shrinkage occurs from the surface and hence the surface layer is first affected. It is possible for the surfaces of members of large cross-section to crack because they are restrained by the inner concrete which has not yet dried. Concrete near to corners is particularly prone to cracking as loss of moisture takes place from the two adjacent surfaces.

The amount of shrinkage for a typical concrete with a water content of 190 l/m^3 can be estimated from Figure 2.11.

Although most cases of drying shrinkage cracking are attributable to changes in volume of the cement paste, some aggregates are susceptible to moisture. Such aggregates have to be used with caution and may require special consideration if undesirable shrinkage or expansion is to be avoided. Their use may also result in greater deflections of members during seasonal drying out.

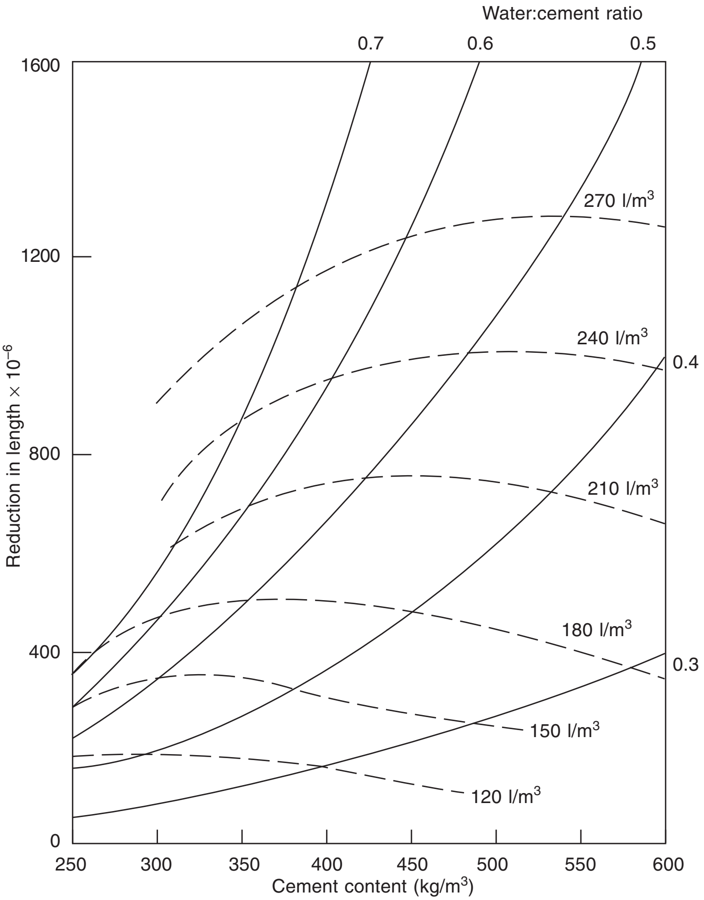


Figure 2.11 Estimation of drying shrinkage.

2.9.2 Visual appearance

There is no typical pattern of drying shrinkage cracking as the cracks form at any location where there is a restraint to shrinkage movement. The cracks are, however, usually approximately at right angles to the direction of restraint. Figure 2.12 shows drying shrinkage cracks on a wall. The widths of the cracks will depend on the extent to which the concrete has been allowed to dry out and the length of the member or the distance between positions of restraint. Because the cracks form after the concrete has gained full strength, the cracks can pass through weak aggregate.

Typical locations where drying shrinkage cracks are likely to occur are:

- Ground slabs where one horizontal dimension is much greater than the other. The cracks form across the middle of the slab, parallel to the shorter side. Cracks sometimes form diagonally across corners.
- Suspended slabs supported on stiff edge beams. The location of cracking can be influenced by voids in the slab such as those left for services or stairwells.
- At significant changes in cross-section.

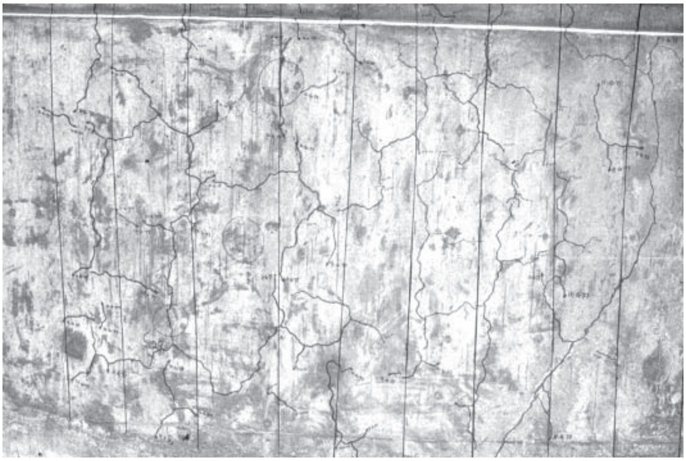


Figure 2.12 Drying shrinkage cracks on wall. Vertical lines indicate the position of reinforcement.

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3

Curing

Bryan Marsh

3.1 Aims and objectives

The aim of this chapter is to examine when and why curing of *in-situ* concrete is necessary and what effect curing has on its hardened properties.

3.2 What is curing?

A typical definition of curing (BS 8110, 1997) is 'the process of preventing the loss of moisture from the concrete whilst maintaining a satisfactory temperature regime'. This particular definition adds that the curing regime should prevent the development of high-temperature gradients within the concrete.

Many other definitions exist which include references to hydration, durability and cost but there are three basic elements to consider:

- Moisture
- Heat
- Time

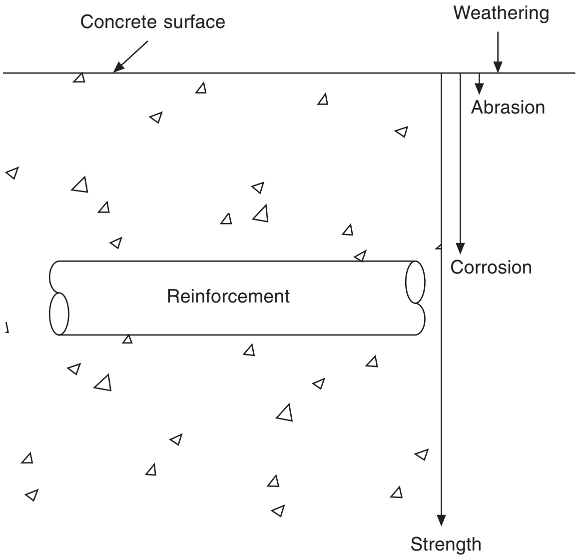
3.3 Why cure concrete?

According to the British Standard for the structural use of concrete, BS 8110 (1997), the intention of curing is to protect concrete against:

- premature drying out, particularly by solar radiation and wind (plastic shrinkage)
- leaching out by rain and flowing water
- rapid cooling during the first few days after placing
- high internal thermal gradients
- low temperature or frost
- vibration and impact which may disrupt the concrete and interfere with bond to reinforcement

Adequate curing will facilitate, but not necessarily ensure, the optimal development of the surface zone of fresh, newly cast concrete into strong, impermeable, crack-free and durable-hardened concrete. The objective is to keep the concrete saturated, or as near saturated as possible, for sufficient time for the original water-filled space to become filled to the desired extent by cement hydration products. According to the research reviewed in CIRIA (1997), the depth of the surface zone directly affected by curing can be up to 20 mm in temperate climatic conditions, and up to 50 mm in more extreme arid conditions. Properties of the concrete beyond this zone are unlikely to be affected significantly by normal curing.

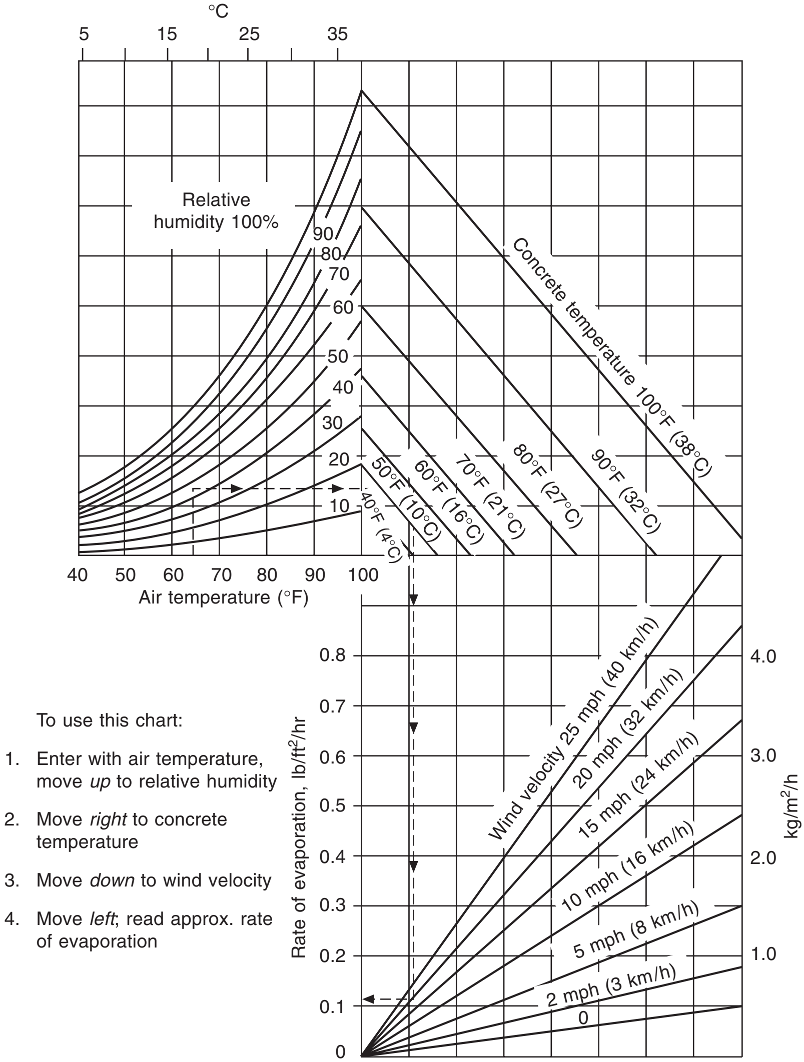
It is, however, this surface zone that is often relied upon to provide many of the essential requisites of a concrete structure or element such as abrasion and chemical resistance and protection of embedded reinforcement. Figure 3.1 shows the relative depths from the surface at which various properties will be affected by inadequate curing. The importance of appropriate curing must therefore not be overlooked.



The arrows indicate the region of concrete influencing the deterioration mechanisms in question.

Figure 3.1 The relative depths at which various properties are affected by inadequate curing (CIRIA, 1997).

The rate of evaporation of water from the surface, taking into account the combined influences of the ambient temperature and relative humidity, the concrete temperature, and the wind velocity can be estimated from Figure 3.2 taken from ACI 308 (1992). This standard requires that curing measures are taken if the predicted rate of evaporation exceeds 1.0 kg/m^2/h, to prevent plastic shrinkage cracking, but also recommends that such measures may be needed if the rate exceeds 0.5 kg/m^2/h.



To use this chart:

1. Enter with air temperature, move *up* to relative humidity
2. Move *right* to concrete temperature
3. Move *down* to wind velocity
4. Move *left*; read approx. rate of evaporation

Figure 3.2 The effect of concrete and air temperatures, relative humidity, and wind velocity on the rate of evaporation of surface moisture from concrete (ACI 308, 1992).

In the absence of any deliberate curing measures the rate of water loss during the first few hours is similar to that of any wet surface. Nevertheless, once the concrete has set, the rate of evaporation of water from the near surface of concrete in temperate climates is relatively slow (depending upon the concrete composition, actual weather conditions, etc.) and hydration of concrete in the outer zone will be able to proceed to a certain degree. Research has shown that in the absence of any curing (laboratory air storage) the time taken for the relative humidity in 0.59 water/cement ratio Portland cement concrete to fall to 95 per cent is 4 days at a depth of 7.5 mm, 18 days at 11.5 mm, and 111 days at 33.5 mm (Parrott, 1988). These times will be greater for lower w/c concretes and higher for blended cements. These figures, however, relate to controlled laboratory conditions and will be affected in practice by ambient relative humidity, air temperature, wind speed and concrete temperature.

Relative humidity within concrete can be measured using electronic probes inserted into pre-drilled holes of the required depth. These probes typically work by dew-point sensing on a chilled plate or by use of a humidity-sensing film sensor that produces an electrical change such as capacitance or resistance (Dill, 2000). They are best suited to laboratory studies and are unlikely to be encountered on site.

3.4 How can curing be achieved in practice?

The most common methods of site curing of *in-situ* concrete are:

- formwork retention
- suspension of covering above the surface before the concrete has set (horizontal surfaces)
- spraying with water
- ponding with water
- covering with wet sand, earth, sawdust, straw, or periodically wetted hessian or cotton mats, or the use of an absorbent covering with access to water
- application of a curing membrane
- waterproof reinforced paper or plastic sheeting
- tenting or other shelter against drying winds
- sunshields
- covering with an insulating layer or heated enclosure

No specific measures are needed when the ambient conditions of moisture, humidity and temperature are sufficiently favourable to provide adequate curing by themselves. But great caution should be exercised in making such a judgement – general assumptions about climate can be dangerous, particularly in temperate climates such as the UK. For example, the *annual average relative humidity,* which may be approximately 80 per cent, is of absolutely no relevance on a hot dry summer day when the relative humidity may be as low as 50 per cent or less.

3.5 Which curing method is best?

In theory the amount of free water in most concretes is more than sufficient for complete hydration. Preventing loss of moisture has thus traditionally been believed to provide adequate curing. Nevertheless, the rate of hydration is considerably slowed by a drop in the relative humidity level within the pore structure of the concrete. Below *95 per cent* relative humidity it is believed that the rate of hydration will be slowed to the extent that no further reduction in large porosity will occur even though hydration itself will not actually cease until about *80 per cent* relative humidity. This is significant as both strength and durability are related to porosity, with large porosity having a particularly large influence on durability. Sealed concretes of w/c below about 0.5 will experience self-desiccation (Neville, 1995), i.e. the water consumed by hydration will reduce the relative humidity level in the pore space such that further hydration will not occur. Thus, in theory at least, optimum properties will only be obtained by use of curing methods which allow ingress of water into concrete (ponding, spraying) rather than those that just prevent or reduce loss of moisture. Consideration must, however, be given to the potential problems of using direct application of free water outlined in the following sections.

In reality the best curing method is the one that actually gets done, and gets done properly!

3.5.1 Retention of formwork

Where the constraints of the construction programme allow, curing can be provided by retention of the formwork. Enhanced curing may be obtained by loosening the formwork, once the concrete has hardened sufficiently to allow this (the next day, for example), to allow water to be introduced into the gap between the formwork and the concrete surface. Care must be taken to avoid the risk of thermal shock from using water that is too cold (see section 3.5.4) and to ensure the concrete is kept moist.

3.5.2 Impermeable coverings

Impermeable material in contact with a concrete surface will prevent loss of moisture. In the form of thin sheet materials such as polythene, it has the advantage of being very lightweight and flexible so it can be applied in most locations and on complicated shape elements. Sheeting on formed surfaces (i.e. cast against formwork) should be in intimate contact with the surface, and securely fixed at the edges to form an effective moisture seal. This can, however, result in a mottled surface appearance and thus should not be used where appearance is a critical factor. Sheeting on fresh unformed (horizontal) surfaces should preferably be suspended above but sealed at the sides and ends to prevent through-passage of air, and to form an effective high relative humidity curing chamber. Thus in this application a small amount of evaporation from the surface is inevitable but should be small provided the enclosure is effectively sealed.

In practice, suspension of sheeting above the fresh concrete surface is often considered impractical and, instead, it is placed directly on the concrete. In this case it is essential that the surface is allowed to stiffen sufficiently to prevent damage from contact with the cover. White sheeting will help control surface temperature of the concrete by reflecting the sun's rays; black will have the opposite effect and should be avoided for outdoor applications in warm conditions. Clear sheeting will have little effect on heat absorption.

The surface of the concrete should be moistened by spraying with water prior to application of the covering to minimize the effect of any initial loss of water into the air space between the concrete and the covering.

3.5.3 Absorptive coverings

Absorptive materials, such as hessian or sand, will keep the surface of the concrete damp for as long as they are in place and are themselves kept wet. Use of such materials may not be appropriate in large sections where it is necessary to insulate the outer surfaces to prevent the risk of cracking through excessive temperature across the section.

Materials should not contain harmful amounts of substances, such as sugars or nitrate fertilizers, that could damage the surface of the concrete. Where appearance is important, these materials should also be free of substances that could stain or discolour the surface.

At the end of the required period of curing, subsequent drying of the concrete will be beneficially slowed if the absorptive material is allowed to dry thoroughly before its removal.

3.5.4 Water addition

Ponding is easily achieved on flat surfaces by building a small bund around the perimeter and keeping the enclosed area flooded. Where spraying or misting is to be used there is a need to consider factors such as continuity of water supply, clogging of nozzles, wind direction and strength, and drainage paths for the surplus water. The risk of thermal shock and excessive temperature differences across sections should also be considered. ACI 308-92 (1992) recommends that the curing water should not be more than about 11°C cooler than the concrete as rapid cooling and contraction of the surface, restrained by the warmer layer below, could result in the development of sufficient stress to cause cracking. The use of free water should also be avoided where the possibility of freezing of the concrete surface exists during curing.

Water used for curing should generally be of the same quality as that used as mix water. In particular, if the concrete is reinforced, or if it contains any other embedded metal, the water should not contain significant levels of chloride that could enter the concrete and increase the risk of premature corrosion of reinforcement. Where the appearance of the concrete surface is important, the water should not contain harmful amounts of any substances that could stain, attack or discolour the surface.

3.5.5 Curing membranes

Curing membranes are hand- or spray-applied sealing compounds applied in liquid form after free water has disappeared from the concrete surface (for horizontal surfaces) or upon removal of formwork (for formed surfaces). Such materials typically include natural and synthetic resins, waxes and solvents that are highly volatile at normal ambient temperatures. They work by greatly reducing the rate of evaporation of water from the surface. When the rate of evaporation exceeds the rate of bleed, it is advisable to wet the surface of the concrete and wait for bleeding to cease before applying a curing membrane. If bleeding continues below the membrane a layer of water may form below the upper layer of matrix which has adhered to the membrane; this layer will be very prone to later damage in service. Map-cracking of the membrane, requiring reapplication, is another possible result.

Opaque compounds have the additional effect of shading concrete; light colours reduce absorption of solar heat. It is difficult to tell by eye whether clear membranes have been applied properly. Curing membranes are generally less effective than properly applied wet curing – but properly applied wet curing is rare so membranes are very worthy of consideration. They have an additional advantage that once they have been applied no further actions or access are required unlike most other curing methods. In some cases, however, it may be necessary to physically remove traces of the curing membrane prior to application of a finish to the concrete surface.

Curing membranes are available in a number of different qualities, expressed in some countries (UK) as ‘efficiency’ and in others as water retention (USA). The greater the efficiency the greater their effect is likely to be. Products with efficiencies as low as 80 per cent are unlikely to provide the required benefit.

Curing efficiency, according to BS 7542, is measured by comparing the water loss from a treated standard mortar sample in a warm, dry atmosphere (approximately 38°C and 32–35 per cent RH for 72 hours) with the water loss from an untreated but otherwise identical control specimen. The ASTM C156 water-retention value is simply the weight loss recorded without reference to a control specimen.

The choice of membrane is especially important when the concrete is to receive further treatment such as plastering or painting. Some compounds, if not physically removed, will prevent subsequently applied materials from adhering. Acrylic-based curing membranes are available that are intended to double up as a permanent surface sealer.

3.6 Protection against vibration

Excessive vibration of immature concrete (from sources such as blasting, piling or very close road or rail traffic) once initial set has taken place can result in decreased strength and reduced bond to reinforcement. Concrete should thus be protected from damaging vibration in the initial period from approximately 4 hours to approximately one day. Information is sparse on the level at which vibration becomes damaging but an indication of risk is given by the peak particle velocity (ppv). Theoretical calculation of ppv is, however, difficult and it may be necessary to use results from actual measurements. The following limits have been suggested (CIRIA, 1992):

Less than one day	5 mm/s
One to seven days	50 mm/s

Well-proportioned concrete mixes are tolerant of low-amplitude, low-frequency vibrations during setting and early strength development. High-slump concrete has been shown, in the laboratory, to be more susceptible to damage traffic-induced vibrations (Highways Agency, 2000). Bleeding of freshly cast concrete can be increased by intermittent vibration.

It may be possible to provide the necessary protection by ensuring the formwork is isolated from the vibration source. In the case of traffic-induced vibration, a minimum distance of 600 mm should be provided between the vehicle wheels and the concrete. A speed limit of 50 mph is also advised (Highways Agency, 2000).

3.7 Is curing always effective?

The commonly encountered method of covering loosely with plastic sheeting can sometimes be ineffective, if not fixed properly, as water still evaporates from the surface of the concrete, condenses on the sheeting and may drain away. Placing the covering directly on the unformed surface will necessitate delay until the concrete has stiffened sufficiently for no damage to the surface. This delay could negate the effect of curing for some purposes, such as avoidance of plastic shrinkage cracking.

An absorbent material which is allowed to dry out can conceivably draw moisture from the surface of the fresh concrete, the effect of which could be worse than applying no curing.

A delay between finishing the concrete surface or striking formwork and applying curing can reduce or eliminate the effectiveness of the curing. Intermittent curing within the first two or three days may have the same effect. Under these conditions, calcium hydroxide that is deposited in the surface capillaries by evaporating pore water becomes carbonated. The resultant calcium carbonate then partially blocks the capillaries making it difficult to get water back into the concrete. Intermittent early curing may also result in surface cracks (ACI 308, 1992). After two or three days of effective continuous curing, intermittent curing may allow continued hydration albeit to a reduced degree.

There is currently no practical on-site test capable of measuring the effectiveness of curing. By definition, the part of the concrete affected by curing is restricted to the outer layers and most *in-situ* techniques rely on properties measured as an average over a depth generally greater than the affected zone.

3.8 How long should curing be applied?

This depends upon:

- the reason for curing (plastic shrinkage, temperature control, strength, durability, etc.)
- the size of the element
- the type of concrete (especially rate of hardening)
- the ambient conditions during curing
- the exposure conditions to be expected after curing
- the requirements of the specification

3.8.1 The effect of cement type

Cements, or combinations, containing fly ash (pfa), blast furnace slag (ggbs), limestone filler (>5 per cent), or condensed silica fume react more slowly than plain Portland cement, particularly in cold weather. Concretes containing blended cements should therefore be cured thoroughly and for a longer period than for PC concrete, particularly if the potential durability benefits are to be obtained in the near-surface and cover zone. Concrete containing condensed silica fume or metakaolin exhibits only minimal bleeding and thus requires early protection to prevent plastic shrinkage cracking.

3.9 When is curing of *particular* importance?

The following circumstances warrant particular consideration of curing needs:

- horizontal surfaces
- dry, hot, windy conditions (one or more of these)
- wear-resistant floors
- high-strength concrete (initial curing is especially important)

This list is not exhaustive and curing may still be of great importance in other conditions. Abrasion resistance is particularly dependent on good curing but also relies upon other factors including materials and surface finishing.

3.10 Effect of temperature

The hardening of concrete is a chemical reaction – the rate of this reaction increases with temperature but so does the rate of evaporation from an exposed concrete surface. The rate of reaction at 35°C is about twice that at 20°C which is, in itself, about twice that at 10°C.

The ultimate strength of concrete cured at low temperature (e.g. in winter) is generally greater than that of concrete cured at a higher temperature (e.g. in summer); but extremes of temperature generally have a negative effect. The slow rate of reaction at low temperatures means the concrete must be cured for a longer period to achieve the desired degree of reaction. The fast rate of reaction at high temperatures gives relatively high early strengths but the long-term strength and durability are generally reduced.

The optimum temperature required to produce the maximum 28-day strength, based on small laboratory specimens, is said to be approximately 13°C (Neville and Brooks, 1987) and ambient temperatures of 15–25°C are generally considered to be most suitable for concreting operations.

Hydration will proceed, to some extent, at temperatures down to as low as –10°C. Nevertheless, little strength will develop below 0°C, and below 5°C the early strength development is greatly retarded (ACI 308, 1992). Even in the temperature range 5–10°C conditions are unfavourable for the development of early strength. These effects will be most prevalent in thin sections, in the near surface of larger sections, and in concrete made with slow hydrating cements. The bulk strength of larger sections will be less affected because, generally, the internal temperature will be elevated by the heat of hydration.

Concrete should not be allowed to freeze before it has gained sufficient strength to resist damage. According to ACI 308 (1992) this strength is approximately 3.5 MPa. Air-entrained concrete should not be allowed to undergo any freeze–thaw cycles until it has reached a strength of approximately 25 MPa. Non air-entrained concrete should, of course, never be allowed to undergo freezing and thawing while saturated.

The temperature of concrete during curing depends on:

- the dimensions of the element
- the weather (ambient conditions)
- cement type

- cement content
- admixtures (accelerators, retarders)
- the fresh concrete temperature
- formwork type/insulation
- formwork stripping time

Mostly, temperature control of *in-situ* concrete during hardening is only attempted at temperature extremes where, for example, there is:

- a risk of freezing
- a risk of an excessive peak temperature or an excessive temperature difference across the section

The peak temperature in a section should generally be kept below about 65–70°C to minimize the effect on compressive strength and to minimize the risk of delayed ettringite formation (DEF). Measures to reduce peak temperatures are beyond the scope of normal curing techniques and may include cooling of the fresh concrete by various means or cooling of the placed concrete by means of cooling pipes within the section.

3.11 What happens if concrete is not cured properly?

Concrete allowed to freeze before a certain minimum degree of hardening has been achieved will be permanently damaged by the disruption from the expansion of the water within the concrete as it freezes. This will result in irretrievable strength loss. Excessive evaporation from an exposed horizontal surface within the first approximately 24 hours after casting will result in plastic shrinkage cracking and a weak, dusty surface. An excessive temperature difference through the cross-section of an element will result in early thermal cracking due to restraint to contraction of the cooling outer layers from the warmer inner concrete. Inadequate curing will result in the properties of the surface layer of concrete, up to 30–50 mm, not meeting the intentions of the designer in terms of durability, strength and abrasion resistance.

3.12 The effect of curing on strength

The effect of curing on strength development is limited to the near-surface of concrete so its effect on strength will depend on the element size and type of loading that will be applied. The effect on large elements loaded in compression will be much less than on slender elements loaded in flexure. It is unlikely that the structural capacity of most elements would be significantly reduced by poor curing. Attempts to assess the effect of poor curing on strength development on small specimens such as cubes or cylinders are likely to give pessimistic results (Marsh and Ali, 1994).

The effect of reduced strength in the surface zone on the structural performance can be calculated by making assumptions about the actual reduction in strength and the depth of the effect. Using a strength reduction of 25 per cent in the outer 25 mm predicts the following overall effects (CIRIA, 1997):

• flexural strength (sagging)	93 per cent
• shear strength	98 per cent
• bond stress	no change

3.13 The maturity concept for estimation of required curing duration

Required curing duration is sometimes expressed in terms of the maturity of the concrete. The maturity concept is used to predict the rate of strength development at different temperatures; it is mostly used for the determination of formwork stripping time and for structural considerations such as time at loading. Maturity allows the strength of a concrete of known temperature-time history to be predicted from laboratory specimens cured under standard conditions.

The maturity is the sum of the product of temperature (above a datum level, usually –11°C, the temperature at which hydration is said to cease) and the time over which the temperature prevails:

$$M = \sum (T \cdot \Delta t)\ [°\mathrm{C.h}]$$

Equal maturity should mean equal strength; but the relationship between maturity and strength depends on the actual cement type and strength class used, the water/cement ratio and whether any significant water loss occurs during curing. The maturity relationship must thus be individually developed for each different concrete using laboratory specimens stored at constant temperature but tested at various ages to give the required maturity range. Later strength development such as can occur with concrete containing pozzolanic or latent hydraulic materials, e.g. fly ash or blast furnace slag, is not picked up by this technique.

The maturity concept cannot, at the current state of knowledge, be directly related to durability aspects rather than strength. Thus where curing is required for properties such as abrasion resistance, permeability, freeze–thaw resistance, etc. it may be necessary to extend curing beyond the period predicted by maturity at which a certain strength develops.

3.14 Some international curing specifications

Specifications for minimum curing periods vary considerably from country to country as shown by the following brief list of current and historical specifications:

UK (BS 8110)	0–10 days depending on cement type, ambient conditions (sun, wind and relative humidity) and average surface temperature of concrete. (See Table 3.1)
UK (DTp, 1986)	2–7 days depending on cement type and ambient conditions (hot weather and drying winds), or use equivalent maturity (1000–3500°C.h). If average surface temperature falls below 10°C curing must be extended to achieve required maturity. (See Table 3.2)

Table 3.1 BS 8110: 1997 curing recommendations

Minimum periods of curing and protection

Type of cement	Ambient conditions after casting	Average surface temperature of concrete	
		5°C to 10°C	t°C (any temperature between 10°C and 25°C)
PC 42.5 or PC 52.5 to BS12 SRPC 42.5 to BS 4027		Days	Days
	Average	4	$60/(t + 10)$
	Poor	6	$80/(t + 10)$
All cements indicated in Table 1 of BS 5328: Part 1: 1997 except for PC 42.5 or PC 52.5 to BS 12, SRPC to BS 4027 and supersulfated cement	Average	6	$80/(t + 10)$
	Poor	10	$140/(t + 10)$
All	Good	No special requirements	

NOTE 1. Abbreviations for the type of cement used are as follows:
PC 42.5: Portland cement (class 42.5) (see BS 12);
PC 52.5: Portland cement (class 52.5) (see BS 12);
SRPC 42.5: Sulfate-resisting Portland cement (class 42.5) (see BS 4027).

NOTE 2. Ambient conditions after casting are as follows:
good: damp and protected (relative humidity greater than 80 per cent; protected from sun and wind
average: intermediate between good and poor;
poor: dry or unprotected (relative humidity less than 50 per cent; not protected from sun and wind).

Table 3.2 UK Department of Transport specification (1986) for curing

Normal curing periods

Conditions under which concrete is maturing	Minimum periods of protection for different types of cement					
	Number of days (where the average surface temperature of the concrete exceeds 10°C during the whole period)			Equivalent maturity (degree hours) calculated as the age of the concrete in hours multiplied by the number of degrees Celsius by which the average surface temperature of the concrete exceeds –10°C		
	Other	SRPC	OPC or RHPC	Other	SRPC	OPC or RHPC
1. Hot weather or drying winds	7	4	3	3500	2000	1500
2. Conditions not covered by 1	4	3	2	2000	1500	1000

Sweden (BBK 79)	Damp cure until reach 45 per cent of design strength – or can use plastic sheet or membrane if concrete grade is raised above that otherwise required.
USA (ACI 308)	Lesser of 7 days or 70 per cent of specified strength for pavements, slabs, structures and buildings; 2 weeks for unreinforced massive sections; 3 weeks for massive sections containing pozzolan.

Italy	3 days of moist curing (not in cold weather).
Germany (DIN 1045)	'Sufficiently long period', generally 7 days will suffice.
Germany (DAfStb)	1–5 days depending on rate of hardening (w/c and cement strength class) and environmental conditions (sun, wind and relative humidity); reduce values if 50 per cent of characteristic strength obtained; double values if concrete surface temperature below 10°C.
France (Fascicule 65-A)	0–10 days depending on rate of strength development (cement type and strength class), ambient conditions (sun, wind and relative humidity) and average midday shade temperature.

3.15 Some food for thought

There is some research evidence to show that water-cured specimens perform worse than otherwise identical air-stored specimens in accelerated chloride ingress and sulfate attack tests. The effect of curing on durability-related properties can prove difficult to detect reliably due to the inherent variability of concrete and the insensitivity of the currently available measurement techniques (air permeability, etc.) to the surface effects produced by curing.

Much of the research on the effect of curing on properties of concrete has been performed on small specimens stored for various periods under water. Results from tests on such concrete cannot be related to *in-situ* concrete cured by normal techniques.

In climates with frequent rainfall, such as in the UK and much of mid- and northern Europe, long-term carbonation depths in outdoor concrete are not much affected by initial curing. The subsequent exposure to rain has a greater influence than the initial curing. In climates with prolonged dry periods, however, and for outdoor elements sheltered from rain, this is unlikely to be true.

There is very little direct evidence from structures on the effect of curing on properties of concrete other than in horizontal surfaces.

The influence of practical on-site curing on durability related performance criteria is summarized in Table 3.3. This is based on a review (CIRIA, 1997) of published information relevant to actual structures rather than small specimens under laboratory conditions. The CIRIA study concluded that 'research findings challenge, or fail to support, established concepts regarding the beneficial effects of practical curing for *some* durability related performance criteria.

3.16 Summary and conclusions

Good curing is essential where it is necessary to obtain the optimum performance from the concrete in the surface zone.

The depth of the zone affected by curing depends on several factors but can be up to 20 mm in temperate climatic conditions, and up to 50 mm in more extreme arid conditions.

Curing methods that employ free water are generally more effective than those that rely on retention of water. They can, however, be impractical in some circumstances.

Table 3.3 The influence of practical on-site curing on durability-related performance criteria (after CIRIA, 1997)

Performance criterion	Demonstrable influence of curing?	Influence on durability	Comments
Carbonation resistance	Yes	Increase	No significant further improvement if curing exceeds 7 days
Abrasion resistance	Yes	Increase	Finishing of surface is likely to be more significant than curing
Sulfate resistance	Yes	Decrease	Initial early air exposure improves performance
Other chemical resistance	Yes	None	Insufficient data. Early air exposure may have some beneficial effects for some chemical effects
Plastic shrinkage	Yes	Increase	Significant factor reducing risk of cracking
Structural performance	Yes	None	Insufficient data. Unlikely to have significant effect
Freeze–thaw resistance	No	None	Insufficient data. Unlikely to improve long-term performance
Chloride penetration	No	None	Insufficient data. Unlikely to improve long-term performance
Alkali silica reaction	No	None	Insufficient data. Unlikely to have significant effect
Impact resistance	No	None	No data. But linked to abrasion resistance
Weathering	No	None	Insufficient data. Theoretically should improve performance
Autogenous deformation	No	None	Not significant factor in reducing risk of cracking
Drying shrinkage	No	None	With exception of thin exposed slabs or topping. Not direct significant factor in reducing risk of cracking

Curing will not significantly affect the structural capacity of most members.

Concretes made with slower hydrating cements require increased attention to curing.

When specifying or choosing a curing regime (method and duration) consideration should be given to the specific reasons for curing.

Curing is of particular importance for dry, hot or windy conditions, horizontal surfaces, wear-resistant floors and high-strength concrete.

There is a lack of real data on the effect of curing on the in-service performance of concrete structures.

The influence of curing is most significant on horizontal, unformed surfaces. The principal properties affected are abrasion resistance and avoidance of plastic shrinkage cracking.

The long-term relative humidity profile in concrete is independent of curing. To be effective, curing must prevent the relative humidity in the concrete from falling below 95 per cent.

But remember:

GOOD CURING OF BAD CONCRETE DOES NOT GIVE GOOD CONCRETE

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4

Concrete properties: setting and hardening

Tom Harrison

4.1 Strength development

4.1.1 Learning objectives

1. Describe the mechanism of strength development of concrete.
2. Compare the rates of strength development for concretes made with different materials.
3. Describe the effects of sub-normal and of elevated temperatures on the rate of strength development for different types of concrete.
4. Describe the effects of curing conditions on the rate of strength development of different concretes.
5. Describe methods for monitoring the rate of strength development of concrete in the laboratory and on-site.

4.1.2 Background

The strength of concrete is not a precise term describing a property of concrete, but the ultimate load sustained under defined loading conditions. The strength of concrete has to be qualified with terms such as tensile strength, flexural strength or compressive strength. Even then further qualification is needed. For example, with compressive strength, it is necessary to define the shape of specimen, e.g. 150 mm diameter × 300 mm cylinder, 100 mm cube etc., and if special loading conditions apply, e.g. tri-axial loading. The significant differences between these measures of compressive strength are due not to differences in the response of the concrete but mainly to differences in the lateral restraint provided by the machine. With cubes, the lateral restraint by the machine platens is a significant factor in resisting failure and consequently the resulting 'strength' is higher than, say, a 150 mmϕ × 300 mm cylinder. The European concrete standard, EN 206-1(2000), put this difference at about 20 per cent for normal weight concrete in their dual classification of strength class. EN 206-1 uses the strength of a 150 mmϕ × 300 mm cylinder as the first (and reference) classification followed by the strength of 150 mm cubes, e.g. C40/50. The strength class for lightweight concrete has a different relationship between these two measures of strength, e.g. LC40/44.

Conditions where the machine platens provide no lateral restraint can be achieved by a number of techniques, e.g. loading via two layers of plastic film which have between them a thin layer of grease. These loading conditions give even lower strengths (Table 4.1) and the specimens fail with tensile cracks parallel to the direction of loading (Figure 4.1).

Table 4.1 Compressive strength with and without lateral restraint by the machine platens (data from Hughes and Bahramian, 1967)

Normally tested		With platen restraint removed	
Cubes[1]	Prisms[2]	Cubes[1]	Prisms[2]
59	41	37	37
100	68	68	66
41	34	34	31
90	66	64	61

Notes:
[1] 102 mm cubes.
[2] 244 × 102 mm prisms.

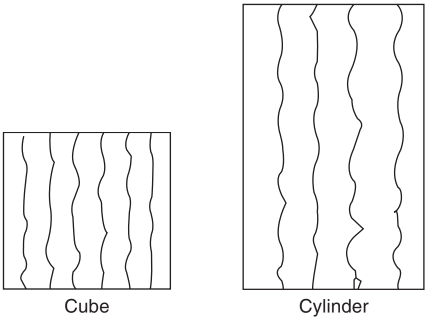


Figure 4.1 Typical failure patterns for a cube and cylinder without lateral restraint from the machine platens.

Various techniques have been used to study the failure mechanism of concrete including:

- stress–strain curves (change in initial modulus)
- acoustic emission
- volumetric strain
- energy method
- pulse velocity.

Figure 4.2 shows a comparison of different methods under uniaxial compression.

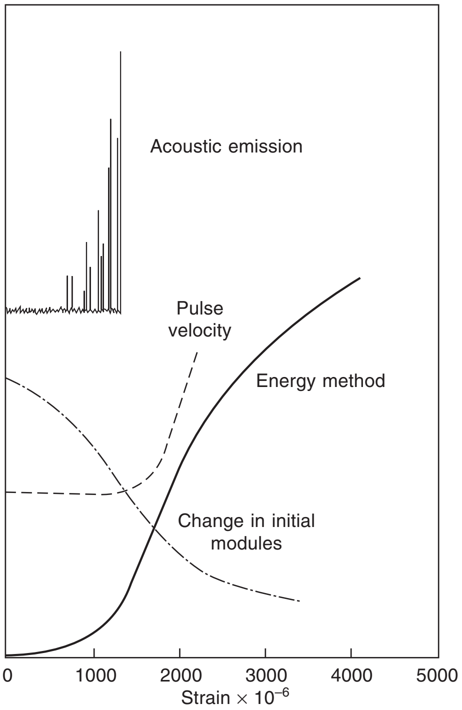


Figure 4.2 Methods of detecting damage (Spooner and Dougall, 1975).

Research by Spooner and Dougill (1975) revealed that the failure of concrete was progressive. During loading microcracking occurs. When the load is released and re-applied, the modulus of elasticity has reduced if cracking has taken place and no further damage occurs until the peak load attained previously is reached. From this point onwards further microcracking occurs. It was noted that the envelope drawn around curves for the cyclic loading fell within the envelope obtained from a specimen subject to monotonically increasing strain, i.e. a single load cycle (Figure 4.3). This research tended to disprove previous theories that postulated that concrete behaved in a linear-elastic way up to some discontinuity stress level (about one third the ultimate load). However, it should be noted that in the Spooner and Dougill model, the amount of damage (microcracking) that occurs when the loading is restricted to about one third the ultimate load is very small. Therefore loading to about half the ultimate load provides a safe basis for design for most structures. At higher levels of loading, many loading cycles will tend to increase the amount of microcracking and eventually may result in failure by fatigue.

4.1.3 Mechanism of strength development

Concrete develops its strength by hydration of the cement and addition to form a complex series of hydrates. The initial hydration fixes the cement particles into a weak structure

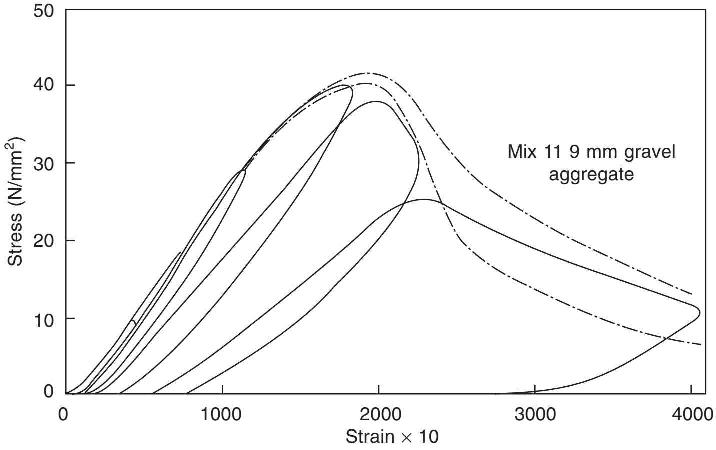


Figure 4.3 Effect of loading cycles on the stress–strain curve (Spooner and Dougall, 1975).

surrounded by a water-filled space. The higher the initial water content, the further will be the average spacing between the cement grains. Provided the concrete is not allowed to dry out, the cement grains will continue to hydrate with time and fill the space between the cement grains with a mixture of hydrates and pores. The further the initial spacing between the cement grains, i.e. the higher the water/cement ratio, the more pores per unit volume and the weaker the resulting concrete. Where the initial water/cement ratio is high, the resulting pore structure within the hydrates is interconnected and the resulting concrete has low strength, high penetrability and low durability.

Hydration will continue for many years provided:

(a) there is water available for hydration;
(b) there is cement/additions available to react.

The section on cement chemistry explains how different cement compounds contribute to strength development, but we cannot quantify the rate of strength development of a concrete based on the cement composition and the water/cement ratio. One reason for this is that cements do not comprise pure compounds and the actual hydration is far more complex than those of single pure compounds. However, as a generalization, cements that are high in tricalcium silicates gain strength rapidly and have relatively low long-term strength development whilst cements high in dicalcium silicates gain strength relatively slowly but have high long-term strength gain. In practice this long-term strength gain will only occur in conditions where the concrete retains or gains sufficient water for hydration to continue. Once dried so that the internal relative humidity falls below 95 per cent (Killoh *et al.*, 1989), further hydration effectively stops. However, if the concrete is re-wetted, hydration will start again.

Various models have been developed to link strength to the porosity of the hydrates. Abrams' 'law' states that the strength of concrete is inversely proportional to the w/c ratio:

$$\text{Strength} = \frac{K_1}{K_2^{\text{w/c}}}$$

where K_1 and K_2 are empirical constants. This is a special case of the Feret formula:

$$\text{Strength} = K\left(\frac{c}{c + e + a}\right)$$

where c, e, and a are the absolute volumes of cement, water and air respectively and K is a constant.

In essence, strength is related to the total volume of voids and the most significant factor in this is the w/c ratio. At a more fundamental level, this can be expressed as a function of the gel/space ratio (x), which is the ratio of the volume of the hydrated cement paste to the sum of the volumes of the hydrated cement and the capillary voids. The data from Powers (1958) gives;

$$\text{Strength} = 234x^3 \text{ MN/m}^2$$

and this is independent of the age of the concrete and the mix proportions. This equation is valid for many cements, but the values of the numerical coefficients vary a little depending on the intrinsic strength of the gel.

The strength of concrete depends primarily on the physical structure of the gel and the chemical composition of the gel has a secondary effect that becomes minor at later ages.

Such models that focus only on the cement paste, ignore the effects of the aggregate characteristics on strength which can be significant. It is not prudent to rely on theoretical models to predict the strength of concrete. The actual rate and magnitude of concrete strength development depends on:

1. the basis for comparison, see section 4.1.4;
2. the cement type, class and source;
3. the type, source and amount of addition;
4. the water/cement ratio or water/binder ratio;
5. type of aggregate;
6. the consistence (workability);
7. the temperature and temperature history.

At some point, a set of materials will give a ceiling strength. Normally it is the cement paste that fails, but with high-strength concrete, failure may be initiated by failure of the aggregate. This is often due to the increase in cement content leading to a proportional increase in voidage (water demand) and the w/c ratio remaining the same. Hence, the concrete does not increase in strength. In other cases, the ceiling strength is the result of failure of the aggregate or the aggregate/cement paste bond.

4.1.4 Comparison of strength development

Depending on the basis for comparison, different rates and magnitude of strength development will be indicated. Some frequently used approaches are:

(a) equal water/cement ratio and equal cement content;
(b) equal cement content and consistence;
(c) equal 28-day strength and consistence;
(d) full compliance with a standard and equal consistence;
(e) equal long-term strength, e.g. 90 days, and consistence.

A basis for comparison relevant to the application should be selected.

Cement (combination) type

Cements of the same type and class do not give the same rate of strength development even under standard conditions (Figure 4.4). Cements or combinations containing slag or pfa gain strength more slowly but have higher ultimate strengths provided there is sufficient water and cement for further hydration (Figures 4.5–4.9).

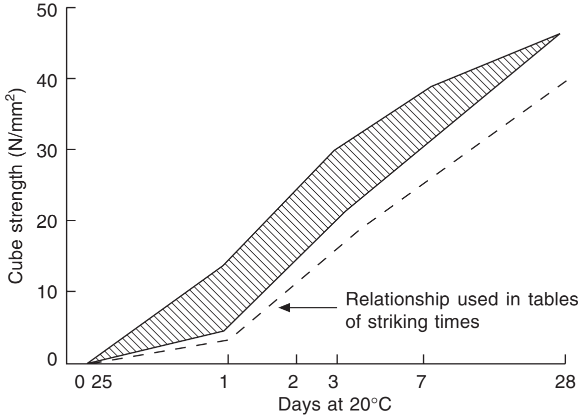
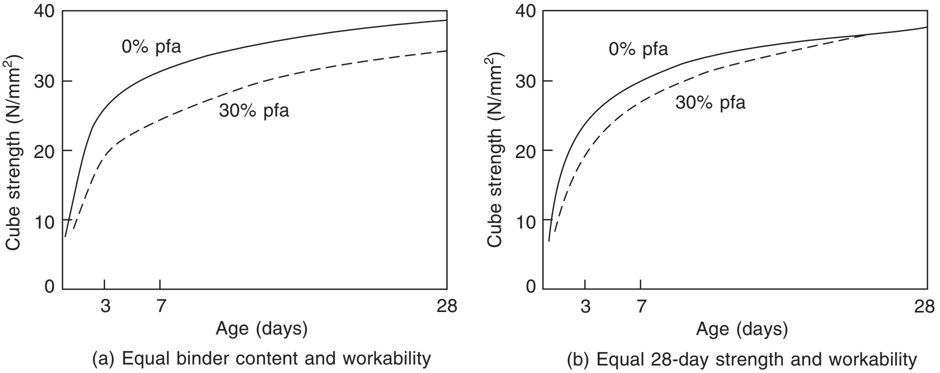


Figure 4.4 Typical envelope of strength for PC-42.5 concretes (Harrison, 1995).



(a) Equal binder content and workability

(b) Equal 28-day strength and workability

Figure 4.5 Early strength gain of PPFAC concretes at 20°C (Harrison and Spooner, 1986).

Consistence

Consistence is the word used in EN 206-1 for what was traditionally known as ‘workability’. Changes in the consistence have a relatively small effect on strength development in comparison with some of the other factors (Figure 4.9).

Aggregate type

Aggregate type has an influence on the strength of concrete but little effect on the proportional rate of strength gain. Table 4.2 shows the effect of some aggregate types on cube strength.

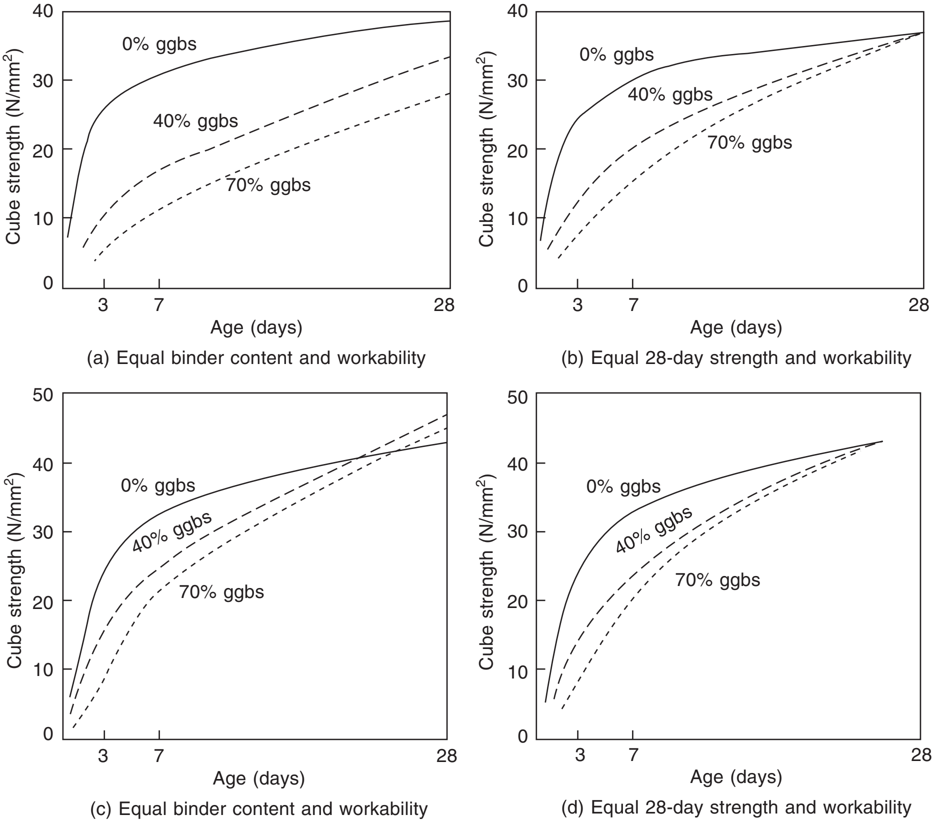


Figure 4.6 Early strength gain of ggbs concretes at 20°C (Harrison and Spooner, 1986).

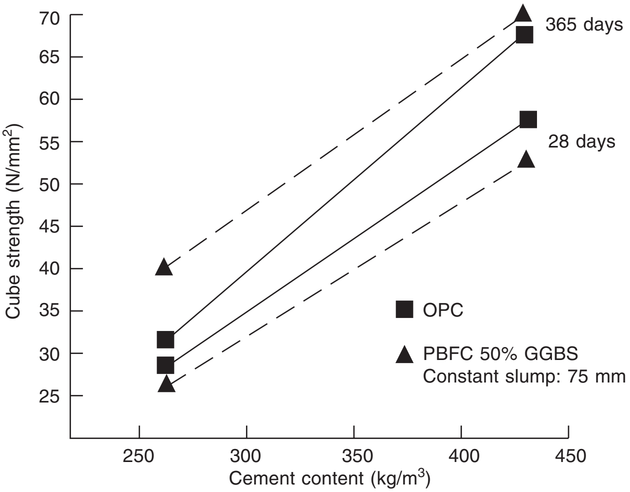


Figure 4.7 Long-term development in relation to cement content (Concrete Society, 1991).

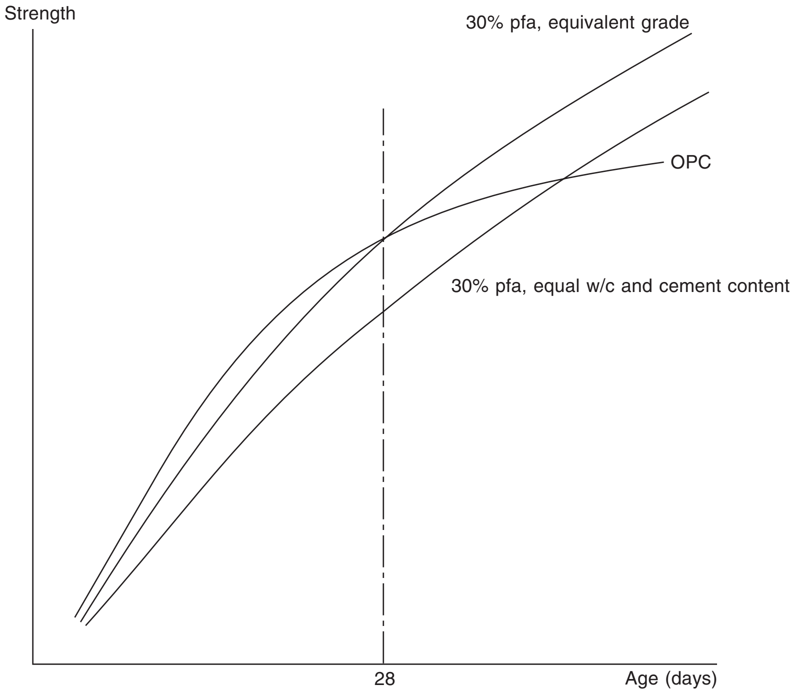


Figure 4.8 Strength development of standard cubes with and without pfa (Concrete Society, 1991).

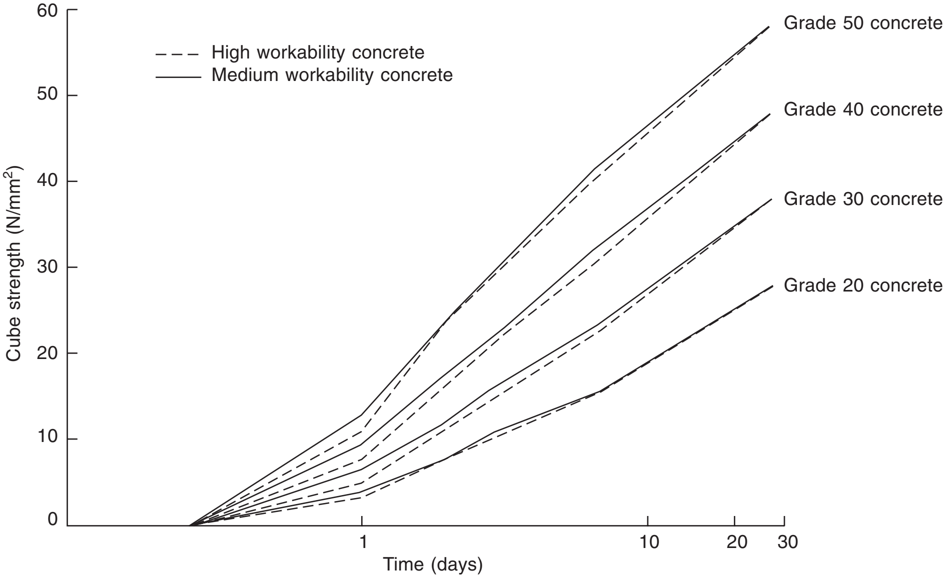


Figure 4.9 Strength gain at 20°C of high and medium workability CEM I-42.5 concretes (Harrison, 1995).

Equal w/c ratio with varying cement contents

The technical literature gives conflicting results. Some papers show at equal w/c ratio no effect of cement content whilst most show a reduction in strength as the cement content increases (Figures 4.10 and 4.11).

Table 4.2 Effect of coarse aggregate type on the 28-day cube strength of concrete (data from Dhir *et al.*, 2000)

Aggregate type	28-day strength Free w/c ratio	
	0.65	0.50
Granite	38.5	54.0
Carboniferous limestone	37.0	54.0
Natural gravel	32.5	53.5
Jurassic oolitic limestone	30.0	43.0
Dolomitic limestone	34.0	43.5
Lightweight (sintered pfa)	28.5	37.0

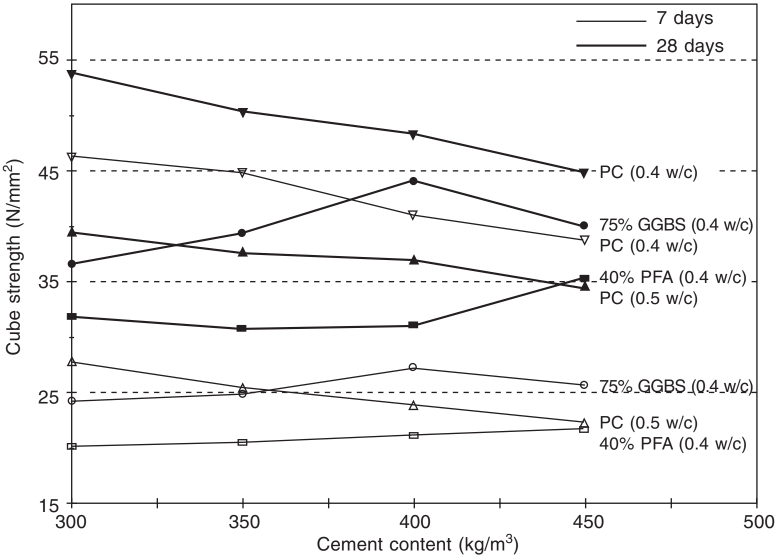


Figure 4.10 Cube strength as a function of cement type and content (Buenfeld and Okundi, 1998).

4.1.5 Temperature and temperature history

Low temperatures decrease the early strength development whilst high temperatures increase the early strength development (Figure 4.12). The temperature at casting has an effect on 28-day strength. A few hours at a low/high temperature prior to standard curing increases/decreases the 28-day strength. Pitcher (1976) showed that only short periods were needed at high temperatures to have a detrimental effect on 28-day cube strength (Figure 4.13). He also found that the effect of low temperature was less pronounced.

The temperature cycle that a large pour undergoes increases the *in-situ* strength relative to standard specimens for the first few days, but in the long term the *in-situ* strength is less than that of standard cubes. Harrison and Habgood, reported in Harrison and Spooner (1986) investigated these effects by subjecting sealed cubical specimens to the temperature cycle they would have experienced in a large pour. Figures 4.14 and 4.15 show some of the results of this investigation. All Portland cement concretes showed a reduction in the 28-day strength compared to standard specimens, but all these concretes continued to

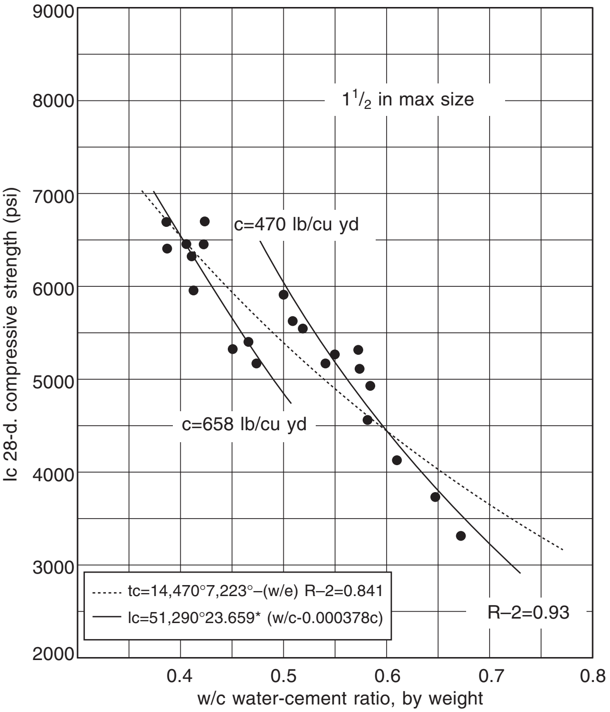


Figure 4.11 Effect of cement content on the concrete strength versus water/cement relationship (Popovics, 1990).

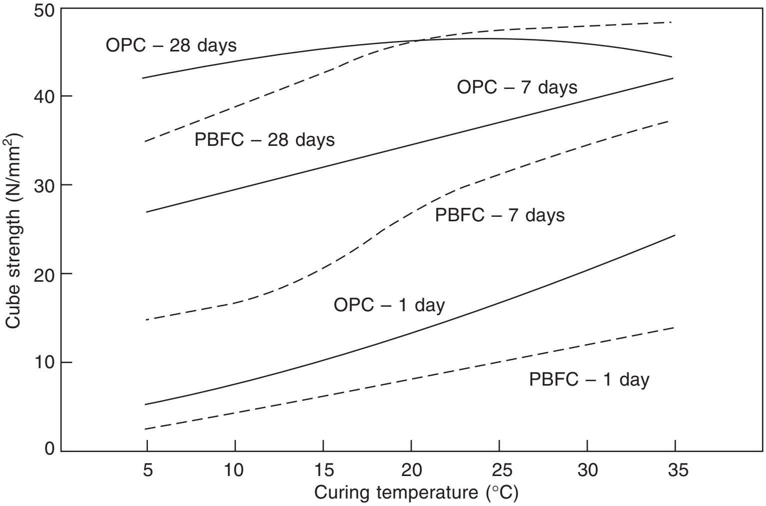


Figure 4.12 Effect of curing temperature upon the strength gain of a PC and PBFC concrete (Harrison and Spooner, 1986).

gain in strength (Figure 4.14). The effect on concrete containing a pozzolanic material is slightly different. At 28 days the *in-situ* strength is enhanced relative to standard specimens due to the acceleration of the pozzolanic reaction. At about 3 months the strength development

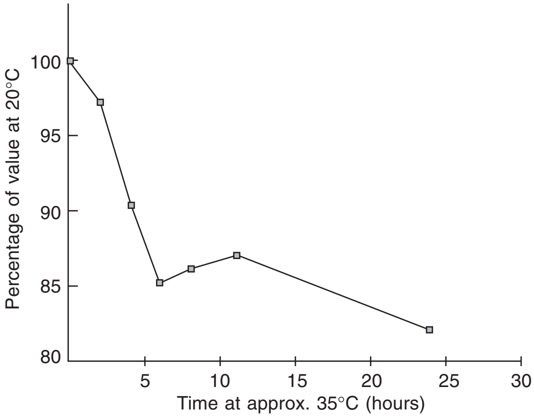


Figure 4.13 Effect of time at 35°C on 28-day strength (Pitcher, 1976).

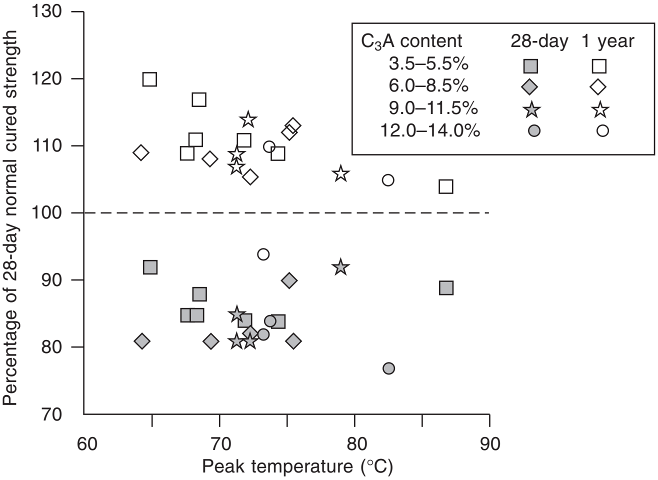


Figure 4.14 Portland cements after a 'large pour' cycle (Harrison, unpublished data).

curves cross so that at 1 year the *in-situ* strength of the Portland cement and pfa concretes are not significantly different (Figure 4.15).

High temperatures at early ages reduce the ultimate strength of all cement types.

4.1.6 Curing conditions

The hydration of cement requires water and reduced water will reduce the hydration. In dry conditions a point will be reached where hydration and strength gain ceases. The effects of air curing are illustrated in Figure 4.16.

Care is needed when interpreting data on the effects of curing on strength as testing concrete in the dry state increases strength compared to testing wet specimens. For example, a recent study (MAT-CT-94-0043) compared the BSI and DIN curing systems for concrete. In BS 1881, specimens are cured for 27 days in water whilst the DIN system

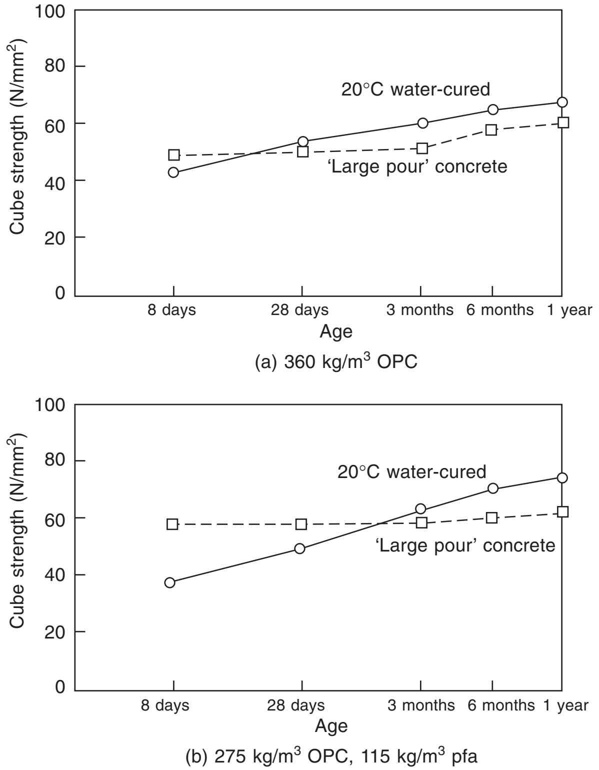


Figure 4.15 Example of the development of cube strength for equivalent PC and PC/pfa concrete (Harrison & Spooner, 1986).

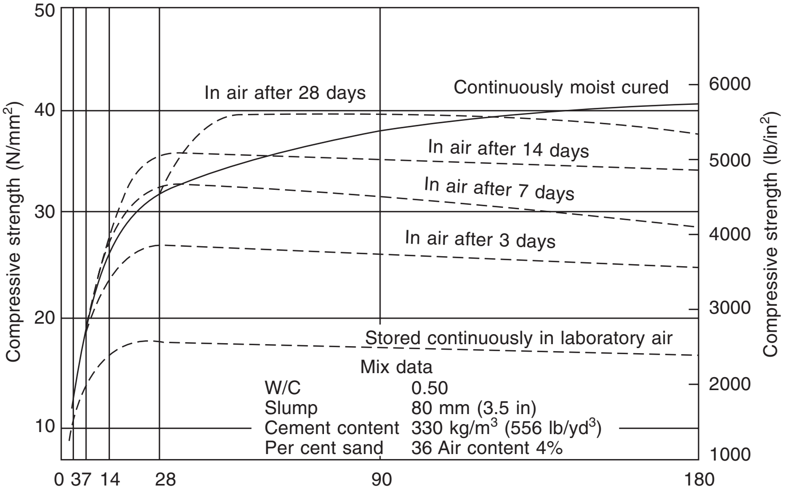


Figure 4.16 Influence of moist curing on concrete with a w/c ratio of 0.50 (Price, 1953).

is 6 days in water and 21 days in air. In both cases, 1 day has been in the moulds prior to curing. The data showed that with the DIN system 100 mm cubes were, on average, 8 per cent higher and 150 mm cubes were 14 per cent higher. This shows that the dominant effect is the moisture condition on testing and not the lack of hydration, which would only have affected the surface zone in the 21 days of air exposure.

4.1.7 Monitoring the rate of strength development

The strength of standard cubes will not be the same as the strength in the structure for the following reasons:

- differences in maturity;
- differences in compaction and curing;
- water and cement migration within the cast element.

A large section will have significant temperature gradients across the section and the temperature history and maturity will vary from point to point. Water migration within an element will cause the upper surface zone to be weaker than the lower surface zone. Even in suspended slabs there will be significant differences. For example, pull-out readings on the upper surface can be about 10 per cent less than those taken at the soffit. In most sections, the exceptions being those heated from outside, the points of lowest maturity and strength will be those near the upper surface. In simply supported bending, the upper surface zones will also be the zones of maximum compressive stress and therefore if the strength in these zones is adequate for, say, formwork striking, the strength in the rest of the section can be deemed to be adequate.

Because of the differences between the structure and standard test specimens, it is very difficult to determine from measurements on the structure whether the quality of concrete supplied to the site was as specified. It is possible to determine whether a structure is adequately strong for the intended loading.

The main methods of assessing the rate of *in-situ* strength gain are:

(a) cubes cured alongside the structure
(b) tables of formwork striking times
(c) temperature matching curing bath
(d) measuring maturity, e.g. COMA probe
(e) penetration tests, e.g. Windsor probe
(f) break-off tests, e.g. TNS-test
(g) pull-out tests, e.g. Lok-test
(h) rebound-hammer
(i) coring.

Of these methods, the first two are most widely used in the UK for formwork striking, but they all have their strengths and weaknesses as shown in Table 4.3. Recent publications (BCA, 2000) are encouraging the use of the LOK-test for the assessment of formwork striking times because of the benefits it gives to process efficiency. Where maximum efficiency in the use of formwork is required, the use of the LOK-test is recommended.

A laboratory may also have other sophisticated indirect techniques available for assessing strength development, e.g. changes in porosity, but unless the use of these techniques is needed for other purposes, the simpler, more direct methods should be used.

Table 4.3 Methods of assessing formwork striking times (strength of concrete)

Method	Advantages	Disadvantages
Cubes cured alongside	1 Samples concrete supplied 2 Good for thin slabs or walls cast in steel formwork	1 Conservative with large or well-insulated sections 2 Requires cubes to be made and tested
Tables	1 Simple to apply but they must take into account the grade of concrete (see sample from CIRIA Report 136)	1 Assume concrete is as specified 2 Can be conservative as they assume the concrete just achieves its grade and has the lower bound strength gain
Temperature-matched curing bath	1 Samples concrete supplied 2 Produces test specimens of equal maturity to the structure 3 Cube test, therefore no problem with conversion of test results	1 Requires 110 V power supply 2 Requires cubes to be made and tested 3 Each pour needs its own TMCB
Measuring maturity with thermocouples	1 Can also be used to measure the temperature gradients across the section	1 Assumes concrete is as specified 2 Need to establish and agree strength/maturity relationship 3 Frequent temperature readings are needed unless automatic logging system is used
Measuring maturity with meter or COMA probe	1 Relatively simple to use 2 Maturity can be read as many times as necessary	1 Assumes concrete is as specified 2 Need to establish and agree strength/maturity relationship
Penetration tests, e.g. Windsor probe	1 Directly test structure 2 Rapid 3 Can have a ‘second’ go	1 Need to establish and agree strength/penetration relationship. This is not easy with normal-sized cubes and low strengths 2 Wide scatter at low strengths 3 Need to repair surface
Break-off tests, e.g. TNS test	1 Directly tests structure	1 Requires inserts 2 Need to repair surface 3 Scatter of results so 5–6 inserts needed 4 Need to establish strength/break-off relationship
Pull-out tests, e.g. Lok-test	1 Directly tests structure 2 An excellent correlation between Lok-test value and cube strength is claimed	1 Until confidence is developed, need to establish strength/pull-out relationship 2 Requires inserts (5–6 per section) 3 Need to repair surface

Notes:

1 Tests directly on the structure can have lower ‘cube strengths’ than tests on test specimens because of assumed differences between test specimens and structure (γ_m).

2 An additional factor is needed to ensure a safe relationship between instrument reading and cube strength, see CIRIA Report 136 (Harrison, 1995).

Cubes cured alongside

Specimens, cubes or cylinders, are made and compacted in the normal way and then placed immediately alongside or on the slab of the element being cast. They are covered with plastic sheeting or similar material to prevent premature drying. Where the slab is insulated, the specimens are placed under the insulation. The specimens may be left in their moulds until tested. At appropriate times, one or more specimens are taken for strength testing. They are normally tested in the as-received condition and not pre-soaked.

Due to differences in size between the cubes and the structure there can be significant differences in the maturity and strength of the cube and the structure. The cube will tend to underestimate the strength in the structure, i.e. give a safe answer because the cube tends to follow the ambient temperature and not the elevated temperature in the structure caused by the retention of the heat of hydration. This generalization may not be true in the rare case of the *in-situ* concrete being forced cooled. With thin sections this is a reasonable and safe system for estimating *in-situ* strength. With large sections, this is not a very effective system as it significantly underestimates the *in-situ* strength.

Tables of formwork striking times

For a given set of conditions and a known strength/maturity relationship, it is possible to produce tables of when the concrete should achieve certain strengths. Table 4.4 gives a sample page from CIRIA Report 136 (Harrison, 1995). These tables were based on the assumption that the concrete just achieves its characteristic strength at 28 days. The assumed rates of strength development are defined in CIRIA Report 136 and the tables may be applied to any concrete that achieves these rates of strength gain and for which the Sadgrove equation is a reasonable maturity law.

The CIRIA Report 136 tables are conservative as they are based on the lowest probable rate of strength development for a Portland cement and the assumption that the concrete only achieves the specified characteristic strength. In at least 95 per cent of cases, the strength would be above this characteristic value. The rates of strength development at 20°C have been given in the Report to allow the tables to be applied to other cement types that achieve these rates of strength development. Whilst they are the simplest of all the systems for assessing strength, they can be uneconomic and they assume that the concrete supplied is that specified.

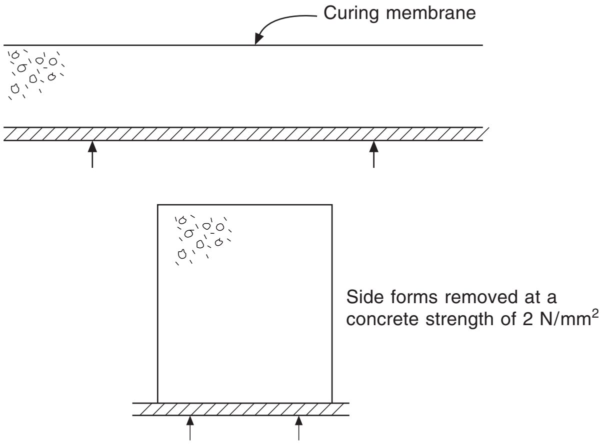
Temperature-matched curing bath

A typical temperature-matched curing bath is shown in Figure 4.17. It comprises a temperature sensor that is placed in the freshly cast element, a water-filled tank with stirrer, heater and temperature sensor and a control system. When the control system detects a difference in temperature between the sensor in the cast element and the water in the tank, it heats the water so that the temperatures are the same. In the UK it has not been necessary to include a cooling system as well as a heating system. With this system, any cubes stored in the tank will have identical maturity to the point in the cast element where the sensor is located. In this system, cubes with the upper surface covered with a steel or glass plate are stored in the tank and at pre-selected times one or more specimens are taken for testing in the normal way.

This system requires a power supply and security for the equipment. The equipment can only be used for one element at a time. As the cubes in the tank have the same temperature history as the point in the structure where the sensor is located, when tested

Table 4.4 Sample page from CIRIA Report 136

Minimum striking times (days) for slab and beam soffits with an uninsulated top surface



Conditions of use

1 The non-formed surfaces are cured.
2 The concrete placing temperature is at least the mean air temperature. Higher placing temperatures are not significant.
3 The table applies to any concrete that satisfies the criteria given in section 4.1.7.

Specified concrete	Required cube strength [N/mm^2]	Type of cement	Mean air temperature (°C)		
			5	10	15
Grade 20	5	PC-52.5	5	$3\frac{1}{2}$	$2\frac{1}{2}$
	5	PC-42.5	6	4	$2\frac{1}{2}$
	10		9	$6\frac{1}{2}$	5
			17	13	10
Grade 25	5	PC-52.5	4	$2\frac{1}{2}$	2
	5	PC42.5	$4\frac{1}{2}$	3	2
	10		$6\frac{1}{2}$	5	4
	15		12	9	7
	20		26	19	15
Grade 30	5	PC-52.2	$3\frac{1}{2}$	2	$1\frac{1}{2}$
	5	PC-42.5	4	$2\frac{1}{2}$	2
	10		5	4	3
	15		9	7	$5\frac{1}{2}$
	20		17	13	10
Grade 40	5	PC-52.5	2	$1\frac{1}{2}$	1
	5	PC-42.5	3	2	$1\frac{1}{2}$
	10		$3\frac{1}{2}$	$2\frac{1}{2}$	2
	15		5	4	3
	20		8	$5\frac{1}{2}$	$4\frac{1}{2}$
	30		20	15	12
Grade 50	5	PC-52.5	$1\frac{1}{2}$	1	1
	5	PC-42.5	$2\frac{1}{2}$	$1\frac{1}{2}$	1
	10		3	2	$1\frac{1}{2}$
	15		4	3	$2\frac{1}{2}$
	20		5	4	3
	30		10	8	6

Notes:

1 The minimum section dimension is not significant.
2 The assumption is made that the side forms to beams were removed when the concrete had a strength of 2 N/mm^2.
3 The table gives separate times to 5 N/mm^2 for PC-42.5 and PC-52.5. At greater strengths, the times apply to both types of cement.

Figure 4.17 Temperature-matched curing bath (Wexham Development Ltd).

these cubes give the strength of concrete test specimens with equal maturity to the point where the sensor is located. It does not give the *in-situ* strength, as this will depend on the compaction and water movements within the cast element.

Measuring maturity

The concept of maturity is described in section 4.2. To determine the maturity and thereby strength of a selected point in an element, the temperature–time history from casting needs to be known and the relationship between strength and maturity for the particular concrete. Maturity can be measured in several ways:

(a) Measuring the temperature at a selected point in the element using, say, a cast-in thermocouple, at frequent intervals, e.g. hourly, and calculating the maturity with an appropriate maturity function.

(b) Measuring the maturity with a maturity meter with its temperature sensor placed at a selected point in the element (Figure 4.18). The meter automatically calculates the maturity using a built-in maturity function. Some meters permit you to select an appropriate function based on cement type.

(c) Using a COMA maturity probe (Figure 4.19). This comprises a capillary tube filled with liquid. When the end of the tube is broken off, the liquid evaporates at a rate that is dependent on the temperature of the liquid. Immediately after placing the concrete, the end of the capillary tube is broken off and the tube with its scale card is placed in its outer shell and then immersed into the fresh concrete. Periodically the capillary tube and scale are removed from the shell and the maturity read directly from the scale. It is then replaced so that further readings can be taken. Advice from the

Figure 4.18 Maturity meter (Wexham Development Ltd).

manufacturer should be sought to determine if these probes are suitable for your particular concrete.

Penetration tests

The best known of the penetration tests is the Windsor Probe test (Figure 4.20). In these tests, a hardened steel probe is fired into the concrete. The depth it penetrates into the concrete is correlated to the concrete strength. For ease of measurement, all the probes are the same length and the length of probe projecting from the concrete is measured and thereby the depth of penetration. At low and very high concrete strengths, these tests are not very sensitive.

With this method and the following three test methods, the variability in the correlation between the test result and *in-situ* compressive strength has to be taken into account. For assessing whether a structure has achieved a required strength, it is normal to use the 90 per cent confidence limit as this gives a 95 per cent probability that the required strength

Figure 4.19 COMA maturity probe.

has been achieved. Keiller (1982) has shown that the 90 per cent confidence limits for a range of mature concretes are about ± 8 N/mm^2.

Break-off test

This is also known as the TNS-test (Figure 4.21). Immediately after placing and levelling the fresh concrete, plastic sleeves are pushed into the concrete so that the top of the sleeve is flush with the concrete surface. At pre-determined times, the sleeves are removed to give cores that are fixed at their bases to the main concrete element. Using special equipment, a horizontal force is applied to the side of the core (Figure 4.22), until it breaks off from the main concrete element. This break-off force is correlated to cube strength. Keiller (1982) has shown that the 90 per cent confidence limits for a range of mature concretes are about ± 8 N/mm^2. ACI 228 (1995) states that the repeatability, expressed as a coefficient of variation, is about 9 per cent.

Pull-out test

The principle of the pull-out test is shown in Figure 4.23. The force needed to pull out a cast in steel disc is directly correlated to the cube strength. If pull-out testing is being used to determine formwork striking times of slabs, it is recommended that the discs are located on the top surface of the slab as this is normally the weakest part of the section. Where the discs are to be located through formwork, it is necessary to provide 'windows' at these locations so that a small section of the form can be removed to enable the test

Figure 4.20 Windsor Probe.

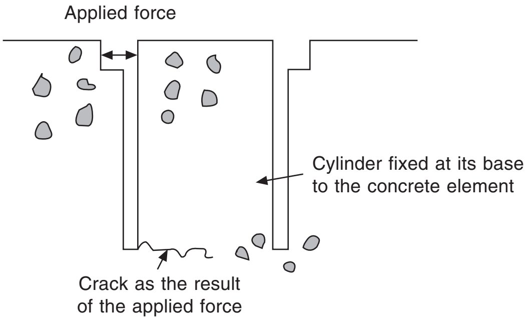


Figure 4.21 Principle of the break-off test.

equipment to rest against the concrete surface. Pull-out testing is described in a European Standard, EN 12504-3 *Testing concrete in structures – Part 3: Determination of pull-out force*. This states that where a general correlation is used, the 95 per cent confidence limits are unlikely to be better than ± 20 per cent of the mean of four valid results. Where there is a specially prepared correlation, this improves to ± 10 per cent. ACI 228 (1995) states that the repeatability, expressed as a coefficient of variation, is about 8 per cent. The best known of the pull-out tests is the LOK-test.

Figure 4.22 TNS-test equipment.

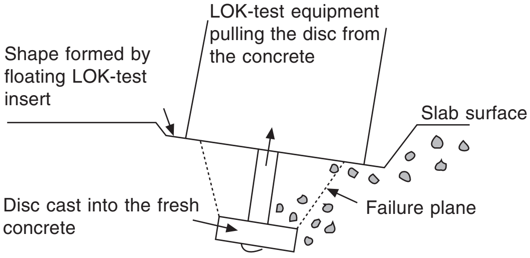


Figure 4.23 Principle of the pull-out test.

Rebound hammer

This test measures surface hardness. It is fully described in a European Standard EN 12504-2 *Testing concrete in structures – Part 2: Determination of rebound number*. It is a useful test for assessing the uniformity of an element and for selecting points for more detailed investigation, e.g. coring. It can be used to estimate strength if it is calibrated for the concrete being tested. ACI 228 (1995) states that the repeatability, expressed as a coefficient of variation, is about 10 per cent. At low concrete strengths these tests are not very sensitive and they are not recommended as a means of determining formwork striking times.

Coring

Coring is a direct measure of the *in-situ* strength. It is sometimes used to provide a calibration of an indirect method. Coring is rarely used for determining the rate of strength gain. However, it is frequently used to determine the *in-situ* strength of mature concrete. The test is described in a European Standard EN 12504-1 *Testing concrete in structures – Part 1: Cored specimens – Taking, examining and testing in compression*. The use of this test to assess the *in-situ* concrete strength is described in EN 13791 *Assessment of concrete compressive strength in structures or in structural elements*.

4.2 Maturity and accelerated curing

4.2.1 Learning objectives

1 Explain the theoretical basis for the concept of maturity
2 Compare formulae used for the determination of maturity and state advantages and disadvantages of each formula for different sets of circumstances
3 Calculate the maturity of concrete in a given set of circumstances
4 Design a curing cycle for a given set of circumstances, using the concept of maturity
5 Describe methods of measuring data for determining maturity
6 Explain the theoretical basis for accelerated curing
7 Describe practical applications of accelerated curing for *in-situ* and precast concrete
8 Describe methods for the accelerated curing of concrete
9 Describe the effects of accelerated curing on the properties of concrete

4.2.2 Concept of maturity

The rate of gain of strength of concrete depends on the reaction rate of the cement and additions with water (hydration). In common with all chemical reactions, the rate of reaction depends on the reaction temperature. Higher reaction temperatures give higher rates of reaction, e.g. the concrete gains strength more rapidly when its temperature is higher. Accelerated curing is the process by which the temperature of the concrete is raised artificially by applying external heat to speed up the rate of gain in strength.

Cube strength is a function of the concrete mix proportions, the time between casting and testing, and the temperature(s) at which the cube was stored. Therefore, for a particular mix, it should be possible to produce a relationship between time and temperature to predict maturity and strength. This is the purpose of maturity laws. For the measured or calculated temperature–time history at a point in a concrete section, the maturity law is used to calculate the maturity at that point and hence the strength of test cubes of equal maturity to that point. Traditionally, maturity was expressed in units of °Ch, but recently the trend has been to express the maturity as being equivalent to ...days at the standard curing temperature of control cubes (20°C in the UK).

1440°Ch = Equivalent age of 3 days at 20°C
3360°Ch = Equivalent age of 7 days at 20°C
13 440°Ch = Equivalent age of 28 days at 20°C

The relationship between strength and maturity is not unique for all concretes of a constant 28-day strength and therefore it has to be determined experimentally for the particular concrete being used. This is most easily achieved by crushing cubes which have been stored in water at 20°C at 1, 2, 3, 7 and 28 days to obtain the strength development curve for the specific concrete mix (Figure 4.24). Then, if the formwork striking times criterion requires, say, 10 N/mm^2 from the cubes of equal maturity to the structure, this equates to a maturity of at least 1.4 days at 20°C (Figure 4.24).

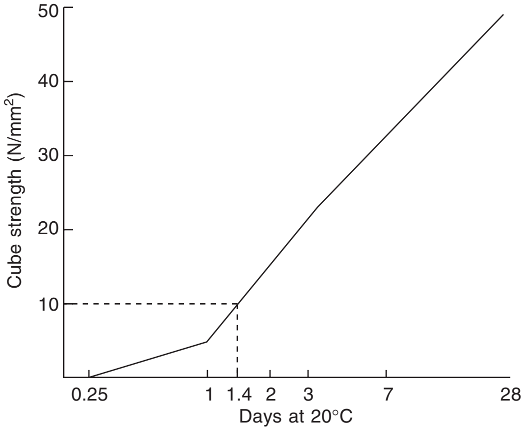


Figure 4.24 Strength gain for a specific PC-42.5 concrete.

4.2.3 Maturity laws

The maturity laws follow the same basic equation of:

$$\text{Equivalent age at } 20^\circ\text{C} = \sum k\Delta t$$

The objective of the maturity law is to produce an equation for maturity function, k, so that samples of a particular concrete with equal maturity have equal strength, regardless of the actual temperature–time history. Numerous laws have been proposed (ASTM C1074, 1987; Hansen and Pedersen, 1977, 1989; Carino and Tank, 1992; De Vries, 1992; Commission 42-CEA, 1981; Bresson, 1982; Sadgrove, 1975), of which the majority were derived empirically. In recent years, laws have been developed based on the concept of activation energy and the Arrhenius law on the rate of reaction. This equation takes the form:

$$\text{Equivalent age at } 20^\circ\text{C} = \Sigma\left[\exp\left[\frac{E}{R}\left(\frac{1}{293} - \frac{1}{\theta + 273}\right)\right]\right]\Delta t$$

where E = Activation energy obtained experimentally, kJ/mol
R = Molar gas constant = 0.008314 kJ/°Cmol
θ = Mean concrete temperature, °C, in time increment Δt

Different types of cement have different activation energies. Hansen and Pedersen (1977) suggest that for concretes containing cements now classified as PC-42.5 or 52.5, the equation becomes

Equivalent age at 20°C for values of θ of 20°C or greater

$$= \Sigma\left[\exp\left[\frac{33.5}{0.008314}\right]\left(\frac{1}{293} - \frac{1}{\theta + 273}\right)\right]\Delta t$$

or

Equivalent age at 20°C for values of θ below 20°C

$$= \Sigma\left[\exp\left[\frac{33.5 + 1.47(20 - \theta)}{0.008314}\right]\left[\frac{1}{293} - \frac{1}{\theta + 273}\right]\right]\Delta t$$

Tables of maturity function have been published (Commission 42-CEA, 1981; Bresson, 1982) to help to simplify the calculation.

For a given type of cement, empirical equations are available such as the Sadgrove equation:

$$\text{Equivalent age at 20°C} = \Sigma\left(\frac{\theta + 16}{36}\right)^2 \Delta t$$

which is valid for PC-42.5 or 52.5 concretes in the temperature range 1–45°C for equivalent ages at 20°C from 5 hours to 28 days (Sadgrove, 1975). Table 4.5 shows that this gives results very similar to those produced from the activation energy equations up to 40°C.

Table 4.5 Comparison of *k*-values for Portland cement 42.5 and 52.5 concretes, obtained from the Arrhenius and Sadgrove equations

Temperature, °C	Arrhenius equation	Sadgrove equation
0	0.15	0.20
10	0.50	0.52
20	1.00	1.00
30	1.57	1.63
40	2.41	2.42
50	3.59	3.36
60	5.22	4.46
70	7.42	5.71

An analysis of existing data (Harrison, 1975) showed that the Sadgrove equation could be applied to *some* concretes containing ggbs, pfa and silica fume in a more restricted range of temperature or time.

Given the very wide range of types and classes of cements, combinations, additions and admixtures, it is important that an appropriate or safe maturity law is selected. A maturity function where $k = \theta/20$ is considered to be a safe relationship in the strength range for formwork striking with any normal cement (including those with ggbs) and therefore, in the absence of data, this function could be used. This function implies that there is no strength development at 0°C and that the rate of strength gain is a linear function between 0°C and 20°C. This function cannot be used at temperatures below 0°C as one would get a negative value, i.e. a reduction in the strength already achieved.

This or some other selected maturity law can be checked experimentally by curing cubes at, say, 5°C, 20°C and 40°C, crushing them at intervals of time and plotting the results as a graph of cube strength against equivalent age at 20°C. The selected curing temperatures should span the likely range of *in-situ* temperatures as it is inadvisable to extrapolate. If all the results fall on the same line, the selected maturity law is appropriate for that concrete mix. When this ideal is not achieved, but the selected maturity law underestimates the strength development, it will provide a safe solution.

4.2.4 Calculations of maturity

Example 1 A hand calculation of maturity using the Sadgrove equation

$$\text{Equivalent age at } 20°\text{C} = \Sigma\left(\frac{\theta + 16}{36}\right)^2 \Delta t$$

Time from casting Hours	Concrete temperature °C	Δt Hours	Average concrete temperature in Δt °C	$k = [(\theta + 16)/36]^2$	$k\Delta t$ Hours	$\sum k\Delta t$ Hours
0	7					
		2	6.5	0.391	0.782	
2	6					0.78
		2	7	0.408	0.816	
4	8					1.78
		2	9	0.482	0.964	
6	10					2.56
		2	10.5	0.542	1.084	
8	11					3.64
		2	11	0.563	1.125	
10	11					4.77
		2	11.5	0.584	1.167	
12	12					5.94
		2	11.5	0.584	1.167	
14	11					7.10
		2	11	0.563	1.125	
16	11					8.23
		2	10.5	0.542	1.084	
18	10					9.31
		2	9.5	0.502	1.003	
20	9					10.32
		2	9	0.482	0.965	
22	9					11.28
		2	8.5	0.463	0.926	
24	8					12.21

After a real-time of 24 hours the maturity of the section was equivalent to 12.2 hours at 20°C.

Example 2 Checking the application of the Sadgrove equation to a Portland blastfurnace cement concrete

Experimental strength data, N/mm^2

Age of test Days	Curing temperature, °C				
	5	10	15	20	35
1	3	4	6	8	13
7	15	17	21	26	45
28	35	38	42.5	47.5	48

For each strength, the equivalent age at testing is calculated from $\Sigma\left(\frac{\theta + 16}{36}\right)^2 \Delta t$

Equivalent ages at testing

Age of test Days	Curing temperature, °C				
	5	10	15	20	35
1	0.34	0.52	0.74	1	2.01
7	2.38	3.64	5.18	7	14.07
28	9.52	14.56	20.72	28	56.28

Plotting the cube strength in the top table against the equivalent ages in the second table gives Figure 4.25.

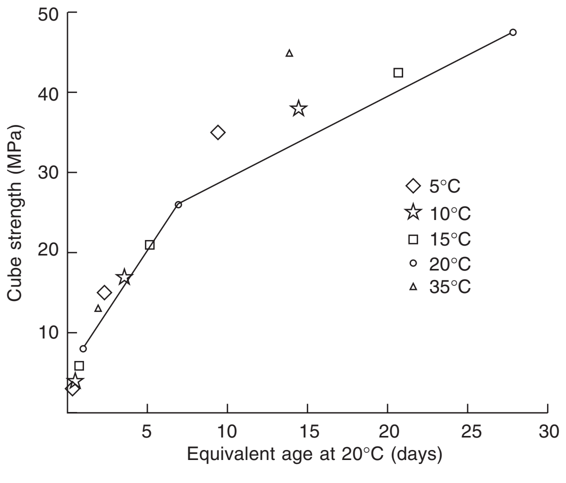


Figure 4.25 Check on the application of the Sadgrove equation.

As the points lie on or above the 20°C data line, the actual strengths are higher than the predicted strengths and therefore the Sadgrove equation can be safely applied to this concrete. **Note:** In practice data should have also been obtained at 3 days to give extra confidence in this critical period.

4.2.5 Methods of obtaining data for maturity calculations

The most common way of monitoring the temperature(s) in a section is to use thermocouples or other cast-in temperature sensors. The temperatures can be measured manually or by automatic data logging. Such systems can also be used to measure the temperature gradients across the section. Tie the thermocouples securely under (extra) reinforcement to protect them whilst concrete is being placed, but ensure that the sensors are not in contact with reinforcement. Suggest that the wire is bent so the sensor is about 25 mm from the reinforcing bar and the last point of tying. The maturity is calculated from the recorded temperatures and times.

An alternative is to use a maturity meter such as the COMA probe, (see section 4.1.7) which will automatically calculate the maturity. It is essential to check that the maturity meter uses an appropriate maturity function.

4.2.6 Applications of accelerated curing

Practical applications of accelerated curing include:

1 Ensuring a daily cycle with, for example, apartment formwork systems.
2 Speeding construction in winter conditions.
3 Ensuring multiple daily use of moulds used for precast concrete.
4 Reducing the time between production and delivery of precast concrete elements.

Applying heat to concrete is an expensive option and therefore it will only be applied in practice where benefits outweigh the cost increase.

4.2.7 Methods of accelerated curing

In *in-situ* construction, a daily cycle of construction only permits about a 14-hour curing period. In this time, concrete to slabs would be required to gain about 10 N/mm^2. In cold conditions, this is not achievable without some form of accelerated curing.

For *in-situ* construction, the heat required for accelerated curing can be calculated by the following rule-of-thumb (Concrete Society, 1995). Use 0.75 kW per cubic metre of concrete per °C rise from the placing temperature to 54°C. Allow an extra 10 per cent for absorption by steel forms. This gives total heat needed per cubic metre of concrete of 40.4 to 24.0 kW for placing temperatures between 5–25°C.

The duration of heating will depend on the outside air temperature, but typical European heating periods are as follows:

Air temperature (°C)	Heating time (hours)
+15	5.0
+10	5.5
+5	6.0
0	6.5
–5	7.0
–10	7.5

The number and layout of heaters will depend on the dimensions of the section to be cast, the duration of heating, the type of heater and the losses of heat to the outside air. The heater capacity will be given in kilowatt hours (kWh). The number of heaters can be calculated from:

$$\text{Numbers of heaters} = \frac{\text{Heat needed (kW/m}^3) \times \text{Volume (m}^3)}{\text{Heating time (h)} \times \text{Heater capacity (kWh)}}$$

In precast concrete construction, the cross-section sizes tend to be smaller and they often have the facilities to heat the concrete more rapidly and to higher temperatures. In these cases, there is often a delay before applying heat, a maximum rate of heating and cooling and a maximum temperature. The cycle is designed to minimize the curing period whilst keeping detrimental changes to concrete properties within acceptable limits (see section 4.2.8).

The main heating systems for accelerated curing are:

1 Steam in pipes or placed into a tent enveloping the structure (precast technique mainly).
2 Gas heating using infra-red heaters directed at the soffit or convector heaters. In both cases the space should be sealed to reduce heat losses. This system is common for *in-situ* construction.
3 Electric heating via overblankets or through low-voltage resistance wires cast into the concrete. The overblanket should be insulated to prevent excessive heat losses. Where concrete is heated directly, care is needed with respect to safety and the forms must be well insulated. This system is easily controlled by a time switch.
4 Turbo heaters which are space heaters fuelled by gas or oil. They heat the soffit by convection. End curtains are used to reduce heat losses.

4.2.8 Effect of accelerated curing on concrete properties

Accelerated curing has the following effects on the properties of concrete:

1 Reduces the ultimate strength by up to 30 per cent depending on the peak temperature reached.
2 A significant increase in coarse porosity depending on the temperature reached.
3 If the curing temperature is high, there is a significant risk of delayed ettringite formation (Lawrence *et al*., 1990).
4 For *in-situ* construction, accelerated curing will increase the risk of early-age thermal cracking caused by external restraint (Harrison, 1992).

The reason for the loss of strength and increase in coarse porosity is believed to be due to the hydration products forming close to the original cement grains and not spreading uniformly throughout the space between the cement grains.

If the peak temperature during accelerated curing does not exceed 65°C, the effect on long-term properties is not significant.

4.3 Assessment of safe striking times

4.3.1 Learning objectives

1. State and explain the main external factors that affect striking times for formwork and explain their significance.
2. Describe how concrete mix variations influence striking times.
3. Calculate safe striking times for a specific set of circumstances.
4. State the principal recommendations for striking of formwork given in standard specifications.

4.3.2 Main external factors that affect striking times

When formwork is removed from a concrete section, the section must not

- collapse;
- deflect excessively in the short or long term;
- be physically damaged as the formwork is removed;

In addition, consideration should be given as to whether the cast element after formwork removal will:

- be prone to freeze–thaw damage;
- crack due to thermal contraction of the surface.

If there is a significant risk of either of these actions, one option will be to delay formwork striking. The alternative is to insulate the sections after formwork removal but this is not always a practical option. However, it does release the formwork for further use.

It is necessary to consider if the section, after formwork removal, needs to be re-propped to take the loads from further construction, e.g. another floor, or to minimize creep deflection. Advice on re-propping and backpropping is given by the Concrete Society (1995).

4.3.3 Calculation of safe formwork striking times

Harrison (1995) describes in detail the background to the determination of formwork striking times. The following summary is based on this report.

To calculate the characteristic strength required by cubes of equal maturity to the structure before soffit formwork can be struck, it is necessary either to calculate the strength required, checking both moment, bond and deflection, or to use the following equation:

Characteristic strength of cubes of equal maturity to the structure

$$= \frac{\text{Dead load + construction load}}{\text{Total design load}} \times \text{grade of concrete}$$

This equation is very conservative for lightly reinforced sections. Because the dead load normally represents such a high proportion of the total design load, this collapse criterion also satisfies the deflection criterion in most situations.

An alternative approach has been developed by Beeby (2000) for flat slabs 300 mm or less in thickness. It is only applicable where the total unfactored construction load is less than or equal to the total unfactored design service load. The formula used to determine the characteristic strength of cubes of equal maturity to the structure, f_c, is:

$$f_c \geq f_{cu} \left(\frac{w}{W_{ser}} \right)^{1.67}$$

where f_{cu} = specified characteristic strength of the concrete
w = total unfactored construction load on the slab, kN/m^2
w_{ser} = total unfactored design service load, kN/m^2.

A minimum construction load of 0.75 kN/m^2 is recommended.

As all the methods give requirements in terms of cubes of equal maturity to the structure, if the test method is directly on the structure, e.g. a LOK-test, it is possible to reduce the strength requirement by 15 per cent as European standards assume that the *in-situ* strength is 85 per cent of the characteristic strength. Such refinements should only be applied by organizations that fully understand the uncertainties associated with limited measurements.

To minimize freeze–thaw damage, a strength of 2 N/mm^2 is required before striking the formwork. In many parts of the world, a higher criterion of 5 N/mm^2 is applied. It is important to prevent the young concrete being saturated before freezing as saturated concrete needs a much higher strength before it is resistant to freeze–thaw damage.

To protect the concrete surface from physical damage as the formwork is removed requires an equivalent age over 6 hours at 20°C for smooth surface forms to 8 hours at 20°C for unsealed plywood with a release agent. For the highest quality of plain finish, a strength of 2 N/mm^2 is required before striking.

The soffit formwork striking times, the most critical, have been determined in terms of the characteristic strength of test specimens of equal maturity to the structure. The practical problem that has to be faced is that any testing system is highly unlikely to provide sufficient information to calculate a good estimate of the population's characteristic strength. One can apply statistical techniques such as the *t*-test, but this assumes the tests are taken at random and results in a large margin. As a pragmatic solution, Harrison (1995) proposed using a 25 per cent margin to establish the *mean* strength required by test specimens of equal maturity to the structure. This is based on the assumption that the method by which this 'mean' strength is determined is conservative, e.g. the probe for a temperature-matched curing bath is placed in the coldest part of the section; LOK-test inserts are placed on the upper surface as this is the coldest and weakest zone.

4.3.4 Effects of the concrete on formwork striking times

Once the minimum strength for striking the formwork has been calculated, one has to assess when the concrete in the structural element has achieved the required strength. The strength in an element varies from point to point due to different temperature histories and for formwork striking, the assessment is made for the surface zone as this is the most relevant zone and it provides a safe value (the strength in the core will normally be higher).

The strength development in the structure will depend on:

1. The concrete used.
   (a) Higher strength class concretes will achieve a given strength in a shorter time.
   (b) Higher cement strength classes will achieve a given strength in a shorter time.
   (c) Cement type will affect the rate of gain of strength with those containing a high proportion of a second main constituent, e.g. ggbs gaining strength more slowly.
2. The concrete temperature at placing (a higher placing temperature will achieve a given strength in a shorter time, but the saving in time will depend on how well the heat is retained in the element).
3. The ambient temperature (higher ambient temperatures will lead to more rapid strength gain).
4. The insulation provided by the formwork or the ground (the more the formwork insulates the concrete, the slower is the rate of loss of the heat of hydration and the shorter is the time to achieve a given strength).
5. The size of the section (the larger the section, the shorter is the time to achieve a given strength).
6. The application of heat (accelerated curing) will shorten the time taken to achieve a given strength.

See sections 4.1.4–4.1.6 for more information on the strength gain of concrete.

4.3.5 Principal recommendations for formwork striking times

The European standard ENV13670-1: *Execution of concrete structures Part 1. Common rules* gives a requirement in performance terms:

Falsework and formwork shall not be removed until the concrete has gained sufficient strength:

- to resist damage to surfaces and arises during the striking;
- to take the loading imposed on the concrete member at that stage;
- to avoid deflections beyond the specified tolerances due to the elastic and ineleastic (creep) behaviour of the concrete.

Most other standards and codes of practice quantify the requirements in days or give rules of application that quantify the requirements. For example, BS 8110 gives a Table 6.2 as a rule of application.

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Further reading

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Concrete Society. (1995) *Formwork – a guide to good practice.* 2nd edn. This contains all the information you are likely to need on formwork, formwork design and accelerated curing.

5

Hot and cold weather concreting

E.A. Kay

5.1 Introduction

Concrete buildings and other structures are built in most countries around the world and in some regions the climates are typified by prolonged spells of either hot or cold weather. Readymix concrete and construction companies in these regions manage to produce good-quality concrete despite these climatic drawbacks. In many regions with adverse climates there are consensus specifications and guidance documents for concrete production (ACI 305, ACI 306, CIRIA/Concrete Society Guide) which give details of methods which can be used to combat the adverse conditions.

In the more temperate parts of the world, cool, humid weather is the norm. In these locations, although prolonged hot or cold spells are not unusual, it usually comes as a surprise when they arrive and it may be too late to apply even the most rudimentary precautions to mitigate their undesirable consequences.

Physiological effects in both hot and cold conditions should not be ignored. Operatives and supervisors cannot be expected to produce good-quality concrete if they have been exposed to the elements for long periods without proper protection.

5.2 Hot weather concreting

There is no simple definition of ‘hot weather’ for concreting purposes. It is not just a matter of a limiting temperature, as a hot, humid, calm day may not pose so many problems as a cooler day with lower humidity and high winds. In the latter case there will be a greater tendency for water to evaporate from exposed surfaces of concrete. ACI 305 defines hot weather as ‘any combination of high air temperature, low relative humidity, and wind velocity tending to impair the quality of fresh or hardened concrete or otherwise resulting in abnormal properties’.

5.2.1 Hot weather effects

High temperatures can affect concrete at all stages of the production and placing process and most of the effects can have consequences for long-term strength or durability. Some of the problems resulting from high temperatures are listed in Table 5.1. They are a consequence of high temperature increasing the rate of the hydration reaction and the movement of moisture within and from the surface of concrete. In the latter case relative humidity and wind speed also have a significant influence.

Table 5.1 Problems resulting from hot weather at various stages in the concrete production process

Stage	Effect
Production	Increased water demand for given workability Increased difficulty in controlling entrained air content
Transit	Loss of water by evaporation Increased rate of loss of workability
Placing, finishing and curing	Loss of water by evaporation Increased rate of loss of workability Increased rate of setting Increased tendency to plastic shrinkage cracking Higher peak temperature during hydration leading to increased tendency to cracking and lower long-term strength
Long-term	Lower strength Decreased durability Variable appearance

Higher water demand

The temperature of concrete has an effect on its workability for a given water content. Figure 5.1 indicates the relationship between concrete temperature and slump when the amount of mixing water is kept constant. It can be seen that an increase in concrete temperature from 15°C to 25°C results in a reduction of slump of approximately 25 mm.

Figure 5.2 shows the water content required to produce a mix with 75 mm slump at different temperatures. At 15°C the water content is approximately 164 l/m^3 while at 30°C this rises to 174 l/m^3. If water alone is used to provide the required workability at high temperatures there is a resultant loss in strength and durability. Increased water content also leads to increased drying shrinkage.

Rapid loss of workability

Concrete at high temperature loses workability at a faster rate because of the combined effects of loss of water through evaporation and the more rapid rate of the hydration reaction. Hydration of cement in concrete is an exothermic reaction, i.e. it produces heat.

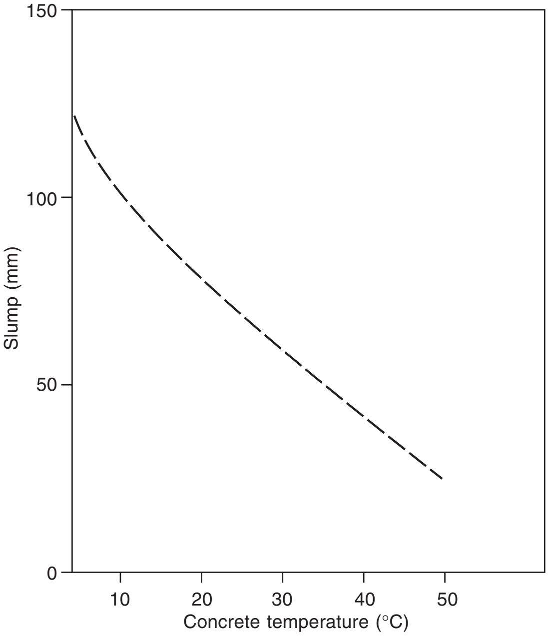


Figure 5.1 Effect of temperature on slump (after ACI 305).

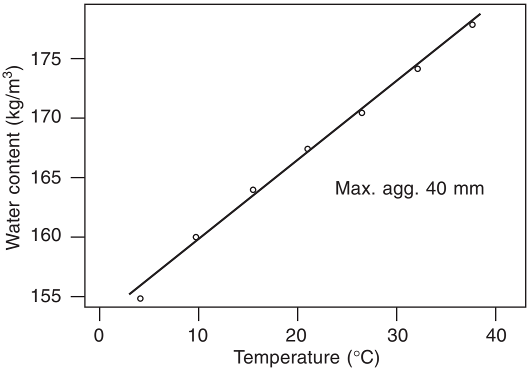


Figure 5.2 Effect of concrete temperature on the amount of water required to produce concrete with 75 mm slump (after ACI 305).

In general, the rate at which an exothermic reaction takes place doubles for each 10°C rise in temperature. Clearly, if the hydration reaction proceeds more rapidly, the paste will tend to stiffen earlier and the mix will lose workability. Thus there is less time available between addition of water to the mix until finishing must be complete. This can lead to a temptation to compensate for lack of workability at site by addition of water with detrimental consequences to strength and durability. Low workability can lead to difficulties in achieving adequate compaction which can again affect the strength and durability of the final product.

Decrease in setting time

For similar reasons related to the rate of the hydration reaction, concrete mixes pass initial set more quickly at higher temperatures. This also shortens the time available for transit, placing and finishing. The rate of set is also dependent on cement type, the physical properties of the cement such as fineness and the presence of other cementitious materials such as pulverized flyash and ground granulated blastfurnace slag.

Plastic shrinkage

There is a tendency in concrete mixes for the solid more dense constituents to move downwards while at the same time water, which is the least dense of the constituents, tends to move upwards. This upward movement of water in recently placed concrete is known as bleeding. The bleed water finds its way to the upper surface of concrete members such as slabs where it may be lost by evaporation. The rate of evaporation increases with increasing temperature and wind speed and with decreasing relative humidity. If moisture is lost from the surface at a greater rate than the rate at which it is replaced by bleeding from below, there is a reduction in volume of the surface layer. This change in volume is resisted by the mass of concrete below which does not experience a volume change. The restraint from the underlying concrete can cause tensile stresses in the surface layer sufficient to result in cracks in the immature concrete of the surface layer. This is known as plastic shrinkage cracking.

Evaporation can be estimated for different conditions by use of the chart in Figure 5.3. Although this chart has been criticized because it relates to evaporation from an open pan rather than a concrete surface, there must be a fairly direct relationship between the two. ACI 305 indicates that precautions against plastic shrinkage should be taken if the rate of evaporation estimated from the chart approaches 1 kg/m^2/h.

Hydration peak temperature/thermal cracking

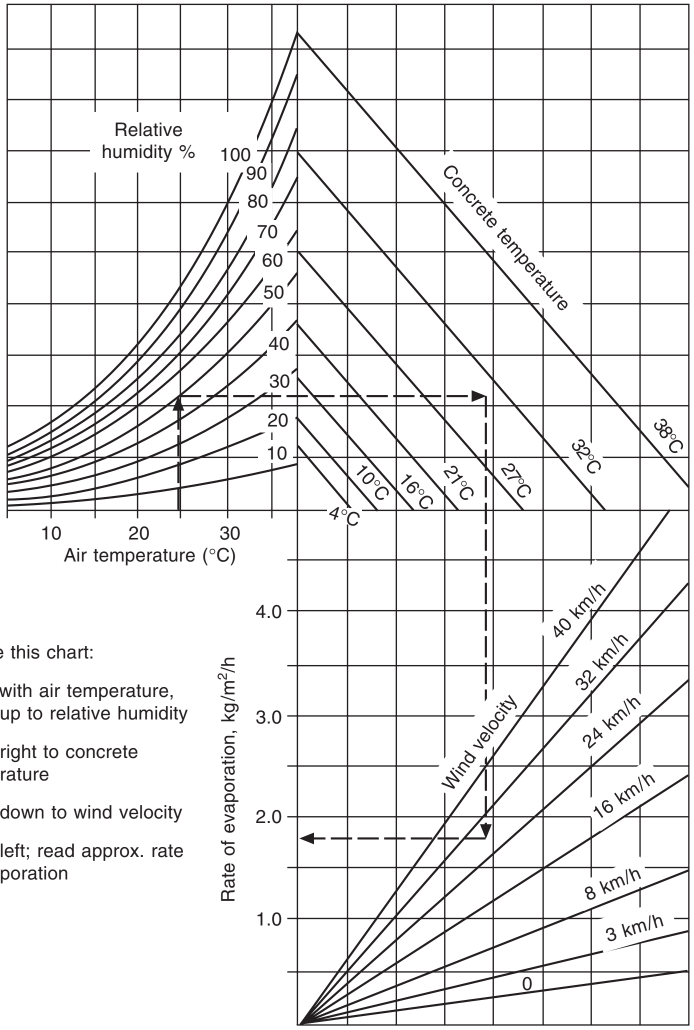
As noted above, the hydration reaction produces heat and the temperature of the concrete rises. If the initial temperature of the concrete is higher, the reaction proceeds more rapidly and the rate of heat evolution is increased. This means that the peak temperature reached is also increased. There is consequently a greater tendency for cracking as the concrete shrinks as it cools from the peak temperature.

Strength

Although the higher rate of hydration leads to higher early strength under hot conditions, this is not reflected in higher long-term strength. This is illustrated in Figure 5.4. The effect on 28-day strength over the typical range of temperatures likely to be encountered in the United Kingdom is not great but in areas such as the Middle East, where fresh concrete temperatures can rise to the upper 30s°C, there can be a significant reduction in long-term strength. The difficulties of achieving good compaction because of the loss in workability described above, can also lead to reduction of the *in-situ* concrete strength.

Durability

Many deterioration mechanisms depend on the passage of fluids or gasses through the concrete pore structure. Achieving a less ‘permeable’ concrete is one if the principal objectives when trying to obtain durability. The main means of doing this is to produce a concrete with low water/cement ratio. As noted above, high temperature effects both the



To use this chart:

1. Enter with air temperature, move up to relative humidity
2. Move right to concrete temperature
3. Move down to wind velocity
4. Move left; read approx. rate of evaporation

Figure 5.3 Effect of ambient temperature, relative humidity, concrete temperature and wind speed on rate of evaporation of surface moisture (after ACI 305).

initial workability and the rate at which workability is lost and hence there can be a temptation to add more water at the mixer or at site. This would lead directly to concrete which is more vulnerable to freeze–thaw, weathering, sulfate attack and the penetration of carbon dioxide and chloride solutions leading to reinforcement corrosion. The lower workability resulting from high temperature can lead to poor compaction which also leaves the concrete more vulnerable to deterioration. Plastic shrinkage or early thermal cracks can also lead to reduced durability as they may permit moisture, carbon dioxide, oxygen or chlorides to gain easy access to the concrete or reinforcement

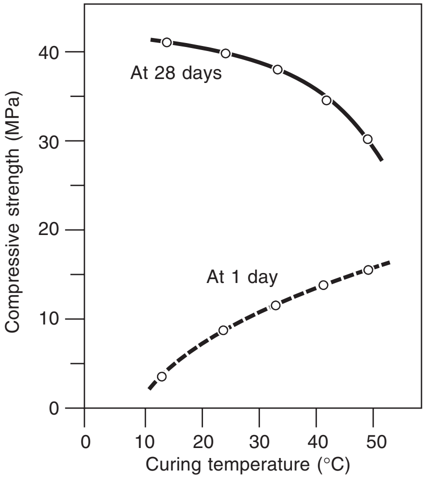


Figure 5.4 Variation of 1-day strength and 28-day strength with curing temperature (after ACI 305).

5.2.2 Control measures

A number of different methods are used to alleviate the effects of hot weather. They are mostly aimed at reducing the temperature of the concrete at the time of placing by either cooling the ingredients, reducing the heat gain experienced during mixing, transit and placing or by cooling the concrete itself.

Ingredients

The amount of heat contained in a body or mass of material is the product of its mass, specific heat and temperature. The various ingredients in a concrete mix are present in different masses and they have widely different specific heats. The temperature of freshly mixed concrete can be approximated to:

$$T = \frac{0.22\,(T_a W_a + T_c W_c) + T_w W_w + T_a W_{wa}}{0.22\,(W_a W_c) + W_w + W_{wa}}$$

where T = temperature of freshly mixed concrete

T_a, T_c, T_w = temperature of aggregate, cement and mixing water respectively

W_a, W_c, W_w, W_{wa} = weight of aggregate, cement, mixing water and free water on aggregate respectively in kg/m^3.

Hence, the reduction in temperature which can be achieved is different for each individual ingredient. As can be seen from the above equation, water has the greatest effect on concrete temperature, kilogram for kilogram, because of its higher specific heat. Figure 5.5 shows the reduction in temperature which can be achieved by replacing mixing water at various temperatures with water at 7°C. For a typical mix containing 180 l/m^3 of water, a reduction of 7°C in the temperature of the resulting mix can be obtained by using water at 7°C rather than 32°C.

To obtain water at this temperature in climates such as the Middle East would require the use of a chiller plant or the placing of ice in the storage tank. Moderate reductions in temperature can be obtained by shading and painting the storage tanks white and insulating the delivery pipework.

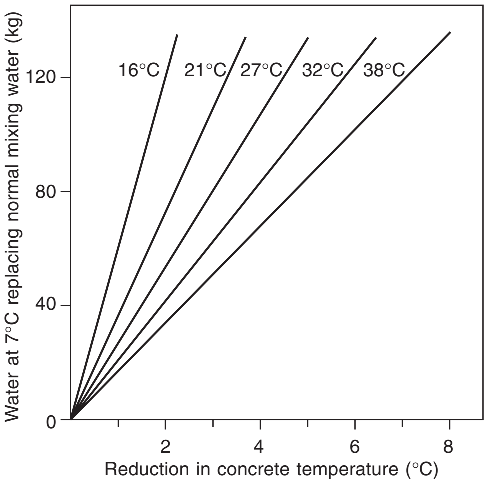


Figure 5.5 Effect of using chilled mixing water on reduction of initial concrete temperature. Values shown are reductions from temperature which would be obtained when using water at the temperatures shown on the curves (after ACI 305).

Inclusion of ice as part of the mixing water is highly effective in reducing concrete temperature because of the latent heat taken in as the ice melts. Ice absorbs 335 J/g as it changes to water. The most effective method is to use flaked ice placed directly in the mixer to replace part or all of the mixing water.

Figure 5.6 illustrates the possible reductions in concrete temperature which can be

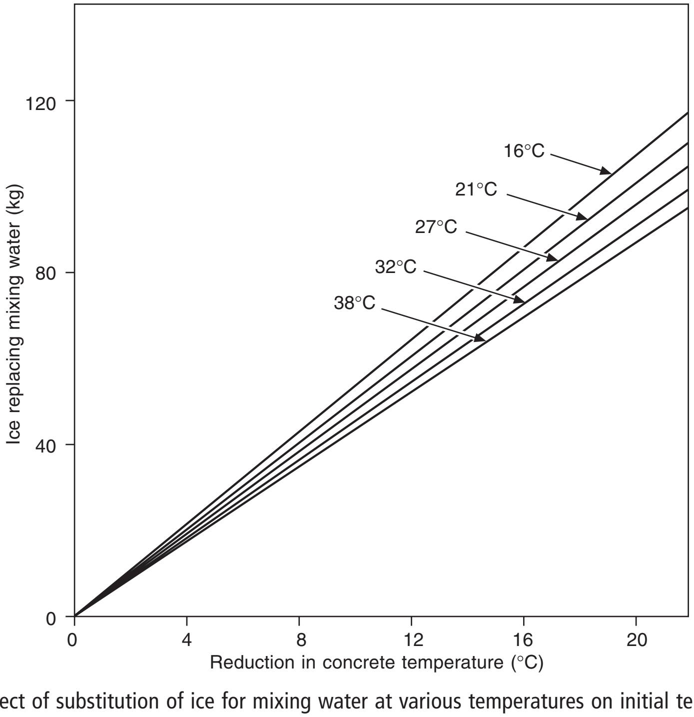


Figure 5.6 Effect of substitution of ice for mixing water at various temperatures on initial temperature of concrete (after ACI 305).

obtained by substituting various amounts of ice at 0°C for mixing water at the temperatures shown. Mixing should be continued until the ice has completely melted. The effect can be maximized by draining the aggregates so that they contain as little moisture as possible. In the hot arid regions of the world this is rarely a problem as most aggregates are delivered in a dry condition.

The equation for estimating the initial concrete temperature modified for the inclusion of ice is as follows:

$$T = \frac{0.22(T_a W_a + T_c W_c)}{0.22(W_a + W_c) + W_w + W_i + W_{wa}} + \frac{(W_w - W_i)T_w + W_{wa} T_a - 79.6 W_i}{0.22(W_a + W_c) + W_w + W_i + W_{wa}}$$

where the symbols have the meaning above and W_i = weight of ice.

Although aggregates have a lower specific heat than water, they constitute such a large proportion of the concrete mix that their temperature can have a significant effect on initial concrete temperature. However, it is much more difficult to reduce the temperature of aggregates than it is to reduce the temperature of water. The best practical approach is usually to keep the aggregates as cool as possible by shading the stockpiles from the direct rays of the sun. This is often accomplished in the Middle East by the use of a lightweight roof at high level (high enough for delivery lorries to tip and for face shovels to extract) with shade netting on the sides. The open access side should be on the face least likely to be affected by the direct rays of the sun, i.e. north in the northern hemisphere. Sprinkling or fog spraying of coarse aggregates with water is effective in reducing aggregate temperatures by evaporation and direct cooling. However, this needs to be controlled as it can result in variations in the surface moisture content.

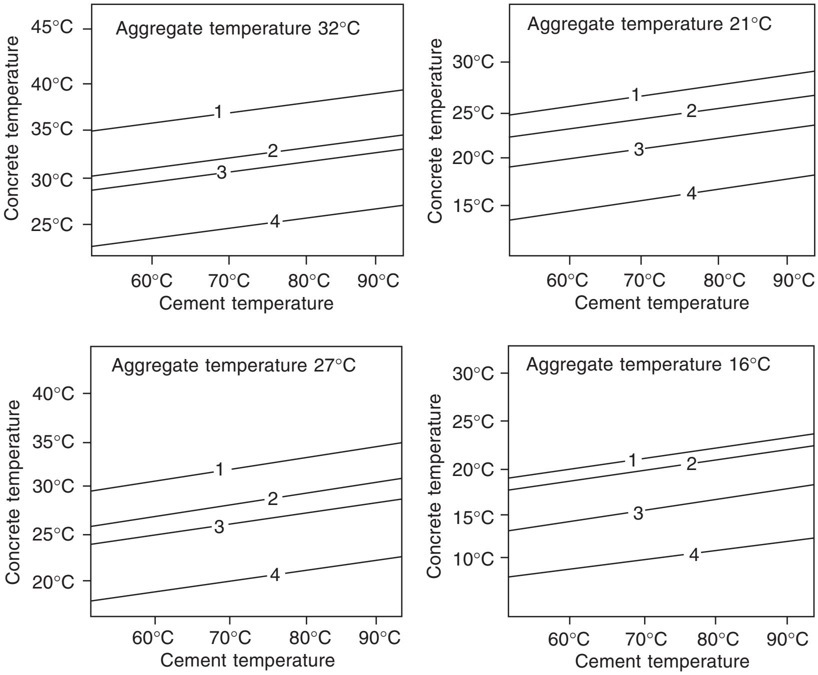
The temperature of cement is difficult to control (Figure 5.7). It may well be delivered hot to site as a result of the heat generated during grinding. It will then lose heat only slowly during storage. On large sites with a significant throughput it may be necessary to install two silos so that the time between delivery and use is extended. Silos should in any event be painted white to minimize temperature build-up from solar gain.

Admixtures can play a large part in reducing some of the adverse effects of concreting at high temperatures. Water-reducing admixtures can be used to offset the reduction in slump described earlier without increasing the water/cement ratio. Their use may somewhat increase the rate of slump loss. However, even if the initial slump is increased to compensate for any increased slump loss resulting from their use, there will still be a beneficial net reduction in water content. This can also be used to compensate for any reduction in long-term strength. Some admixtures may promote early bleeding and this has been found helpful in preventing the drying of the top surface of concrete placed in conditions of high temperature and low humidity.

Production and delivery

If all the precautions outlined above to reduce the temperature of the ingredients have been taken, it is possible to lower the temperature of the concrete still further by paying attention to aspects of production and delivery. The energy imparted during batching and agitation in the delivery truck can result in rises in temperature. Hence these should be kept to a minimum commensurate with the need for thorough mixing. Where truck mixers are being used, it may be possible to batch the dry ingredients at the plant and add the mixing water at site.

Reductions in temperature can be achieved by painting the batching plant and delivery



Curve (1) – Mixing water at temperature of aggregate
Curve (2) – Mixing water at 10°C
Curve (3) – Mixing water at temperature of aggregate; 25% of mixing water by weight replaced by ice
Curve (4) – Mixing water at temperature of aggregate; 50% of mixing water by weight replaced by ice

Figure 5.7 Influence of temperature of ingredients on initial concrete temperature (after ACI 305).

trucks white. As an example, concrete in a clean white drum could be approximately 1.5°C cooler than concrete in a red drum based on a 1-hour delivery time.

Temperature rise due to both ambient conditions and hydration and slump loss all increase with the passage of time. Hence the period between batching and delivery should be kept to the minimum possible. The dispatching of trucks from the batching plant should be carefully coordinated with the rate at which concrete is being placed to avoid prolonged standing at site before the trucks are discharged.

A method which has been used to reduce the temperature of concrete just before placing is to inject with liquid nitrogen. This is accomplished by inserting a lance into the back of the mixer truck. Extremely low temperatures can be achieved but the process is expensive.

Placing and curing

Placing and curing must be carefully planned if good results are to be obtained in hot weather. The aims are in many respects the same as in other climates:

- Place with the minimum of delay so that slump loss is minimized
- Place concrete as close as possible to its final position
- Place in shallow layers so that vibration can be achieved into the layer below
- The timing of finishing operations is controlled only by the condition of the concrete

- Curing is such that the concrete always retains sufficient moisture and its temperature is controlled so that hydration is continuous.

Planning should be aimed at transporting, placing, consolidating and finishing the concrete at the fastest possible rate. One important decision which needs to be taken is the time of the pour. There can be a considerable advantage in concreting at night when the ingredients are probably at their lowest temperature and there is no increase in temperature during transit due to solar gain.

The planning should also take into account that the more rapid rate of slump loss in hot weather places greater strain on vibrating equipment and hence breakdowns may be more frequent. Provision should be made for sufficient numbers of standby vibrators – at least one standby for every three vibrators in use.

The temperature of shutters, reinforcement and previous pours can be reduced by shading them prior to the concreting operation. The fresh concrete should be protected by windbreaks particularly under hot arid conditions.

The freshly placed concrete should also be shaded from the direct rays of the sun and curing should be applied at the earliest opportunity after finishing. Good curing is extremely important in hot climates as the conditions often favour rapid loss of moisture from the surface. One method that is often used for slabs in the Middle East is to cover the surface with a sheet of polythene as soon after finishing as possible. The polythene in turn is covered with wet hessian to reduce temperature build-up. As soon as the concrete has set the layers are reversed. It is not unusual for wet curing to continue for at least 7 days under hot arid conditions.

The measures which can be taken at all stages to reduce to adverse effects of hot weather are summarized in Table 5.2.

Table 5.2 Summary of measures to reduce the adverse effects of hot weather

Stage	Measure
Production	Shade aggregate stockpiles Spray stockpiles with water Increase cement silo capacity Paint batching plant white Shade water tank Paint water tank white Insulate water pipelines Use chilled water Use ice as part of mixing water Use admixtures to counteract slump loss Use cement or combinations with low heat evolution Minimize mixing times
Transit	Paint mixer trucks white Minimize transit times Batch dry and add water at site
Placing and curing	Plan operations carefully Match production to placing rates Reduce layer thickness Provide adequate standby vibrators Place concrete at night Minimize placing time Shade workplace Use windbreaks Apply curing early

Temperate climates

The methods described above have been developed for countries which have prolonged periods of hot weather and they are not all appropriate to temperate climates where hot weather occurs only infrequently. BS 8110 states that 'In hot weather special precautions may be necessary to avoid loss of moisture and/or rapid stiffening of concrete which prevents its proper compaction' and refers to BS 5328 for guidance on selection of materials and specifying concrete for work in hot weather. BS 8110 also requires a limiting concrete temperature of 30°C at time of placing.

BS 5328 suggests that the overall approach to working in hot weather should include consideration of modifying the concrete by one or more of the following:

- Use of admixtures to retard hydration or increase initial workability
- Use of a cement or combination with low heat evolution
- Specifying a maximum fresh concrete temperature

In more cases than not there will have been no preplanning for hot weather and no special measures will be in place. The most common consequences are cold joints and plastic shrinkage cracks. The most appropriate measure which can be taken in an emergency are:

- Reduce layer thickness
- Maintain good contact with the batching plant and match production rate to the placing rate
- If possible, increase admixture dosage to compensate for slump loss
- Carry out slump tests on each delivery of concrete
- Put appropriate curing procedures in place as soon as practicable.

BSEN 206 does not make any reference to production of concrete under hot weather conditions. DDENV 13670-1 does make reference to some hot weather effects; for example paragraph 8.3(7) 'If the ambient temperature is forecasted [sic] to be high at the time of casting or in the curing period, precautions shall be planned to protect the concrete against damaging effects.'

5.3 Cold weather concreting

ACI 306 defines cold weather as 'a period when for more than 3 successive days the mean daily temperature drops below 5°C'. BS 8110 notes that concrete may suffer permanent damage if its temperature falls below 0°C before it is mature enough to resist disruption by freezing. BS 8110 goes on to say that the temperature of the concrete should at no point, fall below 5°C until it reaches a strength of 5 N/mm^2.

5.3.1 Cold weather effects

As for the hot weather effects described earlier, cold weather can adversely affect concrete at all stages. Some of the problems are listed in Table 5.3. However, there can be some benefits from low initial temperature. Concrete which is placed at low temperatures, but which is not allowed to freeze and receives good curing, develops higher ultimate strength,

Table 5.3 Problems resulting from cold weather at various stages in the life of concrete

Stage	Effect
Production	Incorporation of frost-bound material
Transit	Cooling of mix
Placing, finishing and curing	Formation of ice crystals in concrete Increased thermal gradients/increased tendency to thermal cracking Delayed formwork removal Slower gain in strength Greater chance of formwork stripping damage Bleed water may remain on surface
Long-term	Slower setting Slower gain in strength Freeze–thaw damage Variable appearance

greater durability and is less subject to thermal cracking than similar concrete placed at higher temperatures.

The main problems associated with cold weather are frost damage to immature concrete and slow gain in strength leading to later stripping times and the possibility of increased damage when the shutters are removed. Concrete is vulnerable to freezing temperatures both before and after it has stiffened. There are two stages:

1 Expansion of water as it freezes in plastic concrete causes such severe damage that the concrete is unusable
2 Concrete can be permanently damaged by the pressures exerted by ice crystal growth if this occurs after the concrete has stiffened but before it has gained adequate maturity. This weakens the paste–aggregate bond and may reduce strength by up to 50 per cent. The porosity of the concrete may be adversely affected causing a loss of durability.

5.3.2 Maturity

The concept of maturity is important in relation to cold weather concreting. Maturity is the mathematical product of temperature and time. It has been found that concretes from the same mix which have the same maturity, whatever combination of time and temperature make up that maturity, have approximately the same strength. Concrete gains strength only very slowly at a temperature of –10°C and this has been used as the baseline for temperatures used in maturity calculations. Hence maturity is given by

$$M = (T + 10) \times t$$

where M = maturity (°Ch)
T = temperature (°C)
t = time (housrs)

As an example concrete maintained at a temperature of 20°C for 72 hours has a maturity given by

$$\begin{aligned} M &= (20 + 10) \times 72 \\ &= 2160^{\circ}\text{Ch} \end{aligned}$$

If the concrete had reached a strength of 25 N/mn^2 at this maturity, it is possible to calculate the time for the concrete to reach the same strength if it were maintained at any other temperature. If it were maintained at 15°C the time required would be

$$\frac{2160}{(15 + 10)} = 86 \text{ hours}$$

There are other suggested formulae for calculating maturity. For example Sadgrove (1975) developed a formula for concrete cured below 20°C:

$$M = (T + 16)^2 \times t$$

Using the same example as before (20°C for 72 hours) gives a maturity $(20 + 16)^2 \times 72 = 93312$ °C^{2}h. Likewise if the concrete had been cured at 15°C it would have attached the same strength after

$$\frac{93\,312}{(15 + 16)^2} \text{ hours}$$

i.e. 97 hours

5.3.3 Heat transfer and heat loss

Warm bodies tend to lose heat to their cooler surroundings. The heat can be transferred by any of three mechanisms – conduction, radiation and convection. Conduction can be considered as the flow of heat along a body or between one body and another. The heat flows from the hotter region to the cooler until the temperature is uniform. Radiation is the transfer of heat in wave form. A good example of this is the heat from the sun which can travel through space and be felt on Earth. Convection occurs in fluids and involves relative movements of regions of fluid at different temperatures. If a warm region exists in a fluid, the fluid in this region will be less dense than the remainder because of the expansion associated with rise in temperature. Hence the warmer fluid starts to rise. Cooler fluid moves in to replace the rising warmer fluid and convection currents are set up. The process is continuous under conditions where no heat is added until the temperature is more or less uniform throughout the fluid. Concrete can lose the heat produced during hydration by all three of these processes: by conduction to the formwork and adjacent concrete; by radiation to an open sky on a clear night; and by convection to the air above.

The heat loss from concrete increases in high winds. In addition, the wind increases the rate of evaporation thus removing more heat from the concrete.

5.3.4 Control measures

The control measures which can be adopted will depend on the size of the project and whether or not readymixed concrete is being used. The objectives are:

- to obtain a concrete which is delivered to the pour at an adequate temperature and
- to protect and maintain the placed concrete at an adequate temperature until it has gained sufficient maturity to be able to withstand exposure to freezing conditions.

It may also be necessary to keep formwork and falsework in position for longer periods because of the reduced rate of strength gain.

As in the case of hot weather, it is necessary to plan ahead so that the required materials and equipment are available before the onset of cold weather and so that personnel at all levels are familiar with the procedures to be adopted. In most parts of Britain the first frosts are expected during late November. However, in the more easterly and northerly parts of England and in Scotland preparations for cold weather should be made somewhat earlier.

BS 5328 Part 3 requires that the temperature of the fresh concrete at the time of delivery is not less than 5°C and also that the mixing plant, aggregates and mixing water are free from snow, ice and frost when working in cold weather. EN 206 requires a minimum temperature of 5°C at the time of delivery. ENV 13670 requires precautions to be planned to protect concrete against damage due to freezing if the ambient temperature is forecast to be below 0°C at the time of casting or during the curing period.

Ingredients

The most simple and effective means of producing concrete with a temperature above 5°C in cold weather is to use heated water. This will usually be all that is required except in the most severe circumstances with ambient temperatures below freezing point for prolonged periods. In this case it may also be necessary to heat the aggregates.

A number of different techniques are available for heating the mixing water. Steam can be used either by injection or by passing it through a coil in the storage tank. If a steam plant is used for heating the water, steam lances can be used for thawing out the surface of aggregate stockpiles. Immersion heaters are also available for electrical or propane operation. The storage tank must be insulated to cut down heat loss and of sufficient capacity that a supply of heated water is available at uniform temperature at time of peak demand. The mixer's own water tank should also be lagged and all pipework should be lagged or buried at a level below that influenced by frost (450–600 mm deep). An alternative to lagging of pipes is electrically operated low-voltage insulated heating tape.

Aggregate stockpiles should be free of lumps of snow, ice or aggregate. Lumps which are larger than 75 mm can survive the mixing process and remain in the concrete until it is placed. Windbreaks should be erected around the stockpiles and batching plant to reduce the chill factor. The stockpiles should be protected from the action of frost by covering them with tarpaulins or, better still, by an insulating layer covered with tarpaulins or other waterproof sheet. Overhead metal storage bins should have their sides insulated and the covers should be kept in place continually except when material is being loaded.

Aggregate stockpiles can be thawed out or heated by use of steam in a number of different ways:

- As noted above, steam lances can be used for dealing with a superficial layer of frost-affected aggregate
- Closed steam coils under the stockpiles
- Injecting steam into the stockpiles.

If the steam is in a closed-pipe system there is the possibility of hot dry spots within the stockpiles. Steam jets liberated within the stockpile can cause moisture variation. Another means of heating aggregates is the use of flexible insulating mats which contain electrical heating elements.

When a spell of cold weather is forecast, the aggregate stockpiles should be built up in anticipation that there may be a restriction on deliveries either because of poor conditions on the roads or because the pits are frozen.

Few additional precautions are needed for cement storage except that, if it is to be stored in silos for an extended period, these should be lagged to reduce heat losses and also to prevent condensation. Consideration should be given to using faster-reacting cements or avoidance of cement types which prolong gain in strength so that the possibility of damage before the concrete has gained sufficient strength is reduced and shutter stripping times are not excessive.

Accelerating admixtures can also be used to increase the rate of strength gain. Air-entraining admixtures are used to combat the detrimental effects of freeze/thaw cycles on hardened concrete. ACI 306 recommends that any concrete likely to be exposed to freezing in a saturated condition during construction should be properly air-entrained even though it will not be exposed to freezing in service.

Production and delivery

If circumstances permit, the whole of the mixing plant and the materials should be kept under cover. If this is not possible, the plant should be as sheltered as possible with windbreaks to give protection against the wind and driving rain, sleet and snow. Open chutes can result in significant heat losses and ice may build up and cause blockages.

The temperature of fresh concrete can be calculated using the equations given previously in section 5.2. If the aggregates are frozen, the moisture in them will be present as ice and the formula has to be modified to include the effect of raising the temperature of the ice to 0°C and the latent heat required to change the ice into water.

$$T = \frac{0.22\,(T_a W_a + T_c W_c) + T_w W_s + W_{wa}\,(0.5\,T_a - 80)}{0.22\,(W_a + W_c) + W_w + W_{wa}}$$

The fine and coarse aggregate can be treated separately in this equation if only one or other is in a frozen condition. The C & CA publication *Winter Concreting* (Pink, 1967) gives charts for estimating concrete temperature when mixed with heated water. An example is given in Figure 5.8. The same publication also gives a range of charts related to different concrete strengths and pour configurations for the minimum concrete temperatures required for different air temperatures on a falling thermometer. The required temperature is such that the temperature of the concrete in place should not fall below freezing point until it has achieved adequate maturity. If this required temperature cannot be achieved then the alternatives are to change to a mix of higher strength and hence greater heat evolution, use a more rapid hardening cement, improve the insulation on the pour or use a heated enclosure around the pour.

It may be advisable to batch concrete with lower slump than normal if members with a large surface area such as slabs are being constructed. This is because bleeding is minimized and set occurs earlier. It is possible for bleed water to remain on the surface for such a period that it interferes with finishing operations and a soft, dusting surface may result.

If very hot water (say, over 60°C) is being used to produce heated concrete, it may be necessary to change the batching sequence in order to reduce the possibility of flash set or balling. Loading the hot water and coarse aggregate ahead of the cement or slowing

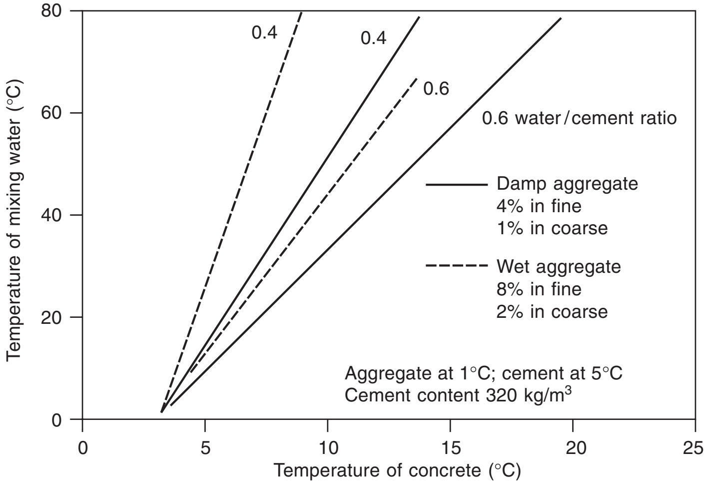


Figure 5.8 Relationship between mixing water temperature and concrete temperature under cold conditions (after Pink, 1967).

water entry while cement and aggregate are loading will reduce the likelihood of this problem.

Transit times to the site should be minimized as significant heat losses can occur. Interestingly, some Swedish work reported in ACI 306 has found that there are greater heat losses from revolving drum mixers than for dump trucks. ACI 306 recommends that heat losses from drum mixers can be minimized by revolving the drum no more than absolutely necessary during delivery.

Placing and curing

The objective when placing concrete during cold weather is to prevent damage from freezing at early ages. BS 8110 indicates that this can be achieved if no part of the concrete falls below 5°C until it reaches a strength of 5 N/mm^2. In addition:

- Shutters should not be removed until the concrete has gained sufficient strength taking into account the slower rate of gain in cold weather
- Methods of curing and protection should be employed so that the concrete undergoes normal strength development without excessive heat and so that the concrete is not saturated at the end of the protection period
- Sudden changes in temperature, such as that due to the removal of insulation, should be avoided before the strength has developed sufficiently to withstand temperature stresses.

All snow, ice and frost should be removed from shutters, reinforcement and adjacent concrete. This can be accomplished by hot-air blowers. When concrete is to be placed on the subgrade, this should also not be in a frozen condition. This can be achieved in some cases by use of insulating mats in place for a few days but quite often it will be necessary to use an external heat source.

In many cases, the temperature of the concrete in position can be maintained above the required value by the use of insulated formwork and insulated mats. Timber is a reasonably good insulator and timber formwork may in itself suffice in short spells of moderately cold weather. When used in combination with insulation it can protect concrete during very cold periods.

Metal is a good conductor of heat and thin metal formwork has little insulation value. It must be used with additional insulation during cold weather. An alternative to insulated formwork is to use a heated enclosure. In this case metal formwork is an advantage because it assists the transfer of heat to the concrete. The enclosure, which can be of sheets of plastic or tarpaulin on timber frames supported on scaffolding, must completely enclose the member. The heating source should not result in concentrations of CO_2 in the enclosure as this can damage freshly placed concrete. Jets of hot air should not be played directly on to fresh concrete as this could induce rapid drying and hence reduced strength.

Water curing is inadvisable in near-freezing conditions as the saturated concrete could be damaged. Under the prevailing weather conditions, rapid loss of moisture due to evaporation is unlikely and, in the case of concrete in insulated formwork it is only necessary to cover the member completely in order to retain sufficient water. When the period required for protective measures is completed the surface temperature of the concrete should be lowered slowly to avoid thermal shock. This is best achieved by first easing back the insulation and subsequently the formwork until there is a small air gap. In the case of a heated enclosure the temperature should be lowered gradually.

As noted above, protection should remain in place until the concrete has reached a strength of 5 N/mm^2. This can be checked by taking additional cubes and storing them alongside the member. Table 6.1 in BS 8110 Part 1 gives minimum periods of curing and protection for different types of cement, different ambient conditions and different average surface temperatures of concrete. Table E.1 in ENV 13670 gives minimum curing periods for different temperatures and also takes account of cements with different rates of concrete strength development. This is by specifying different curing periods depending on the ratio between strength at 2 days and strength at 28 days.

The measures which can be taken at all stages to mitigate the effects of concreting in cold weather are summarized in Table 5.4.

Table 5.4 Summary of measures to reduce the adverse effects of cold weather

Stage	Measure
Production	Build up stockpiles Use warm water Use more reactive cementitious materials Increase concrete grade Reduce slump Shelter batching plant Insulate aggregates Heat aggregates Use accelerating admixtures
Transit	Minimize transit times Reduce truck revolutions Match production rate and delivery rate to placing rate
Placing and curing	Shuttering and reinforcement frost-free Subgrade thawed Insulated formwork Heated enclosure Provide protection to completed work

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PART 3

Properties of hardened concrete

6

Strength and failure of concrete under short-term, cyclic and sustained loading

John Newman

6.1 Deformation, fracture and failure

6.1.1 The structure of concrete

Concrete is a multiphase material containing cement paste (unhydrated and hydrated compounds), fluids, aggregates, discontinuities, etc. The overall mechanical and physical properties of such a composite system depend on the volume fractions and properties of the various constituents and the mechanisms of interaction, whether mechanical, physical or chemical, between the separate phases.

6.1.2 Stresses and strains

At a location in an element of material the generalized stress (strain) state in one, two or three dimensions comprising direct and shear stresses (strains) can be decomposed geometrically to a system of mutually perpendicular principal stresses (strains) σ_1, σ_2, σ_3

(ε_1, ε_2, ε_3) acting orthogonal to the principal planes on which the shear stresses (strains) are zero.

6.1.3 Deformation and failure theories

Since the eighteenth century many theories and models have been proposed to explain or predict the deformation, fracture and failure of composite systems. These are categorized in Table 6.1.

Table 6.1 Categories of theories and models for the behaviour of composite materials

Category	Theory/model	Remarks
1	'Classical' theories	Maximum principal stress or strain Maximum shear stress Maximum strain energy of distortion Maximum octahedral shear stress Internal friction theory Mohr theory, etc.
2	Mathematical models	Fundamental theory
3	Structural models	'Mixture' laws
4	Rheological models	Comprising elements for elasticity, plasticity and viscosity
5	Statistical models	Distributions of properties of elements
6	Physical models	Simulations of real material (Griffith theory, finite element models, etc.)

It is beyond the scope of this chapter to discuss all of these in detail but the following is a summary of the advantages and disadvantages of the various approaches, paticularly with regard to their use for concrete.

Category 1

These predict failure when a particular function of stress or strain reaches a critical value and have limited application to concrete.

Category 2

Such models are based on fundamental theories of physics and mechanics and allow the evaluation of stresses and strains within composite materials and for different geometrical arrangements of homogeneous materials. Inglis in 1913 considered an elliptical crack in an ideal elastic solid under uniform uniaxial tension applied at 90° to the major axis of the crack. For a major axis of $2b$ and a minor axis of $2c$ the radius of the crack tip is b^2/c and the maximum stress at the crack tip is $\sigma(1 + 2c/b)$ where σ is the stress applied to the boundary of the solid. The relationship between the radius of the crack tip (non-dimensionalized) and the intensification of stress at the crack tip $(1 + 2c/b)$ is shown in Figure 6.1 (Inglis, 1913).

Figure 6.1 demonstrates the large stresses developed around sharp cracks which is relevant to the cracks, flaws and voids present in concrete. The model developed by Goodier in 1933 predicts a stress intensification of 3 × the applied stress around a spherical

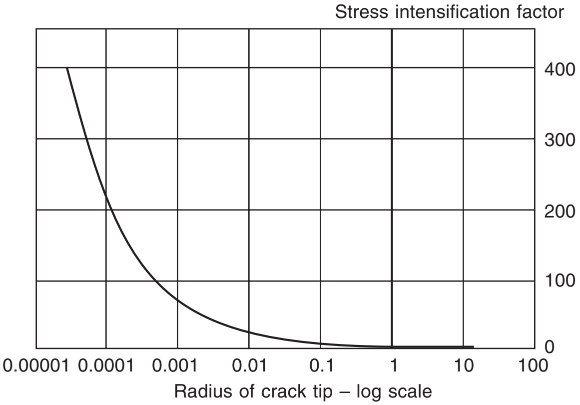

Figure 6.1 Relationship between radius of crack tip and stress at crack tip using Inglis's solution.

inclusion in an elastic material (as predicted for the Inglis model with $b = c$) and this is relevant to aggregate *particles* in hardened cement paste.

Category 3

Assuming concrete to be a two-phase material (matrix and aggregate) then its stiffness (elastic modulus E_c) can be calculated using models in which the matrix phase (E_m) and aggregate phase (E_a) are arranged in various configurations and proportions. All models described below assume that all phases are elastic and the simplest are the Dantu upper and lower bound models (Dantu, 1958) which give the highest and lowest values for E_c. The upper bound model, in which both phases experience the same strain is shown in Figure 6.2(a). For this arrangement, and assuming zero Poisson's ratio for the constituents, then $E_c = (E_m V_m + E_a V_a)$ where V_a and V_p are the volume fractions of the matrix and aggregate respectively.

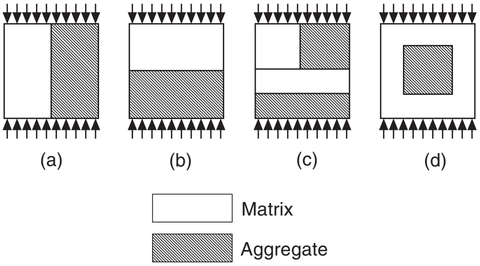

Figure 6.2 Two-phase models for concrete.

The Dantu lower bound model, in which both phases experience the same stress, is shown in Figure 6.2(b). Again, assuming zero Poisson's ratio for the constituents and the same notation then $1/E_c = (V_m/E_m + V_a/E_a)$.

Hansen (1968) suggested that the upper bound model is more relevant to hard aggregate particles in soft paste matrix with the lower bound model being more relevant for soft particles in a hard matrix. He also considered that, due to strain disturbances around aggregate particles in concrete, the actual E-values lie between upper and lower bounds.

The Hirsch–Dougill model combined the upper and lower bound models as shown in Figure 6.2(c). Assuming zero Poisson's ratio for the constituents, for a volume proportion of lower bound model of x then $1/E_c = x[1/(V_m E_m + V_a E_a)] + (1 - x)[V_m/E_m + V_a/E_a]$. However, both the lower bound and Hirsch/Dougill models predict $E_c = 0$ for $E_a = 0$ which is clearly incorrect. To overcome this problem Counto (1964) proposed the model in Figure 6.2(d). For this model $1/E_c = (1 - \sqrt{V_a})/E_m + 1/\{[(1 - \sqrt{V_a})/\sqrt{V_a}]E_m + E_a\}$. Assuming for a paste with a water/cement ratio of 0.45 that $E_p = 12.5$ kN/mm^2 and $E_a = 50$ kN/mm^2 the relationships between E_c and V_a for the various models are shown in Figure 6.3.

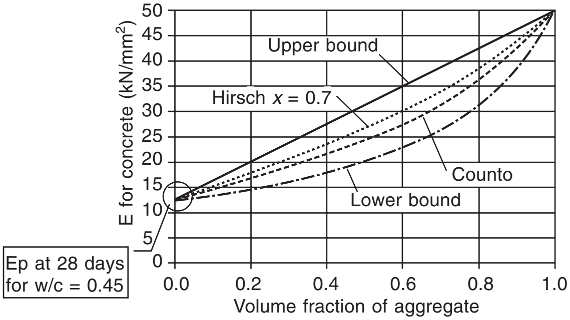

Figure 6.3 Relationships between elastic modulus of concrete and volume fraction of aggregate for various models assuming $E_p = 12.5$ and $E_a = 50$ kN/mm^2.

Category 4

These are based on the use of combinations of elements modelling stiffness (elastic springs), plasticity (yield stress) and viscosity (damper). For example, the combination of elements shown in Figure 6.4 will give the stress/strain relationship shown but only if the relevant constants for the various elements are assumed (Teeni and Staples, 1969).

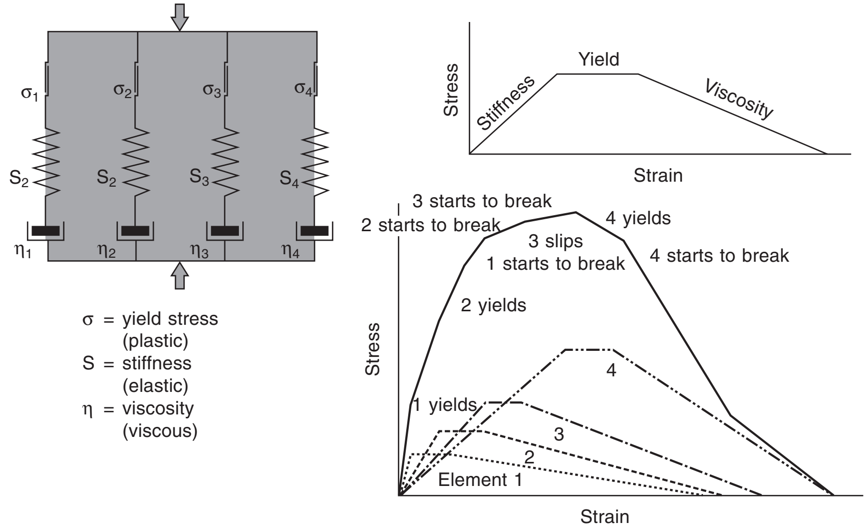

Figure 6.4 Rheological model for the stress/strain relationship of concrete.

Category 5

These are based on the statistical distribution of element properties and are of limited use for concrete since, as for Category 4 models, assumptions must be made before they can be applied.

Category 6

There is a large discrepancy between the theoretical strength of a brittle material (as calculated from the bonding forces between atoms) and its observed fracture strength. The theoretical estimate gives values of 10× to 1000× those determined experimentally. In 1920 Griffith (1920) proposed that this difference could be explained by the presence of microscopic flaws or cracks that always exist under normal conditions at the surface and within the interior of a body of material, with each crack tip acting as a stress-raiser, as discussed in Category 2 models above. During crack propagation there is a release of elastic strain energy (i.e. some of the energy stored as the material is elastically deformed) and new free surfaces are created which increases the surface energy of the system. A pre-existing crack can propagate when the elastic strain energy released during crack extension is equal to, or greater than, the surface energy of the newly formed crack surface. Using Inglis solution (Inglis, 1913) this indicates that, for a single elliptical crack in a thin plate subjected to uniform uniaxial tension, the critical stress for crack propagation is:

$$\sigma_{\mathrm{crit}} = (2\gamma_{\mathrm{w}}E/\pi c)^{0.5}$$

where γ_{w} = energy required to form a fracture surface
E = elastic modulus
$2c$ = crack length

However, the various parameters required by the Griffith criterion to determine the critical stress needed for crack propagation in real materials are difficult to evaluate. For a heterogeneous material such as concrete the task is impossible since (a) many cracks of different sizes, shapes and orientations either pre-exist or are formed under load and (b) the solid particles of aggregate etc. act both as crack arrestors and stress intensifiers. Nevertheless, the theory is useful as an aid to understanding the fracture and failure process and other models have been developed from the Griffith approach.

One such model has been developed by Hoek (1965) for rocks and by Newman (1973a) for concrete. Consider a thin plate of concrete or another brittle material with a pre-existing ‘closed’ crack aligned at an arbitrary angle and apply a uniform uniaxial (compressive) stress to the plate (Figure 6.5). When the stress is gradually increased to a certain stress level the stress and strain intensification near the crack tips causes small cracks to initiate to stabilize the system. This stage of the fracture process (Stage I) has been termed ‘stable fracture initiation’. Further increases in stress cause these cracks to propagate in a direction essentially parallel to the direction of the applied stress but when the stress is maintained constant propagation ceases. This stage (Stage II) has been termed ‘stable fracture propagation’ and ends at a critical stress level after which propagation continues even when the stress is maintained constant. This latter stage (Stage III) has been termed ‘unstable fracture propagation’ and ends with failure of the material. This process has been confirmed experimentally by Hoek and from finite element modelling by Newman (unpublished report).

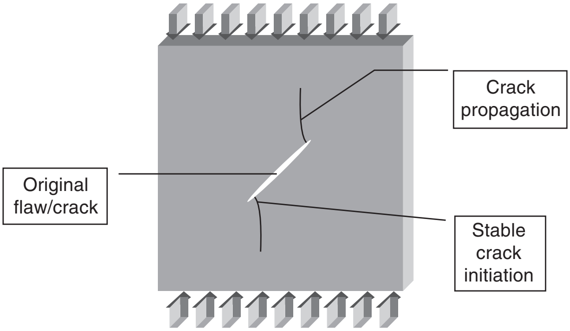

Figure 6.5 Initiation and propagation from a single crack in a brittle material.

Summary

Most concretes can be simplified as two-phase materials in which stiffer and stronger particles are embedded in a softer and weaker matrix. For such materials experience has shown that most of the above theories and models do not adequately explain or predict behaviour but can help in the understanding of the stresses induced within a composite material under load. The influence of the cracks and flaws within concrete can best be explained by Griffith-type physical models and that of aggregate by Goodier-type mathematical models. In view of these complexities, normal practice is to test concrete and fit the results to relationships which have been derived on the basis of a knowledge of fundamental material behaviour but which can be used by engineers.

6.1.4 Deformation of concrete

Stress–strain relationships for a typical concrete subjected to short-term monotonic uniaxial compressive loading to ultimate and beyond are shown in Figure 6.6. To allow the measurement of post-ultimate behaviour the load application was controlled to produce a constant rate of axial deformation (see below). Axial and lateral strains were measured using electrical resistance strain gauges (ERSG) and the overall axial deformation of the

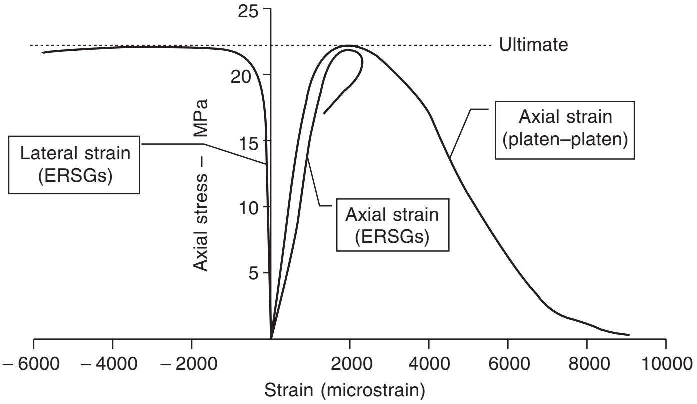

Figure 6.6 Typical relationships between stress and strain for concrete under uniaxial compression.

specimen using linearly variable displacement transducers (LVDT) measuring platen/platen deformation.

Although the axial and lateral relationships appear reasonably linear up to about 40–60 per cent of ultimate strength, they are not strictly linear. Thus, unlike steel (see Figure 6.7), concrete has no readily identifiable elastic limit and, for simplicity, engineers resort to tangent and secant 'elastic' moduli (*E* values, defined in the next section) for design purposes.

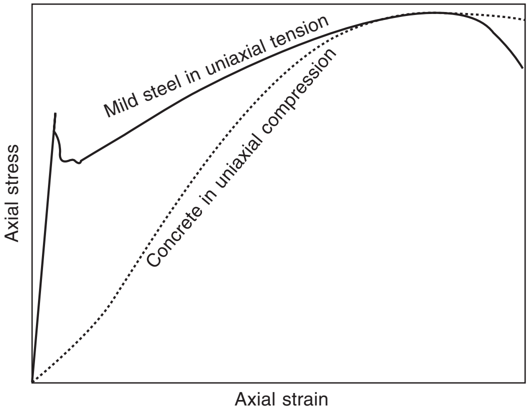

Figure 6.7 Comparison between stress–strain relationships for steel and concrete (not to scale).

Up to a stress of about 60 per cent of ultimate the lateral tensile strain (in a direction orthogonal to the applied compressive stress) is a near-constant proportion of the axial compressive strain. The constant of proportionality is termed Poisson's ratio (ν) and has a value of between 0.15 and 0.20 depending on the mix and its constituents. Above this stress the tensile strain increases at a much faster rate than the compressive strain and ν increases to above 0.5 (the value for rubber). Since the material is discontinuous at these stress levels the concept of Poisson's ratio is not valid. To add to this complexity, the deformations are not reversible (i.e. are inelastic) and are time-dependent.

6.1.5 Modulus of elasticity (*E*-value)

E-value is the ratio between stress and strain but, as discussed in section 6.1.4, the stress–strain relationship for concrete is non-linear and the material is not strictly elastic. Thus, the concept is not strictly applicable so for structural design and assessment three types of *E*-value are used, namely secant modulus, tangent modulus and initial tangent modulus (Figure 6.8).

The secant and tangent moduli can be determined from the stress–strain relationship from a short-term static test in which a specimen is loaded in uniaxial compression. A procedure for determining secant modulus is described in BS 1881: Part 121 in which a 150 dia. × 300 mm specimen is loaded to 33 per cent of ultimate stress and the slope of stress/strain relationship measured after conditioning to near linearity by progressively loading and unloading. The initial tangent modulus can be determined using ultrasonic (BS 1881: Part 203) and dynamic (BS 1881: Part 209) methods.

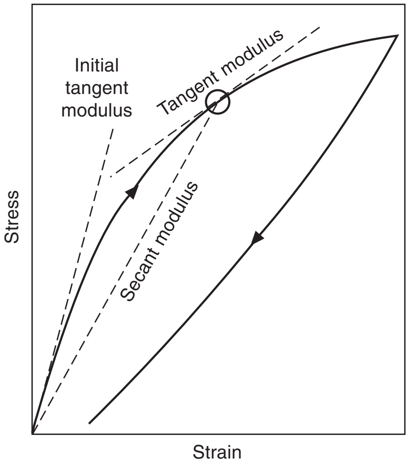

Figure 6.8 Diagram of stress–strain relationship for concrete under uniaxial compression.

The *E*-value of concrete is influenced generally by the same factors as strength and a relationship between strength and *E*-value for normal density concrete (BS DD ENV 1-1) is:

$$E_c = 9.5(f_{ck} + 8)0.33$$

where f_{ck} = characteristic cylinder strength (MPa)

For lightweight concrete *E*-value varies with both strength and density and the following relationship can be used (ACI 209R-92):

$$E_c = 4.3 \times 10^{-5} \times \rho^{1.5} \times f_{cyl}^{0.5}$$

The above relationships are approximate only and the *E*-value should be determined experimentally where its use is important.

6.1.6 Poisson's ratio

For uniaxial loading Poissons's ratio (ν) is the ratio between the strain in the loading direction and that in the unloaded direction. It is determined from static tests and, as discussed above, for most concretes ν lies within the range 0.15 to 0.20 for loading up to about 60 per cent ultimate (Anson and Newman, 1966).

6.1.7 Fracture and failure of concrete under uniaxial loading

Test conditions

The measured deformation and strength of a concrete are strongly dependent upon the specimen type and the testing conditions. Factors include specimen size and shape, moisture condition, temperature, applied stress or strain state, the strain actually induced in the specimens, and the loading technique. These are also considered in Chapter 2 of

Volume 4 (Testing and Quality) of this series. For a given type of loading (uniaxial, biaxial, triaxial, compression, tension) and concrete specimen (size, moisture condition, temperature) the deformational and failure behaviour will depend primarily on the method and rate of loading. The latter will be discussed in the following.

Rate of loading The measured strength increases with an increase in the rate of stressing (McHenry and Shideler, 1951) or straining (Bischoff and Perry, 1991) (Figure 6.9). This shows that at higher strain rates strength increases significantly to values approaching 2.5 × short-term static strength (equivalent to a strain rate of approximately 10^{-5}), probably due to a limiting rate of crack propagation. Lower strain rates result in strengths approaching 0.8 × static strength since cracks have more time to propagate and the mechanisms of creep predominate (see Chapter 7). This behaviour relates closely to that observed when concrete specimens are loaded in uniaxial compression to various proportions of ultimate and then the load is maintained (Rusch, 1960). Creep strains occur with time and by connecting the strain values occurring after similar periods of time for different applied stress levels a family of isochronous curves is obtained as shown in Figure 6.10.

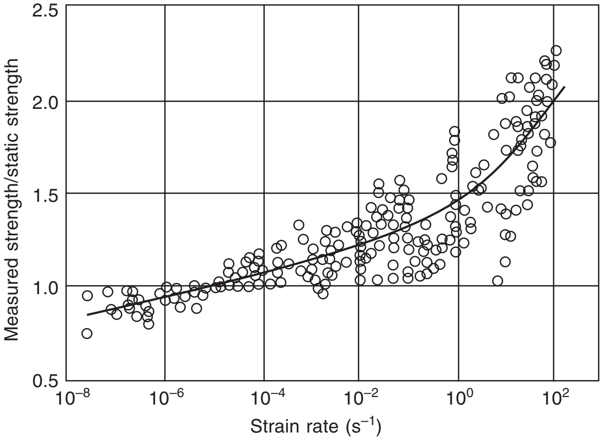

Figure 6.9 Relationship between strength and strain rate for concretes of various grades.

Figure 6.10 indicates that for stress levels below about 80 per cent of ultimate, the creep strain eventually reaches a limiting value. For stresses above this level the specimen always fails after a certain period of time.

Repeated loading Studies have shown that concrete fatigue strength is significantly influenced by a large number of variables including stress range, rate of loading, load history, stress reversal, rest period, stress gradient, material properties, etc. It has been suggested that concrete has no definite fatigue limit (i.e. the stress below which it will not fail under repeated load). However, a fatigue life of 10^7 cycles, adequate for most engineering purposes, can be achieved if the maximum stress is between 50 per cent and 60 per cent of the static strength (Murdock, 1900) (Figure 6.11).

The data in Figure 6.11 relates to load repeatedly applied from zero to the maximum level. However, it is known that the fatigue life of concrete is influenced by the range of applied or induced stress (Ople and Hulsbos, 1966) and such data can be presented in the form of a modified Goodman diagram (Figure 6.12).

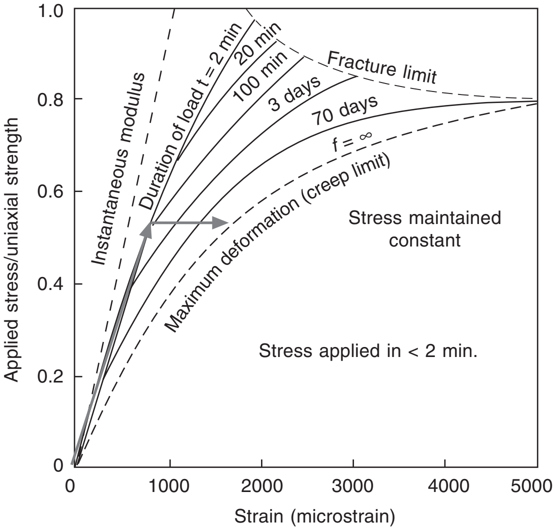

Figure 6.10 Isochronous relationships between deformation and stress under constant load (after Rusch, 1960).

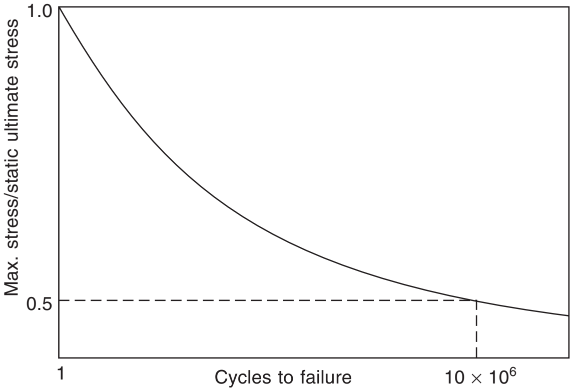

Figure 6.11 Diagram showing variation of load cycles to failure with maximum stress for repeated load applied from zero to maximum stress.

Figure 6.12 illustrates that the number of cycles to failure is reduced as the lower and/ or upper stress levels are increased. The broken lines in the diagram relate to assumed results for situations where the stress range is between compression and tension and for which there appear to be no published data.

The following relationship has been derived (Tepfers and Kutti, 1979) for data from S-N curves for concretes with densities above 1500 kg/m^3:

$$f_{c.max}/f_{c'} = 1 - 0.0685(1 - R)\log_{10}N$$

where $f_{c.max}$ = highest compressive stress of cyclic load
$f_{c'}$ = standard cylinder strength
R = ratio of lowest to highest compressive stress ($f_{c.min}/f_{c.max}$)
N = number of loading cycles to failure

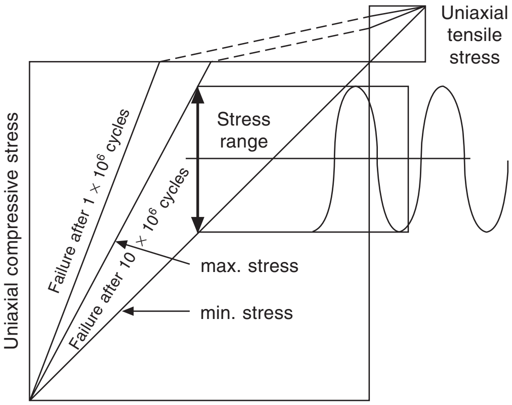

Figure 6.12 Modified Goodman diagram showing the number of cycles to failure for various cyclic uniaxial stress ranges (not to scale).

Load control Standard static tests on plain concrete usually require load to be applied in such a manner as to induce a defined and constant rate of increase of stress within the specimen. This normally results in sudden 'brittle' failure and offers no possibility of investigating the post-ultimate portion of the stress–strain relationship. If it is required to measure both the pre- and post-ultimate behaviour then the testing machine must be 'stiff' (see Chapter 2 of Volume 4 (Testing and Quality)) and capable of applying a controlled rate of strain within, or deformation on, the specimen. Such control could be exercised by using a screw-jack arrangement but for the normal hydraulically based systems it generally requires continuous measurement of strain or deformation which can be used to electronically control the loading rate to maintain the rate of strain or deformation. The loading system must be able to rapidly reduce the stress on the specimen if required. Figure 6.13 shows the typical type of stress–strain relationships for stress- and strain-controlled tests.

Figure 6.13 assumes that the ultimate stress and strain remains the same for both types of test but this has yet to be demonstrated.

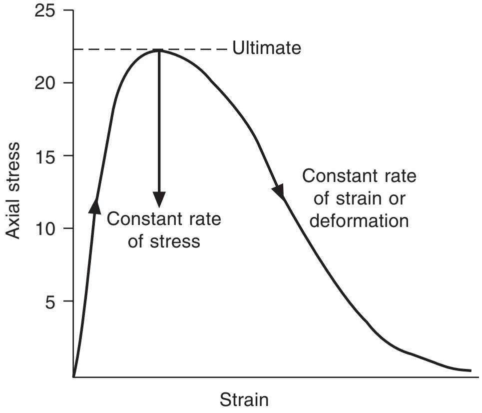

Figure 6.13 Axial stress–strain relationships for plain concrete under constant rate of uniaxial stress and strain.

Interaction between specimen and testing machine In order to establish deformation and failure criteria for concrete, it is necessary to carry out experimental investigations to assess behaviour under average stresses and strains at various stages up to, and beyond, the ultimate stress level. For the measured values to be considered as fundamental properties of concrete, they must be obtained from tests which have produced the required stress conditions in specimens essentially independent of any specimen/testing machine interaction.

It is often not realized that the actual state of stress induced in specimens and the resulting deformational and ultimate strength behaviour depend to a large extent on such characteristics as the longitudinal and lateral stiffness of the testing machine, the influence of machine platens and packings, the shape of the specimen and the method of applying load to the specimen. This applies particularly to test specimens loaded by mechanical means through rigid platens or grips and Chapter 2 of Volume 4 deals with such problems.

Concrete specimens for testing in compression are usually loaded uniaxially through steel machine platens. However, since (a) the value of ν/E for steel is less than that for concrete and (b) there is friction at the platen/specimen interface the resulting restraint induces a complex and indefinable three-dimensional state of triaxial compressive stress near the ends of the specimen. The influence of this restraint gradually decreases with distance from the ends of a specimen until at a distance approximately equal to the specimen width, a central zone exists which is subjected to the desired state of uniaxial compression. Many attempts have been made to overcome these effects by the introduction of various materials at the specimen/platen interface but if the material is too soft it squeezes out under pressure since its ν/E value is greater than that of concrete. This induces lateral tensile stresses which can cause a tensile mode of failure even though the specimen is loaded in compression. Tests carried out on nominal 100 × 100 mm prisms (Newman and Lachance, 1964) (Figure 6.14) show the effect on measured strength of rigid and soft interface materials.

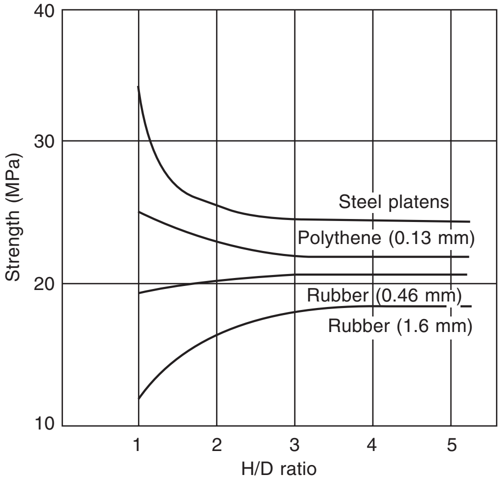

Figure 6.14 Variation of compressive strength with height/width ratio for 100 × 100 mm concrete prisms with different materials at the specimen/platen interface.

These effects will be discussed in more detail in section 6.2.2, but for the moment it can simply be stated that they can be minimized by matching the ν/E value of the platens to that of the material being tested and/or eliminating friction at the platen/specimen interface. However, this matching must be maintained throughout the entire test which is impossible. The simplest, and probably the most effective, method is to use a specimen with a height/width ratio large enough to produce a definable and desired state of stress within the central zone of the specimen but not too large to cause buckling. Figure 6.14 indicates that a height/width ratio of about 2.5 is suitable for uniaxial compressive tests. The relationships for 150 mm diameter cylinders of concrete of various strengths (of 150 dia. × 300 mm cylinders) (Murdock and Kesler, 1957) are shown in Figure 6.15 which indicates that the variation in measured strength increases with decreasing concrete strength.

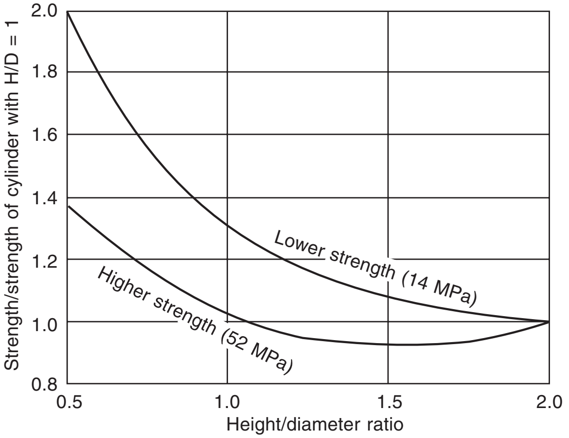

Figure 6.15 Variation of cylinder compressive strength with height/diameter ratio for 150 mm dia. cylinders of various strengths.

In addition, irregularities on the loaded surfaces of the loading platens and specimen can influence the induced stress state and these are discussed in Chapter 4. Even the heterogeneous nature of mortars and concretes can produce non-uniform compressive stress distribution near the ends of the specimen.

Moisture condition of specimen Strength testing of concrete is normally carried out using saturated specimens since this condition is more definable and achievable than any other. Compression specimens tested in unsaturated conditions exhibit lower strengths probably due to dilation and consequent loss of cohesion of the solid products of hydration. A strength loss of up to approximately 20 per cent can be caused by wetting and is reversible. However, the strength of specimens tested in direct tension or flexure decreases on drying due mainly to the formation of shrinkage cracks.

Size of specimen The measured strength of concrete specimens and its variability reduces as the specimen size increases. This may be due to the higher probability of critical cracks being present in larger volumes of material.

Temperature The measured compressive and tensile strength of concrete reduces as the temperature during the test increases. This is discussed in detail in Chapter 4.

Fracture and failure

Introduction Concrete specimens subjected to any state of stress can support loads of up to 40–60 per cent of ultimate without any apparent signs of distress. Below this level, any sustained load results in creep strain which is proportional to the applied stress and can be defined in terms of specific creep (i.e. creep strain per unit stress). Also the concrete is below the fatigue limit. As the load is increased above this level, soft but distinct noises of internal disruption can be heard until, at about 70–90 per cent of ultimate, small fissures or cracks appear on the surface. At this stage sustained loads result in eventual failure. Towards ultimate, cracks spread and interconnect until, at ultimate load and beyond, the specimens are increasingly disrupted and eventually fractured into a large number of separate pieces. The formation and propagation of small microscopic cracks 2–5 μm long (microcracks) have long been recognized as the causes of fracture and failure of concrete and the marked non-linearity of the stress–strain curve near and beyond ultimate.

Measurements of crack initiation and propagation The initiation and proliferation of microcracks produce irrecoverable changes in the internal structure of concrete, including the formation of voids and the dissipation of energy in the form of heat, mechanical vibrations and in the creation of new surfaces. The various methods which have been used to detect these structural changes in concrete and rock materials are as follows:

1. Visual, microscopic and X-ray examination of the surface of specimens during loading and of sections cut from specimens after loading
2. Photo-elastic coatings, Moiré interferometry, strain gauge and deflection readings during loading, involving surface or internal measurements
3. Direct measurements of volume changes during loading and of compressibility of previously loaded specimens
4. Measurements of ultrasonic pulse velocity through specimens under load
5. Measurement of acoustic emissions and absorption of acoustic energy
6. Measurements of electrical resistivity changes of saturated specimens under load

All these methods have confirmed the progressive process of crack initiation, multiplication and propagation to ultimate disruption and failure. However, the differing nature and degree of sensitivity of these techniques means that the effects of cracking are detected at different stress levels.

The stages of fracture in concrete The fracture processes in concrete depend primarily upon the applied state of stress and the internal structure of the specimen. There appear to be at least three stages in the cracking process. In describing the cracking mechanisms, it is important to differentiate between the mode of crack initiation and how this occurs at the microscopic level, and the subsequent paths of propagation and the eventual macroscopic crack pattern at the engineering level.

Although some discontinuities exist as a result of the compaction process of fresh concrete, the formation of small fissures or microcracks in concrete is due primarily to the strain and stress concentrations resulting from the incompatibility of the elastic moduli of the aggregate and paste components. Even before loading, intrinsic volume changes in concrete due to shrinkage or thermal movements can cause strain concentrations at the aggregate–paste interface.

Stage I Within this stage localized cracks are initiated at the microscopic level at isolated points throughout the specimen where the tensile strain concentration is the largest. Their formation relieves the strain concentration and equilibrium is soon restored, the accompanying energy changes and irrecoverable deformation being small. This shows that these cracks are stable and, at this load stage, do not propagate. Owing to the heterophase nature of concrete, there will be a distribution of strain concentrations throughout the specimen at a given applied load. As the applied load is increased during Stage I, there will be a more or less continuous multiplication process of stable crack initiation. The various methods of investigating the fracture process have been given above but the most direct and reproducible technique to estimate the end of this stage is by measuring the change in the ratio between the axial and lateral strains for small increments of stress (incremental strain ratio) (Figure 6.16).

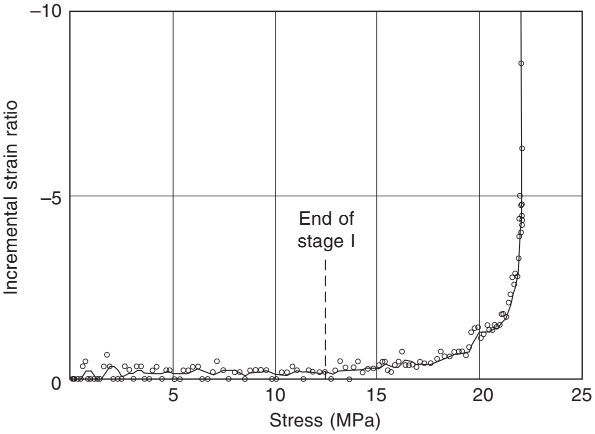

Figure 6.16 Typical relationship between incremental strain ratio and stress for a uniaxial compression test on concrete using a 100 mm dia. × 250 mm cylinder.

The significant increase in incremental strain ratio after the end of Stage I is caused by the propagation of essentially axial cracks which increases the lateral strain disproportionally to the increase in axial strain. A simple numerical procedure can identify the end of Stage I by taking into account the 'noise' in the strain measurements.

Stage II As the applied load is increased beyond Stage I, initially stable cracks begin to propagate. There will not be a clear distinction between Stages I and II since stable crack initiation is likely to overlap crack propagation and there will be gradual transition from one stage to another. This is illustrated diagrammatically in Figure 6.17 in which the distributions of crack initiation and propagation are assumed to be Gaussian.

During Stage II the crack system multiplies and propagates but in a slow stable manner in the sense that, if loading is stopped and the stress level remains constant propagation ceases. However, the degree of cracking eventually reaches a more severe level necessarily involving major structural changes which is easily detectable by acoustic emission, ultrasonic pulse velocity and volume change measurements. The stress level at which more severe cracking is detected will depend on the degree of sensitivity of the technique used. The most direct method of identifying the end of Stage II is by calculating

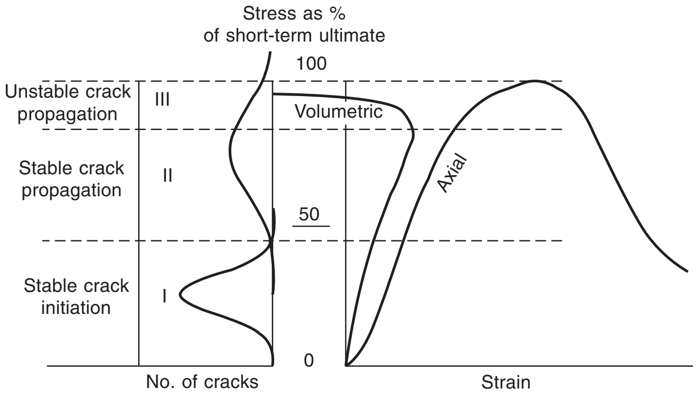

Figure 6.17 Stages of cracking in concrete.

the volume changes occurring during loading as the sum of the measured principal strains. The data in Figure 6.6 gives the typical volume change profile shown in Figure 6.18.

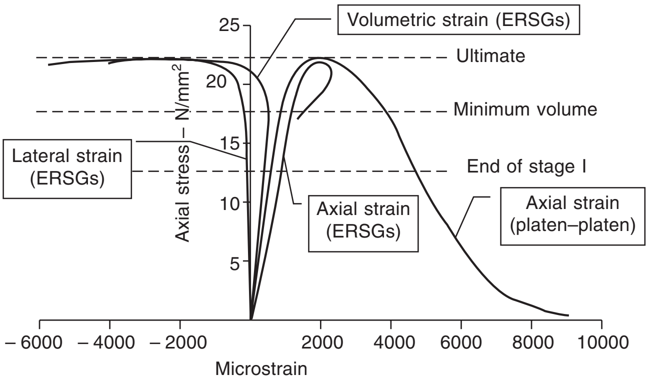

Figure 6.18 Typical stress–strain relationships and volumetric strain for concrete under uniaxial compressive stress.

It has been suggested by Newman (1973) that the end of Stage II can be taken to be the stress level at which the volume is a minimum. Although there is no fundamental reason why this level should represent the end of Stage II it does appear to coincide with the level at which behaviour becomes inherently unstable under sustained (Rusch, 1960) and repeated loads (Newman, 1973a).

The extent of the stable crack propagation stage will depend markedly upon the applied state of stress, being very short for 'brittle' fractures under predominantly tensile stress states and longer for more 'plastic' fractures under predominantly compressive states of stress.

Stage III This occurs when, under load, the crack system has developed to such a stage that it becomes unstable and the release of strain energy is sufficient to make the cracks self-propagate until complete disruption and failure occurs. Once Stage III is reached, failure will occur whether or not the stress is increased. This stage starts at about 70–90 per cent of ultimate stress and is reflected in an overall dilation of the structure as signified by a reversal in the volume change behaviour (Figure 6.18). As stated above, the load stage at which this occurs corresponds approximately to the long-term strength of concrete (Figure 6.10).

Mechanisms of cracking in concrete Figure 6.19 shows the complex strain (and hence stress) around aggregate particles embedded in a mortar matrix subjected to uniaxial compression state as revealed by a photoelastic coating (McCreath, 1968; Pigeon, 1969). Such idealized models and a study of the remnants after concrete failure. (Figure 6.20) indicates that the probable mechanism of crack propagation and ultimate failure in concrete under a uniaxial stress state is by cracks forming in the cement paste matrix due to the presence of microcracks and flaws and the stress and strain intensification around aggregate particles (McCreath *et al.,* 1964).

Figure 6.20 indicates that crack propagation paths may occur at the aggregate–paste interface, in the cement paste or mortar matrix or in the particles of aggregate. The position of crack initiation will depend upon the relative strength of the cohesive bonds and the local state of stress.

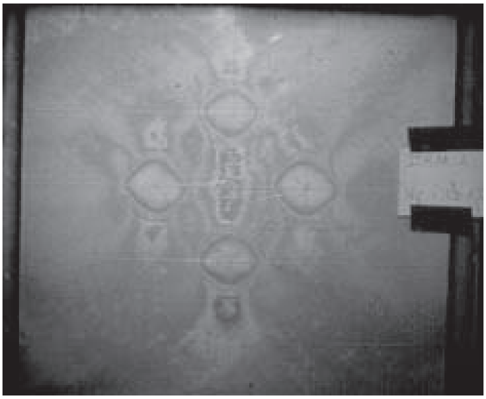

Figure 6.19 Strains around an array of aggregate particles embedded in a mortar matrix.

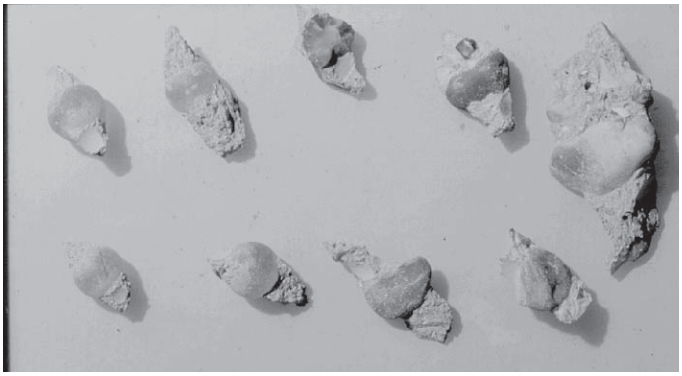

Figure 6.20 Aggregate particles after failure in uniaxial compression.

Under uniaxial compressive states of stress evidence suggests that, first, stable cracks are initiated in the mortar matrix parallel to the direction of applied compression. As the load is increased, the cracks multiply and extend in this same direction until, in the vicinity of mineral aggregate inclusions, the fracture path divides and travels around, rather than through, the hard particles. After final disruption and failure of a specimen, this fracture mechanism produces isolated particles of aggregate adhering to which are small 'cones' of mortar at each end aligned in the direction of maximum principal compressive stress. Figure 6.21 shows this process diagrammatically.

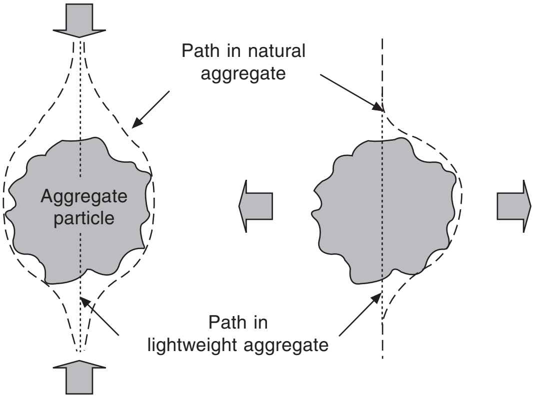

Figure 6.21 Fracture paths.

Under biaxial compressive states of stress, the alignment of the fracture path in both directions of principal compressive stress produces a crack pattern such that, at final disruption, the 'cones' of mortar on aggregate are extended to form complete 'haloes' around the particles.

The fracture process in compression is more stable than for uniaxial tension since the loaded area is less influenced by the cracking. Under uniaxial tensile states of stress, the fracture path runs essentially orthogonal to the maximum tensile stress. For strong, naturally occurring, dense aggregates the crack path tends to follow the aggregate–paste interface across which the cohesive forces are mostly physical in nature (van der Waals type). The tensile strength of these concretes is dependent, therefore, mainly on the aggregate–paste tensile bond strength. With weaker aggregates, including lightweight, the fracture path passes through the aggregate particles. For tensile states of stress, stable crack propagation (Stage II) is of short duration, since the cracks propagate very rapidly through the mortar matrix and around the aggregate–paste interface. The behaviour is much more unstable under uniaxial tension since cracking reduces the loaded area of the specimen whereas under uniaxial compression the cracks are aligned in the direction of loading and do not significantly influence the loaded area. It is for this reason that the tensile strength of concrete is much less than the compressive stength.

In summary, uniaxial, biaxial and especially, triaxial compressive stress states induce a local strain distribution in the mortar matrix which, after stable crack initiation in Stage I, results in prolonged stable crack propagation (Stage II) during which microcracks, initiated in the mortar matrix between hard coarse aggregate particles, begin to extend in the direction normal to the maximum tensile strain and parallel to the maximum compressive stress). At higher applied stress levels (Stage III) cracks begin to propagate around the aggregate particles as the aggregate–paste bond fails. The culmination of Stage III is failure of the element by cracking parallel to the maximum principal compressive stress (Figure 6.22) or orthogonal to the maximum principal tensile stress (Figure 6.23).

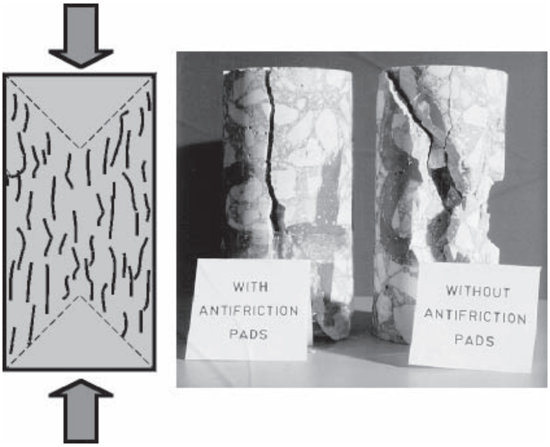

Figure 6.22 Failure modes in uniaxial compression.

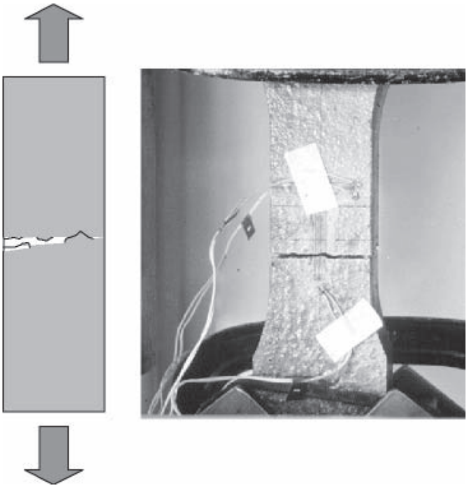

Figure 6.23 Failure mode in uniaxial tension.

Significance of fracture stages

Short-term loading Within Stage I the behaviour of concrete is inherently stable and the stresses in a structure would normally be in this zone. In Stage II cracks propagate in a stable manner in the direction of maximum principal compressive stress or orthogonal to maximum principal tensile stress. In Stage III behaviour is unstable and leads to failure.

Long-term loading If the loading is kept within Stages I or II it is reasonable to assume for design purposes that strains will increase to limiting values and that within Stage I the concept of specific creep can be used. Within Stage III the strains will increase to failure of the material.

Cyclic loading Within Stage I the volumetric strain is shown to decrease to stability while within Stage IIII it dilates to failure (Newman, 1973a). Within Stage II the typical behaviour is dilation to stability. Tests under slow cyclic uniaxial compression loading show that (a) damage becomes significant above a certain maximum stress level and (b) such damage increases with the degree of heterogeneity of the concrete mix. For design purposes it is advisable to ensure that the maximum stress remains within Stage I.

6.2 Behaviour of concrete under multiaxial stresses

6.2.1 Introduction

Most problems in structural engineering require the analysis of the response of a member or assembly of members to an imposed boundary condition which is usually specified in terms of a force (or stress), and sometimes as a displacement (or strain). Unless there is some experimental evidence to indicate the likely response of the particular geometrical arrangement to the given forces or displacements, then a more fundamental solution must be adopted using knowledge of the characteristics of the individual materials comprising the structure. For such an approach, the stress–strain relationships of the materials must be determined by subjecting representative elements of material to the whole range of boundary conditions which it is possible to encounter in the structure unless the materials are such that it is possible to deduce properties in one stress or strain state from those in another. Section 6.1 has been devoted primarily to uniaxial states of stress but the concepts of fracture discussed are applicable to more complex stress states. However, the principal problem to be addressed when measuring the deformational and strength characteristics of concrete under multiaxial stresses is the specimen/testing machine interaction.

Since boundary conditions are more commonly specified as applied loads, it is generally assumed that the properties of a material can be determined merely by applying a load in an arbitrary manner to the boundary of a representative element of the material and measuring the resulting behaviour. Such an approach, however, neglects the possible influence on the structural characteristics of the material of the type of stress or strain field induced in the element by the particular method in which the boundary load is applied. Examples of this can be seen when considering the way in which boundary loads are applied in standard tests used to determine material properties. As discussed above, this is normally accomplished by loading a specimen through a rigid steel platen arrangement which applies a near-uniform boundary displacement. Other possibilities exist, the most extreme being that of applying uniform boundary stress to an element. Of these two principal methods of applying a load to a material in a test, the engineer must use that which will provide the most relevant information for his purposes.

Having decided on the type of boundary condition which it is intended to apply to the material, it is necessary to assess the efficiency of any loading technique in attaining the desired state of stress or strain within the element under load for the results to be of practical value.

6.2.2 Transmission of load through different materials

To assess the load-carrying characteristics of a material it is necessary to transmit the load from the testing device through another material resting against, or attached to, the boundary of the specimen under test. The material adjacent to the specimen may form an intermediate layer between the specimen and the main loading device and, indeed, may form only part of a composite subsidiary loading unit.

As discussed above for uniaxial loading, it is the incompatibility between the various materials forming the loading device and the material of the specimen, together with the conditions at the various interfaces, which can give rise to either compressive or tensile restraint stresses on the boundary of, and hence within, the specimen. Obviously, these restraints are undesirable and should be minimized, but they cannot be avoided completely with practical loading systems. Since they will, inevitably, be present it is necessary to assess their magnitude and sign (i.e. compressive or tensile) before evaluating the results of tests to establish the fundamental properties of materials.

Uniform boundary displacement

Many problems are associated with devices developed for applying a state of uniform displacement to the boundary of a material since the load must essentially be applied to a specimen through platens which are rigid in the direction of loading. Under stress the lateral deformation of such platens will tend to be different from that of the specimen, with the actual extent of the difference being influenced by the frictional characteristics of the platen/specimen interface.

If no material is interposed between the solid platen and specimen the loading device can be placed in one of two categories, namely, platens which strain laterally less or more than the specimen. If full or partial frictional restraint is developed at the interface then the differential movement can cause restraint stresses to be transmitted across the boundary.

If an intermediate layer of material is interposed between the platen and specimen then the material can be classified in a similar manner to the cases above. The principal factors which may influence the restraint stresses transmitted to specimens under load through layered material systems are incompatibility of ν/E value between the various materials, frictional characteristics, thickness and modulus of rigidity of the intermediate material and surface irregularities. However, for concrete or mortar, ν and E change with stress level and this must be recognized when considering the restraint stresses, especially towards and beyond ultimate.

Over the years, various devices have been suggested for overcoming the problem of secondary restraints and one example is the ‘brush’ platen (the Type 3 device in Figure 6.24) where the individual bristles can transmit axial load from the testing machine to the specimen but are free to flex laterally to accommodate specimen deformation. However, lateral movement of the bristles can result in a non-uniform stress distribution over the loaded surface of the specimen.

Uniform boundary stress

The only practical way in which a state of uniform stress can be achieved on the boundary of a specimen of a heterogeneous material such as concrete is by the use of some form of hydraulic loading. If a pure hydrostatic stress is to be applied, then this can be relatively

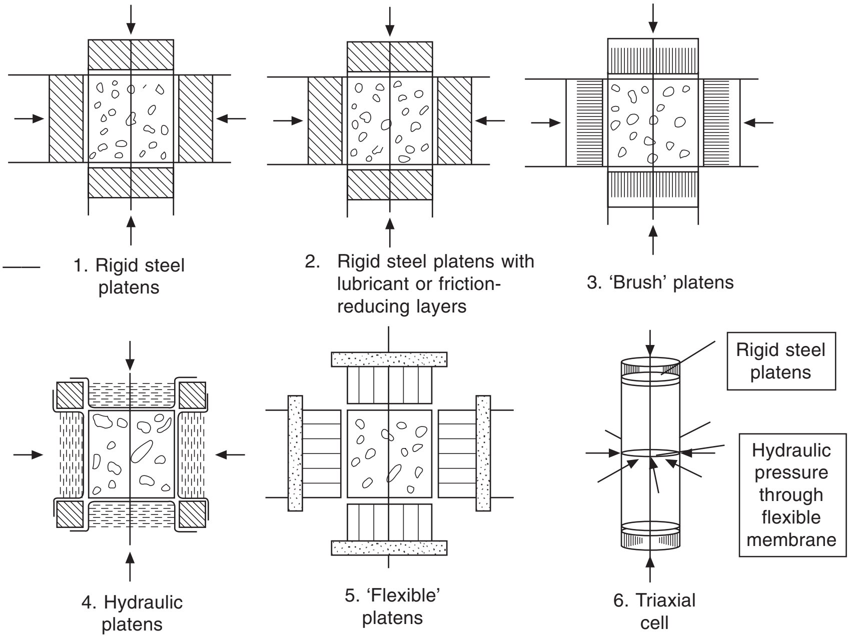

Figure 6.24 Devices for applying multiaxial loads.

simply attained by immersing the specimen, surrounded by a suitable impermeable membrane if desired, in a fluid under pressure (the Type 6 device in Figure 6.24).

For a state of equal biaxial compression a similar technique can be used, provided the specimen can be isolated from the effects of the fluid pressure in one of the three principal directions.

For stress states other than the above, the problem becomes more complex since the hydraulic pressure must be applied in each direction independently. Thus, in any given direction it will be necessary to contain the fluid such that the pressure is applied orthogonal to the face of the specimen, while at the same time eliminating any effects of extrusion between the surface of the specimen and the containment device, and the effects of intrusion into the specimen surface.

Several devices termed 'hydraulic platens' have been developed for the purpose of achieving independent and uniform states of boundary stress on specimens while minimizing lateral restraints. They vary in type from those which apply load directly through fluid-filled flexible membranes (the Type 4 device in Figure 6.24) to those which apply load from flexible membranes through bundles of steel prisms (the Type 5 device in Figure 6.24).

Evaluation of stress state in specimens

From analytical and experimental investigations the following principal conclusions can be drawn.

Loading directly through rigid platens Compressive lateral restraint stresses are induced at the platen/specimen interface which decrease towards the centre of the specimen and increase with the degree of roughness between the specimen and platen. As the height/

width ratio of a specimen is increased, the extent of the central zone of the specimen under a near-uniform lateral deformation increases while the lateral deformation within this zone increases. At a height/width ratio of about 2.5, the central third of the specimen is under near-uniform lateral strain. Failure initiates within this central zone and progresses towards the loaded ends where the restraint stresses inhibit fracture propagation. The failure load decreases with height/width ratios up to approximately 2.5 after which it remains essentially constant (see Figure 6.14). Considerable stress disturbances occur if the contact surface between the specimen and platen is not accurately plane.

Loading through 'soft' materials Tensile restraint stresses are induced at the interface between the intermediate material and the specimen due to extrusion of the insert material. These restraints decrease towards the centre of the specimen and increase with the value of ν/E of the insert. They tend to increase the lateral deformation of the specimen which is greatest near the loaded ends. The restraint stresses increase with thickness of the insert material and reduce with increasing height/width ratio as evidenced by the increase in measured failure load up to H/W = 2.5 (approximately), after which the failure load remains essentially constant at a value below that for rigid-platen loading (see Figure 6.14). Failure initiates within the end zones of the specimen and progresses towards the centre.

Loading through materials which 'bed-down' under load These materials considerably reduce the strain concentrations within specimens, particularly for cases where the loading surfaces of either the platen or the specimen are not accurately plane. The particular merit of such materials is that, although their E-value is very low, they do not induce significant restraint stresses in the specimen since their lateral movement under load is small. In fact the E-value is generally so small that, under the range of loads normally applied, these materials soon compress to a limiting deformation.

6.2.3 Choice of loading technique

Two fundamental possibilities exist for the application of load to specimens of a given material, namely, uniform boundary stress and uniform boundary displacement. The difference in response of a given volume of material to these states may be dependent on the heterogeneity of the material, in that more homogeneous materials (e.g. pastes and mortars) are likely to respond in a similar manner to both types of boundary condition, whereas relatively heterogeneous systems (e.g. concretes) exhibit large differences. For a material such as concrete, consisting of relatively stiff inclusions embedded in a softer matrix, a uniform boundary displacement will tend to concentrate the stress at the inclusions, whereas a uniform boundary stress will tend to induce higher strains in the matrix.

If the size of specimen is relatively small compared with the maximum particle size, which may be inevitable in practice, some differences in behaviour are to be expected and a choice must be made regarding which type of loading condition more nearly represents the loading situation for which the data are to be used.

However, two further points should be considered before a choice of loading technique is made. First, the ability of any given loading device to attain the desired conditions without imposing secondary restraint effects and, second, whether the specimen is to be subjected to a controlled rate of increase of load, deformation or strain. If controlled loading is used then it will not be possible to measure deformational response beyond the maximum stress level. Under conditions of controlled deformation or strain the response can be monitored throughout the entire loading history.

6.2.4 Behaviour of concrete under biaxial stress

Introduction

Most investigations of biaxial behaviour have attempted to apply uniform boundary displacement to specimens since the applied stress in each direction (σ_1 and σ_2) can be varied independently. Thus the entire stress field can be investigated with relative ease. Two main loading methods have been adopted, namely (a) rigid-platen devices with, or without, an intermediate layer interposed between them and the specimen, or (b) non-rigid platens. Some investigators have adopted solid cylinders tested in a hydraulic cell arrangement, with the component of pressure in the axial direction eliminated and with the specimen sealed against ingress of fluid. Using this technique it has been found that the strength of concretes under equal biaxial compression varies from 1.10 to about 1.3 × uniaxial depending on the apparatus used and the type of concrete tested.

Strength envelopes in compression/compression

Typical failure envelopes obtained under uniform boundary displacement are shown in Figure 6.25. The dotted lines are for compression/tension stress states and the solid lines for compression/compression. No data are available for the tension/tension region due to the difficulty of obtaining definable stress states. Zone 1 type failures are evident over most of the C/T region, with Zone 2 failures near uniaxial stress. Elsewhere Zone 3 failures are produced with cracking orthogonal to the unloaded direction which is the direction of zero stress (minimum compression).

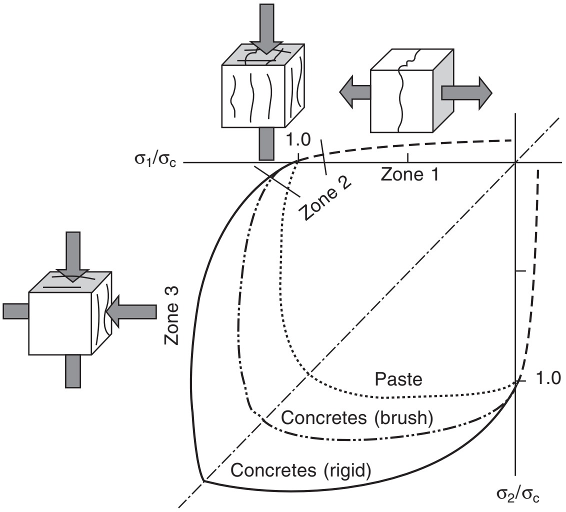

Figure 6.25 Typical biaxial strength envelopes for paste and concrete tested under uniform rate of displacement showing failure modes.

The following principal conclusions can be drawn from strength results obtained in a number of investigations:

- For rigid or non-rigid platen loading the maximum ratio between biaxial and uniaxial strength is not attained under equal biaxial compression. In fact, in most cases, the maximum value is given under a stress ratio of approximately 2:1.
- Cement pastes under either rigid or non-rigid platen loading exhibit maximum biaxial strength ratios of between 1.07 and 1.12 with equal biaxial ratios between 0.95 and 1.02.
- Mortars and concretes give considerably higher maximum biaxial and equal triaxial strength ratios than pastes for all types of rigid and non-rigid platen loading, the ratio increasing slightly with maximum size of aggregate.
- Non-rigid platens, and rigid platens with a friction-reducing intermediate layer, give biaxial strength ratios lower than those obtained using rigid platens without lubricant.
- For mixes with similar aggregate contents the envelopes within the C/T region appear to be independent of concrete strength.

Deformational behaviour

As for strength envelopes the measured stress–strain relationships depend on the loading method used to achieve a given stress path. In addition, the relationships may also be affected by the deformation measuring technique employed. As an illustration of the trends observed under biaxial loading Figure 6.26 shows typical relationships under equal biaxial stress ($\sigma_1 = \sigma_2$, $\sigma_3 = 0$) for a concrete.

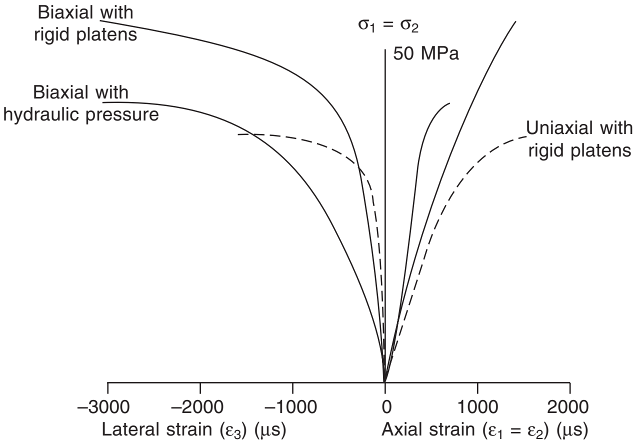

Figure 6.26 Typical stress–strain relationships for a concrete under equal biaxial compressive stress using various loading devices.

Failure modes

Figure 6.27 indicates the mode of failure exhibited by a mortar specimen under biaxial compression using rigid platens. It is evident that cracking does, in general, propagate in a direction orthogonal to the maximum principal tensile stress but the influence of compression restraint can be seen. There is a special case in the vicinity of uniaxial compression where the planes of failure can be aligned in any direction orthogonal to the load (Figure 6.25).

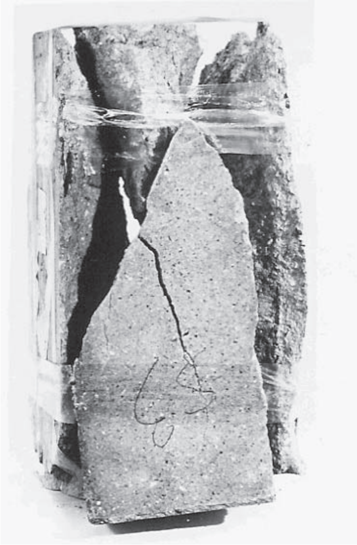

Figure 6.27 Specimen of mortar after failure under equal biaxial compression using rigid platens.

6.2.5 Behaviour of concrete under triaxial stress

Introduction

Over the years, a number of investigations have been carried out to study the characteristics of concrete under multiaxial stress. These have concentrated on the measurement of ultimate strength with deformational behaviour being neglected probably due to the difficulty of accurate measurement. An analysis of data obtained since 1928 from tests under states of stress $\sigma_1 > \sigma_2 = \sigma_3$ with all stresses compressive, performed on concrete with a maximum aggregate size of 20 mm using a hydraulic cell arrangement (device Type 6 in Figure 6.24) are shown in Figure 6.28. The data have been normalized with respect to uniaxial strength to minimize the effects of testing techniques and have been fitted to a normalized parabolic relationship $\sigma_{\mathrm{axial}} / \sigma_c = [A(\sigma_{\mathrm{conf}}/\sigma_c) + 1]^B$ where σ_{axial} is the axial stress at ultimate, σ_{conf} is the hydraulic confining pressure and σ_c is the axial stress at ultimate for zero confining pressure (uniaxial strength). *A* and *B* are constants which depend on the concrete mix characteristics.

In drawing conclusions from this analysis it must be emphasized that data has been extracted from a large number of sources. A variety of types of loading apparatus and test procedures have been used and the mix parameters and curing procedures have, in many cases, been varied simultaneously. With these restrictions in mind it can be concluded that lean concretes, with correspondingly high water/cement ratios, give larger strength increases with increasing confining pressure than rich mixes with low water/cement ratios. For comparison, Figure 6.28 also shows the relationship between axial stress and confining pressure proposed in 1928 (Richart, *et al.*, 1928) namely:

$$\sigma_1/\sigma_c = 1 + 4(\sigma_3/\sigma_c)$$

where σ_1 = axial stress
σ_3 = confining pressure
σ_c = uniaxial cylinder strength

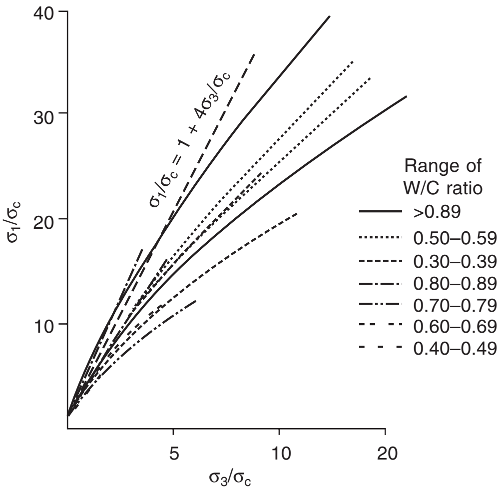

Figure 6.28 Failure envelopes for concrete with 20 mm maximum aggregate size tested under triaxial compression.

Following the above, a comprehensive programme of tests was carried out at Imperial College London to investigate the deformational characteristics of concrete under triaxial stress with particular reference to the internal structural changes which the material undergoes. The principal results of this programme, which was intended to establish criteria for concrete strength and deformation, are described in the following.

Tests conducted at Imperial College London (Newman, 1973a)

Failure envelopes The tests have been performed using a hydraulic triaxial cell capable of applying the following states of stress following any axisymmetric stress path in stress space:

$\sigma_1 > \sigma_2 = \sigma_3$ with all stresses compressive (triaxial 'compression')
$\sigma_2 = \sigma_3 > \sigma_1$ with all stresses compressive (triaxial 'extension')
$\sigma_2 = \sigma_3 > \sigma_1$ with σ_2 and σ_3 compressive and σ_1 tensile (compression/compression/tension)

A hydrostatic cell pressure (confining pressure) was first applied and then the axial (deviator) stress either increased or decreased to failure. Figure 6.29 shows the stress paths and the form of failure envelopes in stress space. Also shown are typical forms of stress–strain relationships exhibited from the start of deviator loading.

It is usual to describe all tests in which the axial stress is increased in compression to failure as 'compression' tests, since failure occurs in the compression mode. Tests in which the axial stress is decreased to failure are termed 'extension' tests with failure occurring in the tensile mode even if the resultant axial stress is compressive. A special case of this latter type of test is when the specimen is suitably shaped, usually by 'waisting', such that the confining pressure can induce a tensile stress in the axial direction. This test is termed a compression/compression/tension (C/C/T) test.

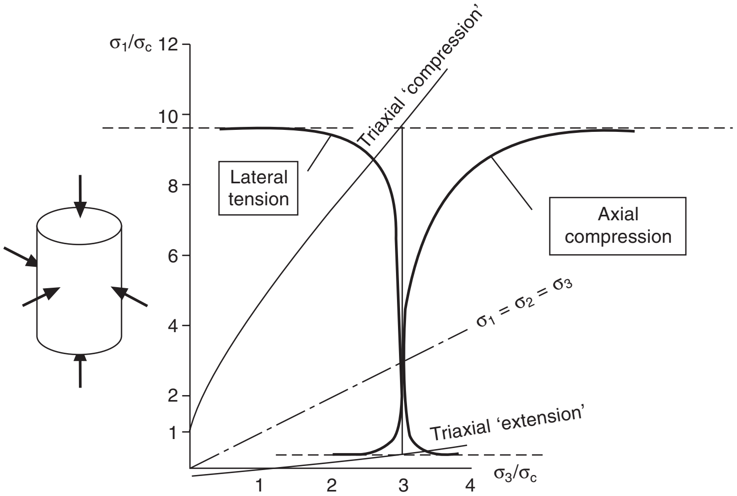

Figure 6.29 Typical failure envelopes and stress–strain relationships for concrete under triaxial compression and extension.

Stress–strain relationships Figure 6.30 shows the variation of axial and lateral strains for a concrete mix. Each relationship covers the entire loading regime, including hydrostatic as well as deviatoric, for a particular maximum level of confining pressure applied.

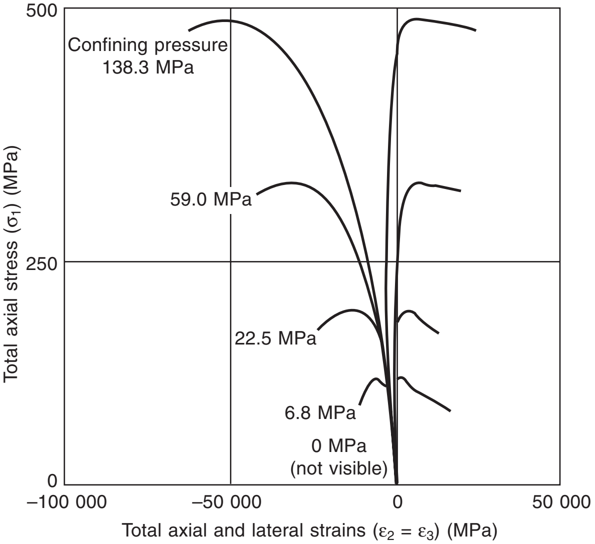

Figure 6.30 Stress–strain relationships for a concrete under triaxial 'compression' with various confining pressures.

Figure 6.30 shows that, for a given confining pressure the axial stress–axial strain relationship for 'compression' tests gradually increases in curvature with increasing stress. The corresponding axial stress–lateral strain relationship, however, exhibits marked increases in curvature only as ultimate failure is approached. This phenomenon can be attributed to the process of cracking and the alignment of the cracks for this type of loading. Similar behaviour is observed under 'extension' and C/C/T stress states. Concrete also exhibits a transition from relatively 'brittle' to more 'ductile' behaviour as the maximum confining pressure is increased. At low confining pressures, the slope of both the axial and lateral stress–strain curves reverses distinctly at ultimate. At higher confining pressures, this reversal is less obvious.

Stage I behaviour As discussed previously, studies of the fracture mechanisms of brittle materials (e.g. rocks and concrete) indicate that such materials fail by crack extension in a direction parallel to the maximum principal compressive stress, or in other words, orthogonal to the maximum principal tensile stress. For uniaxial and biaxial compression and tension this behaviour can be easily identified, as it can for triaxial 'extension' tests or 'compression' tests. However, for triaxial 'compression' at high confining pressures the mode of failure changes from 'brittle' to 'ductile' as evidenced by the stress–strain relationships and no clear fracture mechanism is visible since the cracks are extremely small and localized.

As in the case of uniaxial and biaxial stress states an indication of the level at which cracking becomes significant can be identified by examining the relationships between the strains in the directions of maximum and minimum principal stress, since such cracking will influence the strain in the direction of the minimum compressive stress more than in the direction of the maximum compressive stress. Therefore, as the cracks extend, the ratio of the strain in the direction of minimum compressive stress to the strain in the direction of maximum compressive stress should increase numerically. A typical variation of this relationship for a concrete under triaxial 'compression' is shown in Figure 6.31. Only a small portion of the relationship is shown in each case up to and just

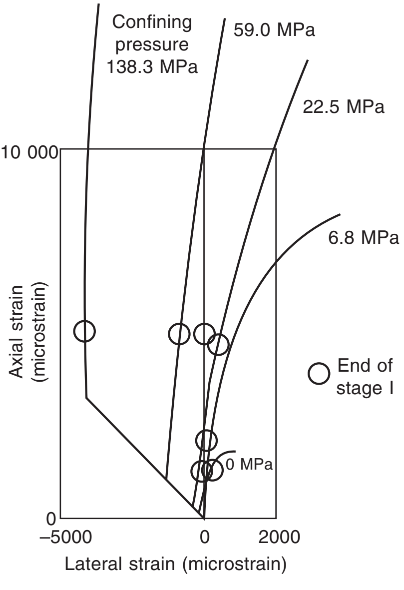

Figure 6.31 Axial–lateral strain relationships for a concrete under triaxial 'compression' with various confining pressures.

beyond the departure from linearity (end of Stage I) which is also indicated on the relationships. The end of Stage I marks the start of the significant influence of cracking on the deformational behaviour of the material. For clarity not all strain relationships are shown and some points representing the end of Stage I behaviour are for the omitted axial–lateral strain relationships.

Figure 6.31, together with information from traixial 'extension' tests indicates that the area bounded by the end of Stage I will be essentially 'closed' in stress space.

Stage II behaviour As discussed above, another clearly defined and more advanced level of the cracking process (Stage II) under stress states is that defined by the stage at which the volume change relationships attain a minimum value before dilating to failure. Figure 6.32 shows typical relationships between the volume changes and total axial stress at various confining pressures.

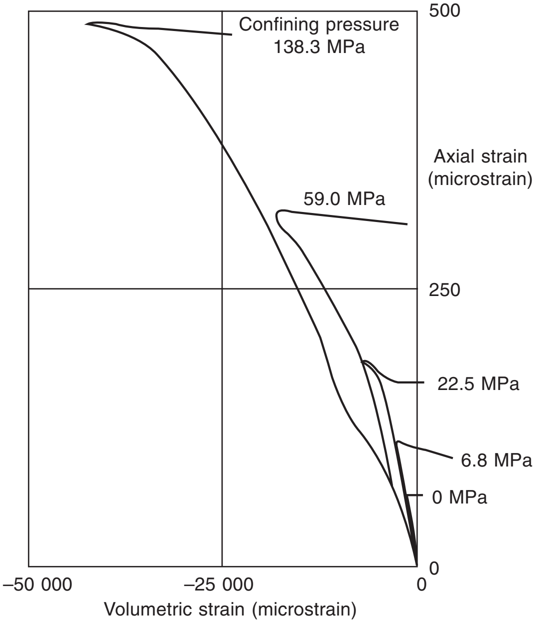

Figure 6.32 Volumetric strain–axial stress relationships for a concrete under triaxial 'compression' with various confining pressures.

Figure 6.32 shows a non-linear volume decrease for an increase in hydrostatic pressure followed by a further decrease in volume on application of deviator stress. Mixes of a given free water/cement ratio containing a higher volume fraction of aggregate give rise to smaller volume changes at a given level of confining pressure. Greater volume changes are given for mixes with a higher water/cement ratio. Reversals of the volume change occur at a stress approaching ultimate, the point at which the volume attains a minimum value marking the end of Stage II.

Failure modes The cracking of concrete under multiaxial stress results in the complete disruption of specimens at, or beyond, the ultimate stress level in a manner which is dependent on the type of stress state imposed. For the cases of uniaxial and biaxial stress states the failure modes are covered above. For triaxial ‘compression’, where each principal stress is maintained compressive or increased in compression, under low values of confining pressure failure can be observed to occur along planes parallel to the maximum compressive stress in a similar manner to uniaxial compression. For larger values of confining pressure the failure planes become less distinct as the behaviour becomes more ductile. If a conventional hydraulic triaxial cell is used to apply loads then this behaviour gives rise to the ‘bulged’ profiles of specimens under high confining pressures allied to the considerable shortening in the axial direction (Figure 6.33).

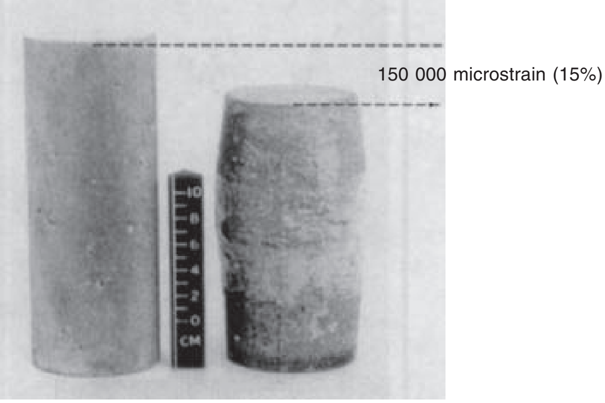

Figure 6.33 Typical failure mode for concrete under triaxial ‘compression’ at high confining pressure.

For the triaxial ‘extension’ (or compression/compression/tension) types of test failure occurs abruptly on planes orthogonal to the minimum compressive stress.

Stresses at initial cracking, final breakdown and ultimate Figure 6.34 shows envelopes denoting the variation of axial stress with confining pressure for mortar and concrete mixes at the end of Stage II. All relationships in Figure 6.34 show the same trend namely,

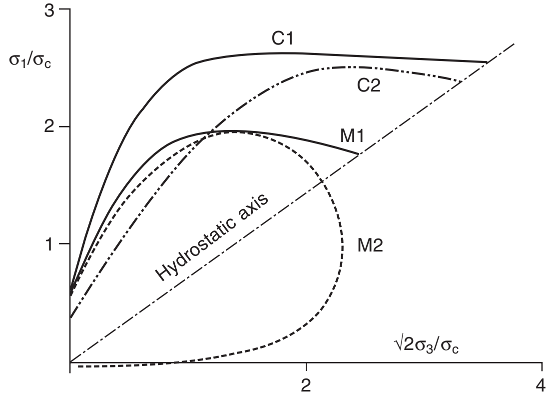

Figure 6.34 Variation of axial stress with confining pressure at the end of Stage II for mortars and concretes under triaxial ‘compression’ and ‘extension’.

that, as the confining pressure of the test is increased for a given mix the envelope tends towards intersection with the hydrostatic axis such that a closed portion of stress-space is defined. Such behaviour may be due to both local stress or strain concentrations in the heterogeneous material under hydrostatic compression and collapse of the cement paste phase. This is supported by evidence of failure mechanisms found for concretes under hydrostatic pressure and failure envelopes for pastes, lightweight concretes and other porous materials.

This behaviour is to be contrasted with the ever-increasing stress at ultimate (and the end of Stage II) with increasing compressive confining pressure which gives rise to open-ended envelopes, at least within that portion of stress space covered by the tests (Figure 6.35).

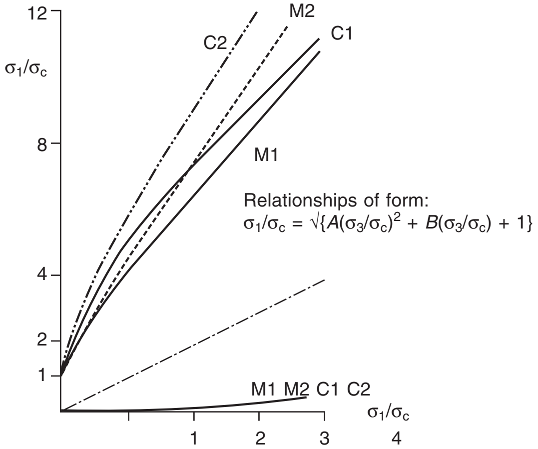

Figure 6.35 Variation of axial stress with confining pressure at the ultimate for mortars and concretes under triaxial 'compression' and 'extension'.

Design criteria

Data from tests on various concretes under multiaxial stress states has been analysed to derive simple lower bound linear failure envelopes (Hobbs *et al.,* 1977) for compatibility with the limit state approach in BS 8110. In deriving the envelopes from variable data it has been assumed that the intermediate principal stress (σ_2) has no effect on failure stress. The bending strength in a member is $0.67f_{cu}$ and the design strength is f_{cu}/γ_m where $\gamma_m = 1.5$ for ultimate. The serviceability limit state (i.e. the stress level that can be maintained without distress under sustained or fluctuating load) is 0.7 × ultimate for σ_1. It is recommended that the lowest credible value of σ_3 throughout the life of the structure be used for design.

For example, for concrete made with natural aggregate under a stress state $\sigma_1 \geq \sigma_2 \geq \sigma_3 > 0$ the lower bound relationship for failure is:

$$\sigma_{1f} = 0.67f_{cu} + 3\sigma_3$$

$\gamma_m = 1.5$ allows for quality of concrete in structure and is applied to f_{cu} only so for the ultimate limit state of collapse:

$$\sigma_{uf} < 0.45f_{cu} + 3\sigma_3$$

For serviceability limit state $\sigma_{1s} = 0.7\sigma_{1u}$, thus:

$$\sigma_{1s} = 0.3f_{cu} + 2\sigma_3$$

Expressions are also given for lightweight aggregate concrete, damaged concrete and concrete subject to pore pressure.

More design information and rigorous methods based on later and less variable data are available (Kotsovos and Newman, 1977, 1979, 1980; Gerstle *et al.,* 1980; Newman and Newman, 1978).

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7

Elasticity, shrinkage, creep and thermal movement

Jeff Brooks

7.1 Learning objectives

The main learning objectives of this chapter are to explain and describe the following features appertaining to concrete:

- Principal causes and factors affecting elastic, creep, shrinkage and thermal movements
- Typical ranges of strain occurring in practice
- Mechanisms of shrinkage and creep
- Measurement of shrinkage and creep
- Effects of movements on concrete in service
- Practical prediction of movements

7.2 Introduction

Although the elastic and thermal deformation behaviour of concrete have been known for some time, it is only relatively recently that the importance of creep and drying shrinkage have been recognized. It was probably at the beginning of the twentieth century when Hatt (1907) first reported increased, non-elastic deflections of reinforced concrete beams under a sustained load. Since that time, there have been many hundreds of research publications and design documents, dealing with the subject, such as ACI (1973), BS 1881: Part 2 (1985), CEB-FIP (1990) and RILEM (1995).

When concrete is subjected to external stress, there is an initial (elastic) strain followed by a slow time-dependent increase in strain (creep). There can be other time-dependent moisture movement strains that are not associated with external stress. For example, drying shrinkage occurs in most structural elements stored at usual temperature and relative humidity. To calculate the deformation and deflection of structural members in order to check their serviceability, we need to know the relation between stress and strain. Too much long-term deflection or cracking due to induced tensile stress should be avoided in order to provide adequate durability.

Although this chapter concentrates on creep and drying shrinkage, there are other types of movement that contribute to the total deformation or stress induced by restraint to movement. Thermal movement can be significant on a daily as well as a seasonal basis. It is equal to the product of the coefficient of thermal expansion (approx. 10×10^{-6} per °C) and the change in temperature in °C. Autogenous shrinkage is small for normal strength concrete but not for high-strength or high-performance concrete. Swelling occurs for saturated concrete and can be significant for lightweight concrete.

7.3 Elasticity

The definition of pure elasticity is that strains appear and disappear immediately on application and removal of load. Examples of materials behaving in that manner are steel (linear) and timber (non-linear). Other materials behave in a non-elastic manner, e.g. glass (linear) and concrete (non-linear). It should be emphasized the concrete only behaves that way when it is young or loaded for the first time; as seen in Figure 7.1, there are possible ways of obtaining a modulus of elasticity. The shape of the stress–strain curve depends to some extent on the rate of application of stress, application of load quickly reducing the curvature. The deviation from linearity is also due to microcracking at the interface of aggregate and cement paste (*transition zone*). Because of these effects, the distinction between elasticity and creep is not clearly defined and, for practical purposes, the deformation during application is considered elastic and the subsequent increases are regarded as creep. The slope of the stress–strain curve at the stress considered is the *secant modulus of elasticity*.

For estimating the total deformation in design calculations, the static modulus of elasticity is often used as an approximation to the secant modulus, its method of determination being specified in BS 1881: Part 121: 1983. Here, the effects of creep are reduced by loading the specimen three times, the static modulus being determined from the slope of the now-linear stress–strain curve. Generally, the stronger the concrete the greater the static modulus of elasticity. However, it is usual to estimate the modulus from one of the several empirical relationships between static modulus (E_c in GPa) and compressive strength (f_{cu} in MPa), e.g. BS 8110: Part 2: 1985:

$$E_c = 9.1 f_{cu}^{0.33} \tag{7.1}$$

for normal weight concrete of density $\cong$ 2400 kg/m^3, and

$$E_c = 1.7\rho^2 f_{cu}^{0.33} \times 10^{-6} \tag{7.2}$$

for lightweight concrete of density (ρ) between 1400–2400 kg/m^3.

Other countries use different expressions that are based on cylinder strength, which is approximately 0.8 × cube strength for normal strength concrete. For estimating the strains at very low stresses, the dynamic modulus of elasticity is used, as determined by the method in BS 1881: Part 5: 1970. The dynamic modulus corresponds to the initial tangent modulus in Figure 7.1, and Neville (1995) quotes empirical relationships that exist between the static and dynamic moduli of elasticity.

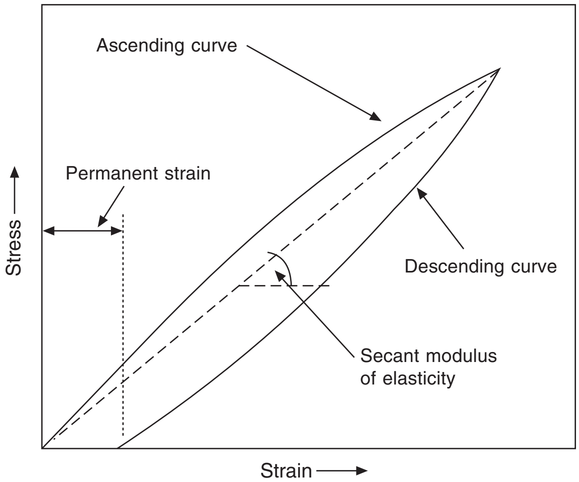

Figure 7.1 Magnified stress–strain curve for concrete loaded for the first time.

7.4 Shrinkage

7.4.1 Structure of cement paste

Before discussing shrinkage and creep, it is pertinent to outline the structure of the 'seat' of those long-term movements and, in particular the role of water. Figure 7.2 shows the components from concrete observed at the engineering level to the C–S–H at the submicroscopic level. Concrete is described as a multi-phase composite material consisting of coarse aggregate particles embedded in a matrix of mortar, the mortar consisting of grains of unhydrated cement embedded in a matrix of the products of hydration of cement. These products are a cement gel or C–S–H, with a system of water-filled or empty capillary pores. At the submicoscopic level, the C–S–H is a mixture of mostly crumpled sheets and foils, which form a continuous matrix with the water-filled interstitial voids (gel pores). The C–S–H sheets have a thickness of about 3 Nm and the gel pores have a diameter between 0 and 4 Nm, which means that only a few molecules of water can be absorbed on a solid surface. Gel pores occupy about 28 per cent of the total volume of the cement gel or C–S–H, and are much smaller than capillary pores (10^{-3} mm).

Water is held in the hydrated cement paste in varying roles. At one extreme there is free water, which is beyond the surface forces of the paste while, at the other extreme,

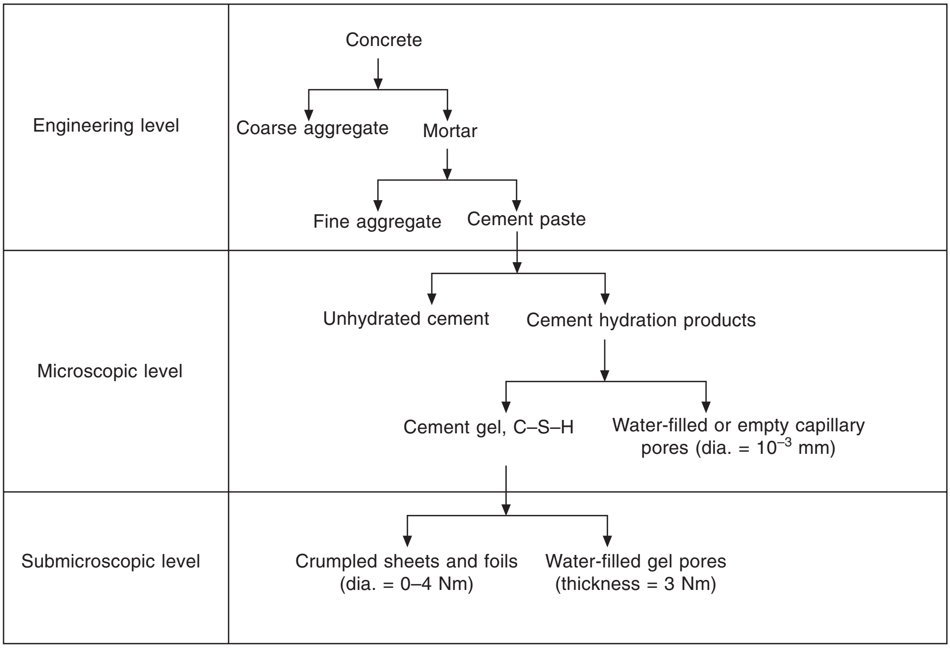

Figure 7.2 Structure of hardened cement paste.

there is chemically combined water forming a definitive part of the hydrated compounds. Between these two extremes there is gel water consisting of adsorbed water held by the surface (van Der Waals) forces of the gel particles, interlayer water (zeolitic water), which is held between the C–S–H sheets, and lattice water, which is water of crystallization not chemically combined.

The different types of water are difficult to determine quantitatively and in practice water is divided into evaporable water, as determined by the loss in weight on heating to 105°C and non-evaporable water. The latter is deduced from the original water content but, if unknown, it can be determined by the weight loss on heating to 1000°C. The evaporable water includes the free water and some of the more loosely-held adsorbed water.

7.4.2 Mechanism of shrinkage

In a drying environment where a relative humidity gradient exists between the concrete and surrounding air, moisture (free water) is initially lost from the larger capillaries and little or no change in volume or shrinkage occurs. However, this creates an internal humidity gradient so that to maintain hygral equilibrium adsorbed water is transferred from the gel pores and, in turn, interlayer water, may be transferred to the larger capillaries. The process results in a reduction in volume of the C–S–H caused by induced balancing compression in the C–S–H solid skeleton by the capillary tension set up by the increasing curvature of the capillary menisci. This is known as the capillary tension theory. At lower relative humidity, the change in surface energy of the C–S–H as firmly held adsorbed water molecules are removed is thought to be responsible for the reduction in volume or shrinkage. Another theory is that of disjoining pressure, which occurs in areas of hindered adsorption (interlayer water); removal of this water causes a reduction in pressure and, hence, a reduction in volume.

The foregoing theories apply to reversible behaviour and shrinkage is not fully reversible, probably because additional bonds are formed during the process of drying. Moreover, carbonation shrinkage can occur, which prevents ingress of water on re-wetting.

This chapter is mainly concerned with drying shrinkage, namely, shrinkage resulting from the loss of water from the concrete to the outside environment. It should be mentioned that plastic shrinkage occurs before setting and can be prevented by eliminating evaporation after casting the concrete. Like drying shrinkage, autogenous shrinkage occurs after setting. It is determined in sealed concrete and is caused by the internal consumption of water by hydration of cement, the products of which occupy less volume than the sum of the original water and unhydrated cement. In normal strength concrete, autogenous shrinkage is small ($<100 \times 10^{-6}$) and is included with drying shrinkage. On the other hand, in high performance or high strength concrete made with a low water/cementitious materials ratio, autogenous shrinkage can exceed drying shrinkage. At the moment, design guidelines do not provide methods of estimating autogenous shrinkage.

7.4.3 Measurement of shrinkage

Shrinkage of concrete is normally measured at the same time as creep using identical, control, test specimens (see section 7.5.1). It is recorded as a linear contraction since the real volumetric shrinkage is approximately three times the linear shrinkage. The specimens can readily be sealed to determine autogenous shrinkage, drying shrinkage or partly sealed to simulate the effect of size of member (see section 7.4.4).

7.4.4 Factors in shrinkage

Shrinkage of concrete is affected by several factors, the most important being the aggregate, which restrains the shrinkage of the hardened cement paste. The influence is quantified as follows:

$$S_c = S_p[1 - a]^n \tag{7.3}$$

where S_c = shrinkage of concrete, S_p = shrinkage of cement paste, a = aggregate volumetric content (fine + coarse) and n = constant for mixes of constant water/cement ratio $\cong 2E_a \div [E_a + E_c]$. E_a = elastic modulus of the aggregate and E_c = elastic modulus of the concrete.

Equation (7.3) indicates that the greater the volume of aggregate, the lower the shrinkage. For example, increasing the aggregate content from 71 per cent to 74 per cent will reduce the shrinkage by about 20 per cent. Also, the stiffer the aggregate (high E_a), the lower the shrinkage of concrete. Thus, lightweight concrete has a higher shrinkage than normal weight concrete. Aggregate grading hardly affects shrinkage of concrete. However, at a constant water/cement ratio, larger aggregate permits the use of a leaner mix (more aggregate by volume), so that larger aggregate leads to a lower shrinkage.

There are a number of rocks in the UK that shrink on drying so they offer less restraint to the cement paste shrinkage. Igneous rocks of basalt, doleritic types, and greywacke and mudstone sedimentary rocks may increase the shrinkage of concrete substantially. BS 812: Part 120: 1989 recommends categories of use for aggregates that shrink, while BS 1881: Part 5: 1989 specifies a method of assessing shrinkage of concrete.

As could be anticipated, for a constant volume of aggregate, shrinkage increases as the free water/cement ratio increases (see Figure 7.3).

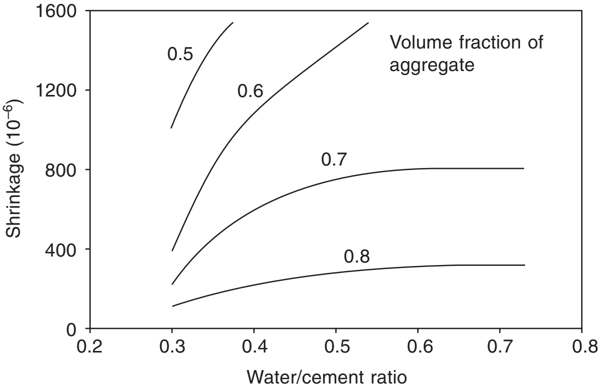

Figure 7.3 Effect of water/cement ratio on shrinkage of concrete for different aggregate contents (based on Odman, 1968).

The lower the relative humidity the greater the shrinkage because the higher relative humidity gradient between the concrete and the environment promotes a greater loss of water. The effect is demonstrated in Figure 7.4. The same figure shows that swelling of concrete stored in water (100 per cent relative humidity) is about six times smaller than shrinkage in air at 70 per cent relative humidity.

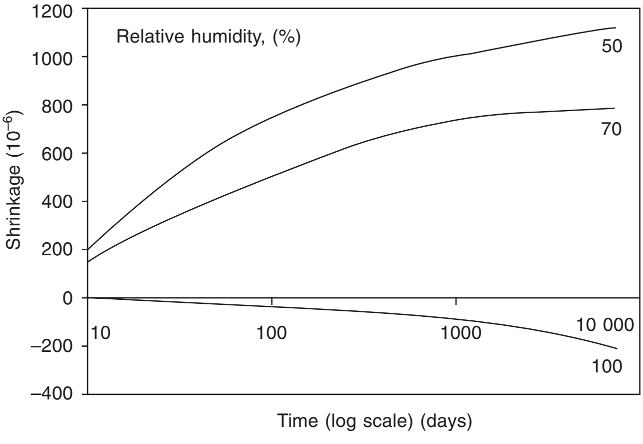

Figure 7.4 Shrinkage as a function of time for concrete stored at different relative humidity; time is from the age of 28 days after wet curing (from Troxell *et al.*, 1958).

Clearly, the time of drying is a factor in shrinkage as it takes place over a long period of time with a high initial rate of shrinkage that decreases rapidly. Typically, as a proportion of 20-year shrinkage: 20 per cent occurs in 2 weeks, 60 per cent occurs in 3 months and 75 per cent occurs in 1 year (see Figure 7.4).

Since drying results from evaporation of water from the surface of a concrete member, the size of the member is a factor in shrinkage. Members having a large cross-sectional area undergo less shrinkage than those with a small cross-sectional area because it is more difficult for water to escape from the former. The effect of size can be expressed as the volume/surface ratio (*V/S*) or effective thickness (= 2*V/S*), which represent the average drying path length, so that shrinkage decreases as the volume/surface ratio increases. The determining parameter is the surface area exposed to drying of the member. Figure 7.5 illustrates the influence of size. There is a secondary, much smaller, influence of shape of drying shrinkage of concrete that is normally neglected.

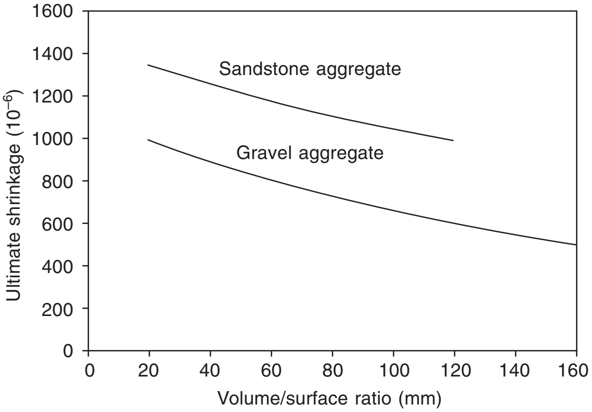

Figure 7.5 Influence of volume/surface ratio on shrinkage of concrete (from Hanson and Mattock, 1966).

7.4.5 Carbonation shrinkage

Drying shrinkage normally includes any carbonation shrinkage although the latter is different in nature. Carbonation is more likely to be known in connection with a cause of possible corrosion of steel reinforcement, it being caused by the reaction of calcium hydroxide with the carbon dioxide (CO_2) present in the atmosphere and in the presence of moisture. First, CO_2 reacts with the moisture to form carbonic acid, which then reacts with $Ca(OH)_2$ to form calcium carbonate ($CaCO_3$). Along with other decomposed cement compounds, the process of carbonation results in a volumetric contraction or carbonation shrinkage.

The rate of carbonation is slow and depends upon the permeability of the concrete, the moisture content and the relative humidity of the ambient medium. For high carbonation shrinkage, the conditions are a high water/cement ratio and storage at about 55 per cent relative humidity. In practice, it is restricted to the outer layers of concrete so its overall effect on movement is small, except in thin sections. Also, it can cause crazing and warping of cladding panels if restricted to one face.

7.4.6 Prediction of shrinkage

Shrinkage and swelling of plain concrete after periods of exposure of six months and 30 years are given in BS 8110: Part 2 (1985). Alternatively, the CEB-FIP method (1990) gives shrinkage as a function of time and is applicable to concrete containing some admixtures. The BS method is shown in Figure 7.6 and applies to concrete made with high-quality normal weight and non-shrinking aggregates, with an initial water content of 8 per cent of the original mass of concrete. For other water contents, the shrinkage of Figure 7.6 is adjusted in proportion to the actual water content. Shrinkage estimated by these methods is not very accurate (±30 per cent at best) and, for better estimates, Neville and Brooks (2001) recommend a short-term test using specimens made from the actual concrete, and then the measured shrinkage-time values extrapolated to obtain long-term shrinkage.

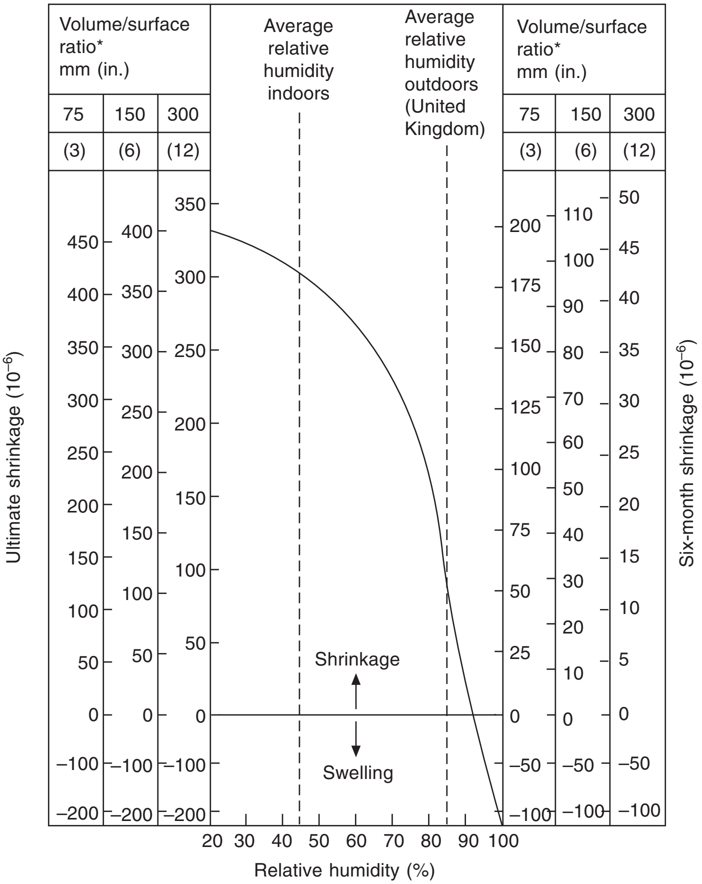

Figure 7.6 BS 1881: Part 2 (1985) method for the prediction of shrinkage.

*Sometimes effective section thickness is used instead of volume/surface ratio; effective section thickness = 2 × volume/surface ratio.

7.4.7 Effects of drying shrinkage

In normal concrete structures drying shrinkage can be as high as 600×10^{-6} which is about six times as high as the failure strain in tension. Consequently, if shrinkage is restrained cracking can occur. Cracks can be induced by internal or external restraint, for example, in the surface by the inner core concrete or by reinforcement. Foundations can externally restrain concrete. The actual tensile stress developed depends on creep (see section 7.5), which is beneficial as it relieves the elastic stress induced by restraint. Figure 7.7 demonstrates the schematic pattern of crack development due to restrained shrinkage.

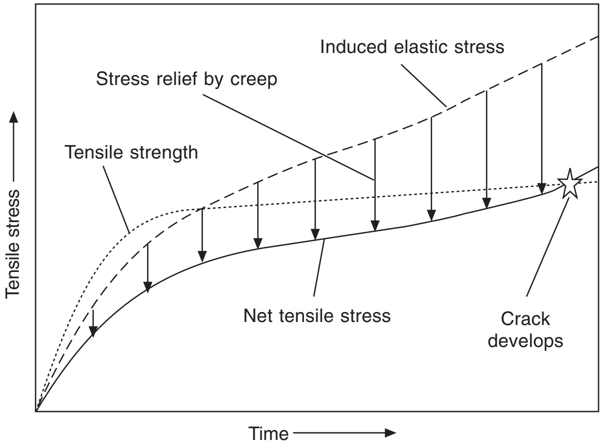

Figure 7.7 Development of induced tensile stress leading to cracking after relief by creep, according to Neville and Brooks (1997).

Shrinkage also causes a loss of prestress in prestressed concrete, and increases deflections of asymmetrically reinforced concrete. In high-strength or high-performance concrete, autogenous shrinkage may be greater than drying shrinkage but the majority occurs early in the life of concrete. To avoid undesirable effects, it may be possible to delay construction operations until after most autogenous shrinkage has occurred.

7.5 Creep

Creep is defined as the gradual increases in strain or deformation with time for a constant applied stress, after taking into account other time dependent deformations not associated with stress, i.e. shrinkage, swelling and thermal deformation. The increase can be several times as large as the elastic strain at loading so that creep is a significant factor in the serviceability of structures and needs to be accounted for in design. Since under normal conditions of loading the elastic strain depends upon the speed of loading (see section 7.3), it will include some creep. In practice, this is not too important as it is the total strain that matters. Also, the modulus of elasticity increases with age and, hence, the elastic deformation gradually decreases and, strictly speaking, creep should be taken as the strain in excess of the elastic strain at the time when creep is being determined. However, this effect is usually small and creep is reckoned from the initial elastic strain on first application of load.

Concrete is not unique in having the ability to creep. Most engineering materials do, for example, rocks at high stresses, plastics (especially thermoplastics), steel at high temperature and even timber. Figure 7.8 shows the general form of a material undergoing creep and, at some time, it will develop tertiary creep and fail by creep rupture or static fatigue. Concrete also behaves in this way but only at stresses greater than about 0.6–0.7 of the static strength. Although concrete is often thought of being brittle in nature because of its tendency to crack under small strains, it is not strictly brittle in the sense that it can develop large strains prior to failure, which is an advantage as sudden and catastrophic failure is avoided.

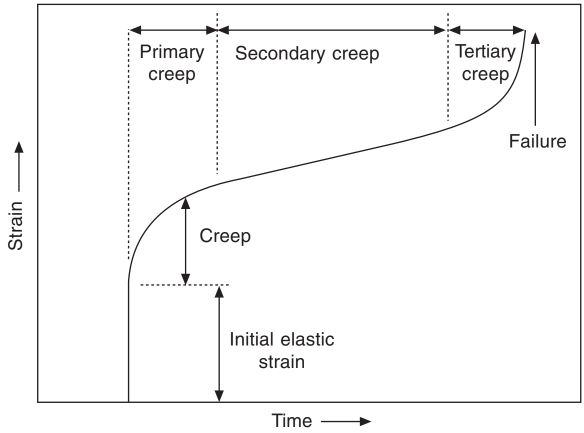

Figure 7.8 General form of a strain–time curve for a material subject to creep rupture.

At normal working stresses, i.e. those within the range of linearity between creep and stress, concrete never exhibits tertiary creep and will continue to deform for several years (up to 30 years has been recorded). Creep is of practical significance since it can be two or three times the initial elastic strain after one year under load, which represents approximately 70 per cent of 20-year creep. In terms of ultimate specific creep or creep per unit of stress, the range is between 20 and 350 × 10^{-6} per MPa.

Figure 7.9 shows the components of time-dependent strain of drying concrete, which applies to general structural elements. The total creep consists of basic-plus-drying creep. Shrinkage is determined on a separate identical specimen but not subjected to load. For sealed concrete, representing mass or large-volume concrete only basic creep occurs and there is no drying shrinkage or drying creep. It should be noted that, according to Brooks (2001) drying creep exists even after allowing for shrinkage because there is an interaction between moisture movement and the external stress to cause an increase in internal stress. Creep of concrete is strongly affected by temperature in a complex manner and the topic is outside the scope of this book; information is given by Neville *et al.* (1983).

Consider concrete loaded to a stress, σ_o, at age t_o. If the specific creep after a time under load, $t - t_o$ is C_s, then the total strain due to load at age t when loaded at age t_o is:

$$\sigma_o \Phi(t, t_o) = \frac{\sigma_o}{E(t_o)}(1 + \varphi) \qquad (7.4)$$

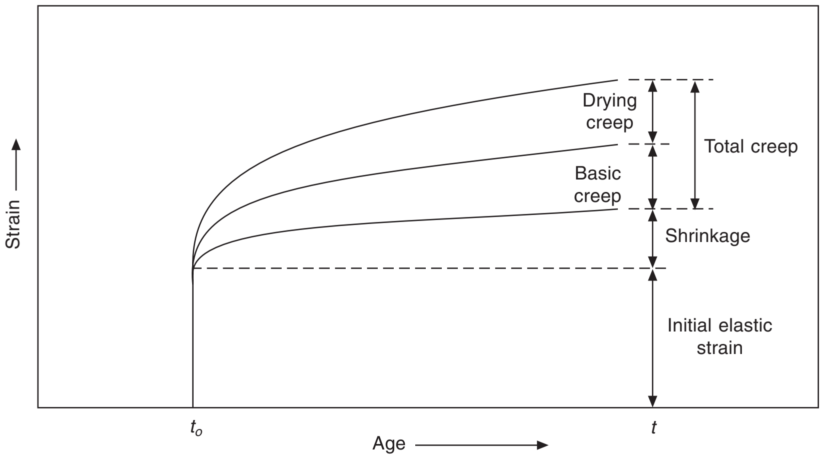

Figure 7.9 Components of strain at age *t* of drying concrete subjected to load at age, t_o.

where $\Phi(t, t_o)$ = total load strain per unit of stress or creep function or creep compliance, $E(t_o)$ = secant modulus of elasticity at age to and $\varphi(t, t_o)$ = creep coefficient, which is defined as the ratio of creep to the elastic strain = $C_s E(t_o)$.

Sometimes, the effects of creep in structures are different from those of purely increasing strain or deflection. Where members are restrained in some way, then the stress will change with time and creep will affect the level of stress. For example, if a specimen is subjected to a constant strain, creep will cause the stress to decrease with time, a process known as stress relaxation.

Creep is a partly recoverable phenomenon. When a sustained load is removed after some time, there is an immediate recovery of elastic strain (generally smaller than the initial elastic strain because the modulus of elasticity has increased), followed by a gradual decrease in strain, called creep recovery (Figure 7.10). The recovery reaches its maximum value more rapidly and is much smaller than the preceding creep so the most of creep is irreversible in nature. Basic creep recovery is about 25 per cent of the preceding creep while drying creep recovery is almost zero.

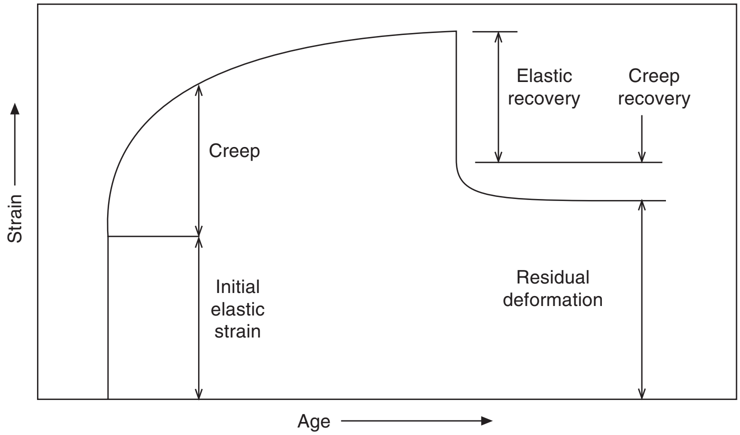

Figure 7.10 Elastic and creep recovery on removal of stress.

7.5.1 Measurement of creep

The only standard test available for the general measurement of creep of any type of concrete under uniaxial compression is that of ASTM C512 (1987). However, BS EN 1355: 1997 exists for the determination of creep of test specimens taken from prefabricated components of autoclaved aerated concrete or lightweight concrete with an open structure. The ASTM C512 apparatus is quite large and expensive especially when large numbers of specimens are required. Researchers tend to use smaller, less expensive equipment such as that shown in Figure 7.11. Two concrete specimens are held in series with a calibrated steel-tube dynamometer by four tie-rods and subjected to a stress of between 0.2 and 0.3 of the strength at the age of loading. After applying the load by manually tightening the four nuts, the strain is recorded as the initial elastic strain together with the time taken to apply the load. The subsequent increase in strain, after allowing for shrinkage or swelling as determined on a separate control specimen, is recorded as creep. A disadvantage of the equipment is that, since there is no spring, there is a loss of load due to creep and so the nuts have to be re-tightened often during the early stages of creep. Strain is usually measured by a de-mountable mechanical strain gauge to approximately 10×10^{-6}. Neville *et al.* (1983) describe alternative types of apparatus for determining creep. To minimize experimental variability, it is important to store specimens and apparatus under controlled environmental conditions of temperature and relative humidity

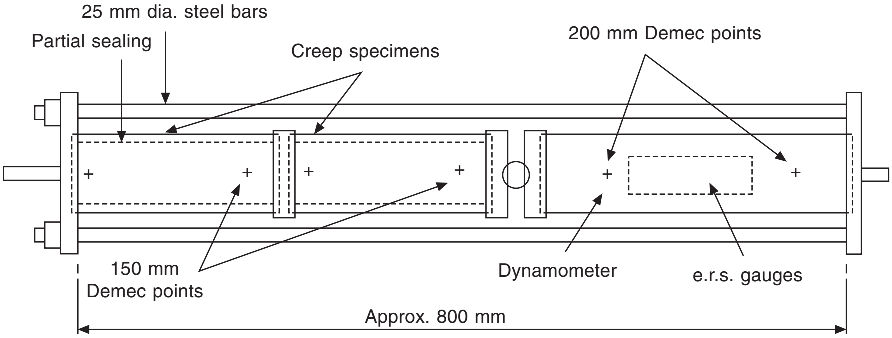

Figure 7.11 Apparatus for measurement of creep.

7.5.2 Mechanism of creep

The exact mechanism of creep is unclear. It is related to the internal movement of adsorbed or interlayer water since concrete from which all evaporable water has been removed exhibits little creep. Movement of water to the outside of concrete is essential for drying creep. Moreover, it is thought by Brooks (2001) that some of the basic creep of mass or sealed concrete is possibly to the internal movement of water, which can occur because all the pores do not remain full of water. The strong dependency of basic creep on porosity or strength is indirect evidence suggesting that it is the empty pores that govern much of creep. The creep of the solid gel and long-term creep after all the water has disappeared may be due to viscous flow or sliding between gel particles.

7.5.3 Factors in creep

The source of creep in concrete is the hardened cement paste since good quality normal weight aggregate does not creep at the levels of stress existing in concrete. The important role of the aggregate is to restrain the creep of the paste in a similar manner to shrinkage (see equation (7.3)). The greater the volumetric fraction and the modulus of elasticity of the aggregate, the greater the restraint and the lower the creep of concrete. For example, limestone aggregate concrete has a low creep and lightweight aggregate concrete has a high creep.

For drying concrete, creep is greater the lower the relative humidity of the surrounding environment, as shown in Figure 7.12. Here, concrete specimens were cured at 100 per cent relative humidity for 28 days and then subjected to load in air conditioned to a different relative humidity. Thus, even though shrinkage has been taken into account in determining creep at a relative humidity less than 100 per cent, there is still an influence of drying on creep. That influence can be less if the concrete is allowed to dry prior to application of load, but such a procedure is not recommended in practice since inadequate curing is regarded as bad practice.

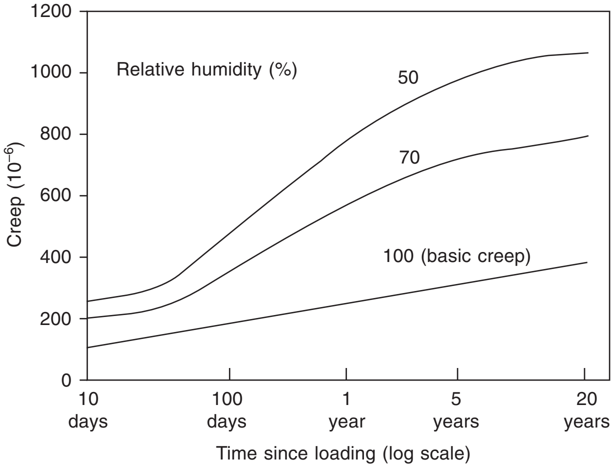

Figure 7.12 Influence of relative humidity on creep of concrete (from Troxell *et al.*, 1958).

Like drying shrinkage (see section 7.4.3), creep of drying concrete is affected by the size of member. Expressing size as the volume/surface ratio (*V/S*) or effective thickness (=2*V/S*), which represent the average drying path length, creep decreases as the volume/surface ratio increases. Figure 7.13 illustrates the influence of size of member. The influence of shape of member on creep is a secondary factor and is normally neglected in prediction methods. Basic creep of sealed concrete or large-volume concrete is unaffected by size of member.

In general, there is a direct proportionality between creep and the applied stress with a certain range of stress so that the term specific creep is applicable. Also, for mixes of constant proportions, creep is approximately inversely proportional to the strength at the time of application of load. Therefore, combining the two influences, creep is approximately proportional to the stress/strength ratio, the upper limit being 0.5 to 0.6. At higher levels, microcracking plays a dominant role and a time-dependent creep rupture is likely.

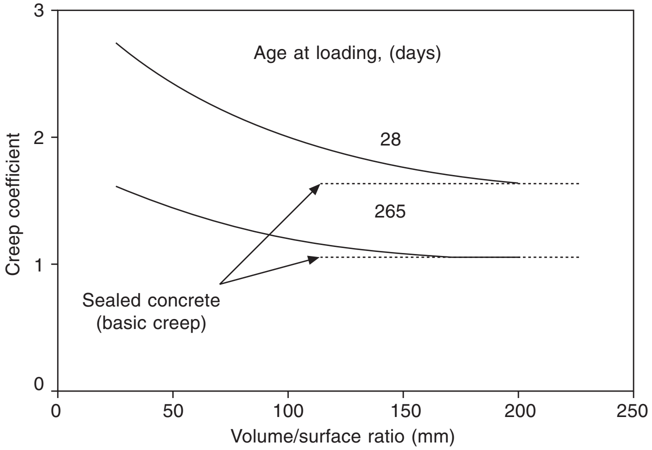

Figure 7.13 Influence of member size on 30-year creep coefficient of concrete stored at a relative humidity of 60 per cent (based on Neville and Brooks, 2001).

The type of cement affects creep in so far as it influences the strength of concrete at the time of application of load. On the basis of equality of the stress/strength ratio, Neville and Brooks (2001) state that most Portland cements lead to approximately the same creep. However, for the same initial stress/strength ratio, creep is also affected by the development of strength after application of the load, a greater strength development leading to lower creep. The latter feature explains why the use of mineral admixtures in concrete, such as fly ash, ground granulated blast furnace slag, silica fume and metakaolin generally result in lower creep. The slow pozzolanic reaction contributes to later strength development by producing a cement paste of lower porosity.

Chemical admixtures, such as plasticizers (water-reducers) and superplasticizers (high-range water-reducers) affect creep in a similar manner, i.e. their influence emanates from the factors affecting the strength at the time of loading and the subsequent development of strength. However, for all types of admixtures, there are circumstances when the foregoing trends do not always apply, for example, under drying conditions. For more accurate assessment of creep, it is recommended that tests be carried out.

It was stated earlier that the cement paste by volume was the main factor in determining creep of concrete, because it is restrained by the aggregate. However, in practical mixes having the same workability, the range of cement contents is small. For example, comparing two normal-weight concretes having aggregate/cement ratios of 9 and 4.5 by mass, with water/cement ratios of 0.75 and 0.40, respectively, the cement paste contents are correspondingly 24 per cent and 29 per cent. Consequently, creep might not expect to be affected to a great extent, but this is not the case since the water/cement ratio influences creep considerably. The effect of water/cement ratio can be perceived in terms of strength, although the fundamental influencing property is porosity. A lower water/cement ratio causes a lower porosity, a higher strength and, therefore, applying the inverse strength rule, a lower creep occurs.

The age at which the load is applied greatly affects the magnitude of creep and its influence can be also be perceived by the change in strength. As the age increases, the degree of hydration of the concrete increases and porosity decreases. Consequently, the strength is greater so that, applying the inverse strength rule, creep is lower. Figure 7.14 shows the trend for concrete stored at a relative humidity of 75 per cent.

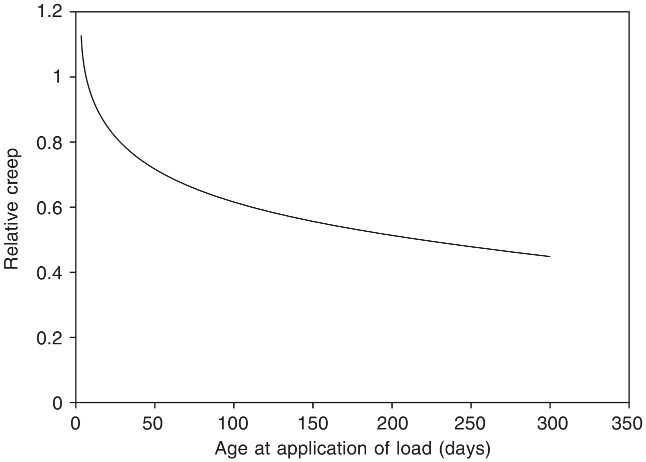

Figure 7.14 Influence of age at application of load on creep of concrete relative to creep of concrete loaded at 7 days; based on L'Hermite (1959).

7.5.4 Prediction of creep

The CEB-FIP (1990) method gives a series of equations to estimate the creep coefficient (see equation (7.4)) as function of time, age at loading, relative humidity, size of member and strength. That method also allows for the use of some admixtures in concrete. On the other hand, the BS 1881: Part 2 (1985) method is not appropriate for concrete containing admixtures but is simpler to use as it gives estimates of ultimate creep coefficient, ϕ_∞, as represented in Figure 7.13. The ultimate creep function, Φ_∞, is then given by:

$$\Phi_\infty = \frac{1}{E_c(t_o)}(1 + \phi_\infty) \tag{7.5}$$

where $E_c(t_o)$ is the modulus of elasticity at the age of loading, t_o, which is related to the compressive cube strength, $f_{cu}(t_o)$ as follows:

$$E_c(t_o) = E_{c28}\left[0.4 + 0.6\frac{f_{cu}(t_o)}{f_{cu28}}\right] \tag{7.6}$$

The strength ratio term of equation (7.6) is best obtained by measurement; however, the values in Table 7.1 may be used. The modulus of elasticity at 28 days, E_{c28}, is obtained from the 28-day cube strength, f_{cu28}, by:

$$E_{cu28} = 20 + 0.2f_{cu28} \tag{7.7}$$

Table 7.1 Values of the strength ratio, $f_{cu}(t_o)/f_{cu28}$, for use in equation (7.7)

Age, t_o	Strength ratio
7	0.7
28	1.00
90	1.17
365	1.25

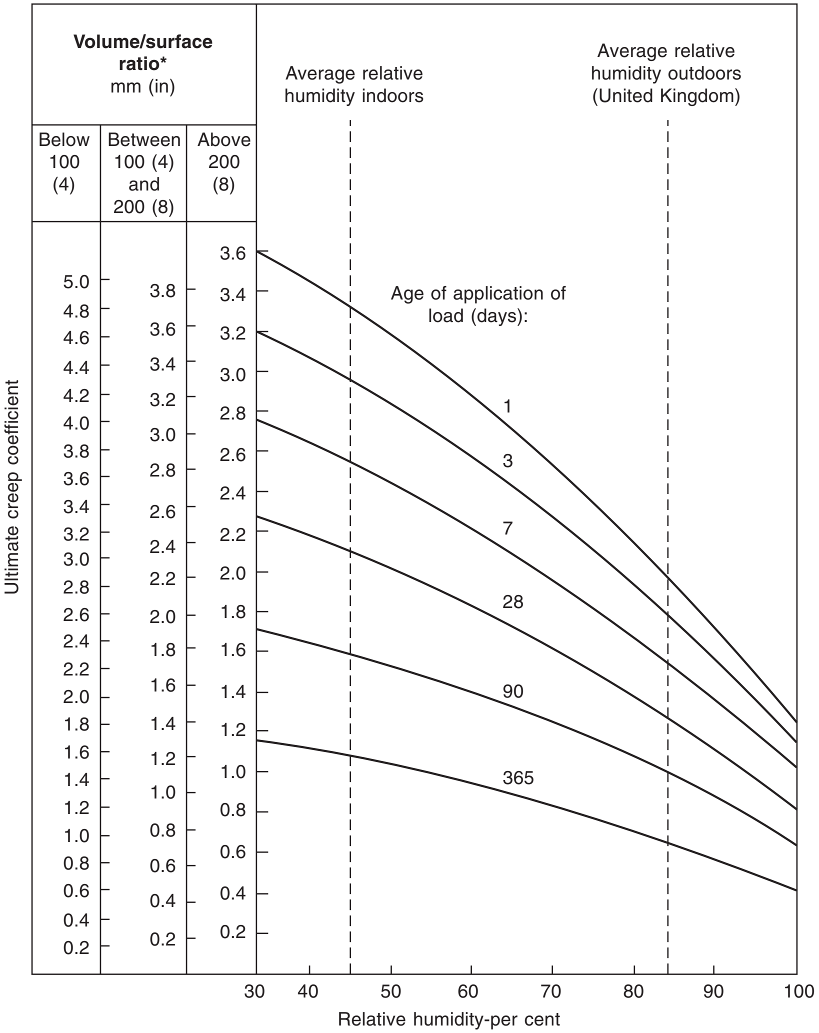

Figure 7.15 BS 1881: Part 2 (1985) method for estimating the ultimate creep coefficient. *Sometimes effective section thickness is used instead of volume/surface ratio; effective section thickness = 2 × volume/surface ratio.

If there is no moisture exchange, i.e. the concrete is sealed to represent mass concrete, creep is assumed to be equivalent to that of concrete with a volume/surface ratio greater than 200 mm at 100 per cent relative humidity. As for the prediction of drying shrinkage, the accuracy of prediction is low. For more accurate long-term values especially when unknown aggregates or admixtures are contemplated, a short-term test is recommended using the actual concrete and the measured data then extrapolated by the method suggested by Neville and Brooks (2001).

7.5.5 Effects of creep

In general the effects of creep are disadvantageous in concrete structures, although there are some benefits. Creep increases the deflection of reinforced concrete beams and causes a loss of prestress in prestressed concrete beams. In reinforced concrete columns, creep causes a gradual transfer of load from the concrete to the reinforcement. Once the steel yields, the concrete takes any increase in load, so that the full strength of concrete and steel is developed before failure occurs. Creep increases the deflection of eccentrically slender columns and can lead to buckling.

On the beneficial side, in statically indeterminate structures, creep can relieve stress concentrations induced by shrinkage, temperature changes or movement of supports. Moreover, in all concrete structures creep reduces internal stress due to non-uniform or restrained shrinkage so that there is a reduction in cracking.

Conversely, in mass concrete, creep may be a cause of cracking when restrained concrete undergoes a cycle of temperature due to heat of hydration and subsequent cooling. Creep may also cause excessive deflection in tall buildings and long bridges. Cracking of partitions and failure of rigidly fixed external cladding can occur due to differential movement.

7.6 Thermal movement

The thermal movement of concrete is the product of the temperature change times the coefficient of thermal expansion. The latter is generally taken to be 10×10^{-6} per °C, but it depends on the mix composition and on its moisture condition at the time of temperature change, which normally is within –30 to 65 °C. Table 7.2 gives the coefficient of thermal expansion of 1:6 concrete made with different aggregates. The reason given by Neville and Brooks (1997) for the slightly higher coefficient for air-cured concrete is that, when partially wet, the water vapour pressure of cement paste increases as the temperature rises and there is a consequent expansion.

Table 7.2 Coefficient of thermal expansion of 1:6 concretes made with different aggregates [Neville and Brooks (2001)].

Type of aggregate	Linear coefficient of thermal expansion, 10^{-6} per °C	
	Air-cured concrete	Water-cured concrete
Gravel	13.1	12.2
Granite	9.5	8.6
Quartzite	12.8	12.2
Dolerite	9.5	8.5
Sandstone	11.7	10.1
Limestone	7.4	6.1
Portland stone	7.4	6.1
Blast furnace slag	10.6	9.2
Foamed slag	12.1	9.2

7.7 Summary

This chapter demonstrates that the short- and long-term deformations of concrete are influenced by many factors, the approach considering each type of deformation independently of each other. This approach is not necessarily fundamentally correct but it has the merit of simplicity and is used by researchers seeking a further understanding of the causes of movements, especially of creep. It is also adopted by prediction methods in Codes of Practice. Of course, in practice, it is the total movement that is of interest to the designer and, in more complex problems, the interaction of elasticity, shrinkage, creep and thermal movement with time-dependent stress arising from a restraint to movement and redistribution of stress. For further details of the methods of analysis in those cases and effects of shrinkage and creep in structural (reinforced and prestressed concrete) members, Neville *et al*. (1983) can be consulted.

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PART 4

Durability of concrete and concrete construction

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8

Durability concept; pore structure and transport processes

Lars-Olof Nilsson

8.1 Introduction

Most concretes are excellent at 28 days. If not, a simple repair or replacement may be done. However, concrete is meant to last for decades or centuries. After the first 28 days concrete will continue to mature and age, depending on the original material composition and properties and the environmental actions during service.

In that ageing a number of transport processes are involved. Most of the changes and deterioration that occur in concrete over time follow from transport of various substances. This chapter aims at introducing the present knowledge on understanding and quantifying the deterioration processes, especially the decisive transport processes, that limit the service life of concrete in structures.

8.2 Durability concept

Concrete may deteriorate with time in a number of ways. The most common durability failures in an outdoor climate are due to reinforcement corrosion or frost attack. In special environments concrete may suffer from chemical attack by various substances such as sulfates, acids, soft water etc. causing disintegration or expansion. Durability failure may also occur because of internal expansion from concrete constituents that are swelling, usually because of a reaction product absorbing water.

The concept of 'durability' is difficult to quantify. Durability may be 'good' or 'better', but such a description has no meaning without a proper definition. Additionally, durability is not a property of a concrete material, or a concrete structure, but 'behaviour', a performance, of a concrete structure in a certain exposure condition.

'Service life' is a much better concept for describing the durability of concrete. The service life is defined as 'the time during which a concrete fulfils its performance requirements', without non-intended maintenance. Consequently, service life is a quantitative concept, with the dimension [years], that can be compared for very different alternative selection of materials or structural design concepts.

To be able to define service life, the 'performance' of the concrete must be identified and the performance requirements must be defined. Traditionally, the load-carrying capacity of a concrete structure is taken as the design parameter, but from practice, experience shows that the performance could involve a number of other things, i.e. aesthetics, apparent reliability, lack of visible signs of deterioration, etc. The definition of service life is shown in Figure 8.1.

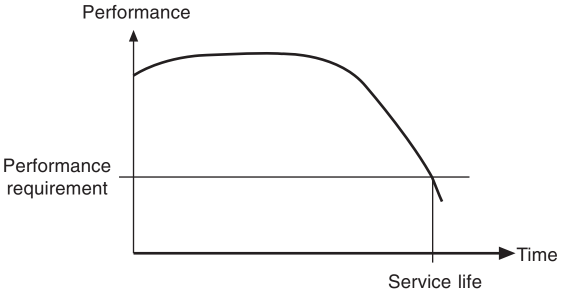

Figure 8.1 Definition of service life.

'Service life design' (SLD) is based on predictions of future deterioration. To be able to make a design for service life certain information must be available (Fagerlund, 1985):

- *Performance requirements*; must be known, relevant and quantified.
- *Environmental conditions*; decisive parameters must be known, including future changes.
- *Deterioration mechanisms*; must be known; if not, the prediction methods, test methods and properties will be irrelevant.
- *Prediction methods*; preferably non-accelerated tests or, better, a theoretical model, decisive material properties and environmental parameters.

Service life design in this way is carried out today mainly by considering initiation, and propagation, of reinforcement corrosion. Elaborated design models consider the uncertainties in the models, decisive properties and environmental actions by applying probabilistic methods in the design procedure (Engelund *et al.*, 2000).

8.3 Forms of physical and chemical deterioration

Different types of concrete deterioration may be described by the nature of the attack, whether it is external or internal, and in what environments the attack will occur. The basic nature of deterioration is mainly of three types: chemical, physical or electrochemical, the latter concerning reinforcement corrosion.

A *chemical attack* involves dissolution of substances or chemical reactions between substances and components of the concrete. Reaction products might cause problems, due to dissolution or expansion. Examples are numerous:

- *Acid attack* dissolving the binder from the concrete surface
- *Sulfate attack* from the surface, by ground water or sea water, or internal sulfate attack (‘delayed ettringite formation’) creating a reaction product that absorbs a significant amount of water, causing internal swelling and cracking
- *Alkali–aggregate reactions* from alkali from the cement, or the exterior, reacting with components of certain reactive aggregates, to produce expansive products
- *Carbonation* or neutralization from weak acids, including airborne carbon dioxide, that reacts with components in the pore liquid, to reduce pH
- *Soft water attack* causing leaching of the alkalis and calcium oxide, that in turn causes dissolution of deposited calcium hydroxide $Ca(OH)_2$ and binder components.

A ‘pure’ *physical attack* could be a non-reacting liquid, or heat, penetrating into concrete or a concrete component, causing internal stresses and expansion, that will result in internal cracking or surface scaling. Examples are:

- *Extreme temperature* changes and gradients due to fire or other significant heating and cooling
- *Frost attack* or frost and salt attack
- *Erosion, weathering* etc.

The typical *electrochemical attack* is reinforcement corrosion, where chemical reactions at the anode and cathode are combined with an electrical current through the steel and through the concrete.

An important type of *physical process,* that is not necessarily a physical ‘attack’, is the particular transport mechanism involved in many deterioration processes. In a large number of deterioration processes several different chemical and physical reactions are combined, sometimes in a very complex way. In these combinations, one or several transport processes are usually decisive for the rate of deterioration. The permeation properties of hardened concrete are, of course, decisive for transport processes occurring in the pore system of concrete and, consequently, in many cases decisive for the durability and service life of concrete.

8.4 Transport processes

8.4.1 Significance of transport processes

Transport processes and permeation properties are highly significant for *ingress, internal redistribution or loss of substances* that are harmful or beneficial to concrete, its constituents or reinforcement, either individually or when combined with other events. Important examples are:

- *Transport of sulfates* from external sources reaching and reacting with aluminates to form ettringite
- *Internal diffusion of alkalis* in the pore water to reach reactive aggregate particles, to ‘provide’ a reactant for the alkali–aggregate reaction
- *Ingress of chloride* from sea water or de-icing salts and *carbon dioxide* from the air, penetrating the concrete cover, destroying the passivity of reinforcing steel
- *Penetration of water* that saturates the capillary pores, fills the air voids and freezes to cause frost damage
- *Movement of water and moisture* from external and internal sources, being absorbed by ettringite (including delayed) or alkali silica gel causing expansion, or acting as an obstacle to gas and vapour transport and as a prerequisite for the movement of ions,
- *Diffusion of oxygen* participating in the corrosion process
- *Dissolution and diffusion of entrapped air* in and from the air void system that makes further water absorption possible
- *Leaching of alkalis and calcium hydroxide* from the pore water to surrounding water
- *Penetration of steam* through a dry surface layer from an evaporation front being created at a certain depth during a fire
- *Penetration of alkali–silica gel*, more or less viscous, from an expanding reactive particle into the pores of the surrounding cement paste
- *Drying out of moisture* causing shrinkage and shrinkage cracks.

8.4.2 Transport mechanisms

The various ways in which aggressive agents can permeate concrete, or substances involved in deterioration processes can be transported in concrete, are described below.

Permeation

Permeation is the process by which a fluid, gas or liquid, will move in the pore and crack systems of concrete due to pressure differences. The resistance to such a flow is created by the viscosity of the fluid, the friction at the pore and crack walls and the narrowness and the tortuosity of the pores and cracks.

The degree of saturation of the pore and crack system will have a significant effect on permeation. If one of the fluids itself (i.e. water) does not saturate the system completely, empty parts that are filled with another fluid (i.e. air) will block part of the fluid flow. If the degree of saturation is low, the fluid might be disconnected, leaving ‘islands’ of fluid that constitute no, or small, fluid paths.

The fluid pressure might be negative, as for liquids not saturating concrete, giving liquid suction that will create pressure gradients and permeation. This is called capillary suction. In most cases non-saturated permeation of a liquid will be affected by permeation of the other fluid, since the respective fluid pressures are interdependent.

Water is the main substance that moves by permeation in concrete and is relevant to durability. However, since water can be a solvent for a number of substances, various water solutions will move by permeation in and into concrete.

Diffusion

Diffusion is the transport of a vapour, gas or dissolved substance in a fluid due to concentration gradients. Areas with a higher concentration of substance tend to be 'diluted' if no source is available. A concentration of a substance that has a source maintaining the concentration tends to spread until equilibrium is achieved. This is called diffusion. The resistance to such a transport process is created by the denseness of the pore system, the pore sizes and the tortuosity of the pores and cracks. In very small pores diffusion will be affected by molecular collisions with the pore walls.

The degree of liquid saturation of the pore and crack system will have a significant effect on diffusion. Vapours and gases will diffuse very slowly in pores filled with a liquid, finding their way much easier through 'open' empty pores that are connected to form air-filled flow paths. Dissolved substances will, in contrast, require continuous liquid paths to be able to diffuse through concrete.

Water vapour *pressure* is frequently regarded as the driving force for water vapour flow, and the material property of concrete is called water vapour *permeability,* even though the mechanism is diffusion. This causes some confusion with permeation of water and water permeability, since the vapour pressure is only a *partial pressure* of the gas mix containing the vapour. To avoid this confusion, vapour flow in air should be regarded as a diffusion process, driven by gradients in the concentration of vapour. The material property should be expressed as the water vapour diffusion coefficient. However, the definitions will be difficult when vapour and liquid flow are combined, such as for moisture (see below).

A number of substances move by diffusion in, into and out of concrete including water as water vapour, gases in air, individually or all, and a large variety of dissolved ions.

Electromigration

Ions are charged and do not only move by pure diffusion. In test methods where an electrical field is externally applied it is obvious that ions move because of the electrical field. This is called electromigration. However, electromigration is also a transport mechanism in concrete without an external electrical field.

Different ions have an individual mobility that is unique for each ion. Since an ion cannot exist alone, but must be balanced by another ion of opposite charge, the movement of ions will create electrical fields since they tend to move at different rates. This electrical field will significantly reduce differences in rate of movement in such a way that 'slow' ions will move faster and 'rapid' ones will slow down. An important example is NaCl, where the sodium ion will retard the diffusion of the chloride ion.

Electromigration is a transport mechanism that will affect *all* ion transport in concrete and can explain a number of characteristics in describing ion transport as pure diffusion.

Combined transport

The transport of a substance in concrete may be derived from a combination of transport processes. When a substance is part of a fluid, a gas mix or a solvent containing ions, and the fluid moves, the substance is transported by *convection*, but the fluid moves by permeation. Within the fluid the substance may diffuse or move because of electromigration.

Airflow through a dry, very porous concrete is one example of transport by convection, where water vapour in the air will be transported by convection *with* the air stream. Another example is when chloride is moving by pore water transport in and out of concrete in the splash zone of marine structures or structures exposed to de-icing salts. The chloride will diffuse in the pore liquid, but more significant, at least in porous concrete, is the movement of the liquid water itself, transporting the dissolved ions.

A substance, especially water, might move in concrete in different states. In such a case it is usually referred to as ‘moisture’, being a combination of water vapour in the air of the pores, the liquid water in the larger pores, bound water at the pore walls and bound water in the gel. The total transport of moisture is a combination of transport of water vapour by diffusion in air, liquid water by permeation and bound water by another type of ‘diffusion’ because of differences in the state of the bound water; a kind of solid-state diffusion. In practice these different processes may not be distinguished since they cannot be separately measured. In transport laws and test methods the *total* moisture flow is described.

Binding

Most substances will not move in concrete without a more or less significant interaction with the concrete constituents. This interaction is sometimes called ‘binding’ or fixation and the material property is referred to as the *binding capacity.*

Binding of a transported substance will reduce the penetration depth and prolong the time required to penetrate a certain thickness of concrete. The concentration of free substance will also be reduced because of binding effects. Binding of transported substances is also responsible for the slow rate, and small depths, of leaching of calcium hydroxide from concrete and the slow drying of concrete.

The type of interaction behind the binding properties could be very different. Gases, vapours and ions that do not chemically or physically interact with the concrete constituents will show a binding capacity of the concrete depending on the available pore space and water content of the pore system. Such a small interaction is relevant for oxygen and alkalis, for example.

A significant example of binding being the decisive part of a transport process is carbonation, where the gas CO_2 is diffusing through concrete but continuously bound, by chemically reacting with CaO, to such an extent that the depth of penetration is very low in a 100-year perspective, even though concrete may be fairly open to the diffusion of a gas. Similar examples are frequent for many transported substances, i.e. moisture, chloride and sulfates.

The binding properties of concrete are given as ‘binding isotherms’ since most binding properties are temperature dependent. A binding isotherm gives the total or bound amount of the substance versus the state of the substance.

8.4.3 Transport laws in general

Several equations may describe the rates of movement of liquids, gases and ions in concrete. The most important give the steady-state flow and the non-steady-state penetration or leaching/drying profiles and depths.

Steady-state flow

This describes the flow that will be reached once steady state is reached. The description of steady-state flow contains two parts. One part describes the driving force, usually as a gradient in flow potential ψ, with the potential being, for example, pressure, concentration, state or electrical potential. The other part describes the properties of the concrete, and sometimes the substance or pore liquid, and is expressed as a flow coefficient k_ψ. In one dimension the steady-state flow of a substance is

$$q = -k_\psi \cdot \frac{\partial \psi}{\partial x} \quad (\mathrm{kg/m^2 \cdot s}) \tag{8.1}$$

The flow coefficient could be a 'diffusion coefficient' with the 'free' concentration c as driving potential, a permeability with the pressure P as flow potential, etc.

Binding capacity

The substance could be bound or fixed to the material. For a number of substances the amount of bound substance C, and the total amount C_{tot} (= $C + c$), could be described as a function of the flow potential ψ. The binding capacity is the change in bound substance ΔC when the flow potential changes $\Delta\psi$:

$$\text{binding capacity} = \frac{\Delta C}{\Delta \psi} \tag{8.2}$$

Non-steady-state transport

When the substance transport varies with time t (and space), the mass balance equation, with equation (8.1) inserted, gives 'Fick's second law' (if the flow coefficient and the binding capacity are regarded as constants):

$$\frac{\partial C_{\mathrm{tot}}}{\partial t} = D_{\mathrm{app}} \frac{\partial C_{\mathrm{tot}}}{\partial x^2} \tag{8.3}$$

where D_{app} is a 'substance diffusivity', i.e. a 'material property' that gives information on the rate of changes in amount of substance.

From equations (8.1)–(8.3) the 'substance diffusivity' follows from the flow coefficient and the binding capacity:

$$D_{\mathrm{app}} = \frac{k_\psi}{\dfrac{\Delta(C + c)}{\Delta \psi}} \tag{8.4}$$

Equations (8.3) and (8.4) are commonly used for diffusion of substances, but the same approach could be used for other transport processes as well, as long as the flow potential and the binding capacity are correctly identified.

The present knowledge of individual transport processes is summarized and discussed in the following sections. Need and lack of understanding and quantification are exemplified. Selected test methods for various transport processes are presented and questioned.

The description starts with moisture transport since most other transport processes are affected by moisture transport and moisture conditions.

8.4.4 Moisture transport

Moisture transport processes cannot be understood without considering the moisture fixation in the concrete pore system. The moisture sorption isotherm is then a key parameter, giving the relationship between the moisture content and the state of moisture.

The moisture sorption isotherm

The amount of moisture in concrete usually is described as moisture content w_e (kg/m^3) or moisture ratio u (kg/kg). The state of moisture in concrete may be expressed as the relative humidity (RH) φ, since there are unique relationships between RH and the 'adsorbed water' in the gel and RH and the 'capillary condensed water' in the larger pores. Consequently, the specific surface area and the pore size distribution of a concrete will give a relationship between the total amount of physically bound water and RH. This relationship is called the 'moisture sorption isotherm'. Examples for OPC concrete are shown in Figure 8.2.

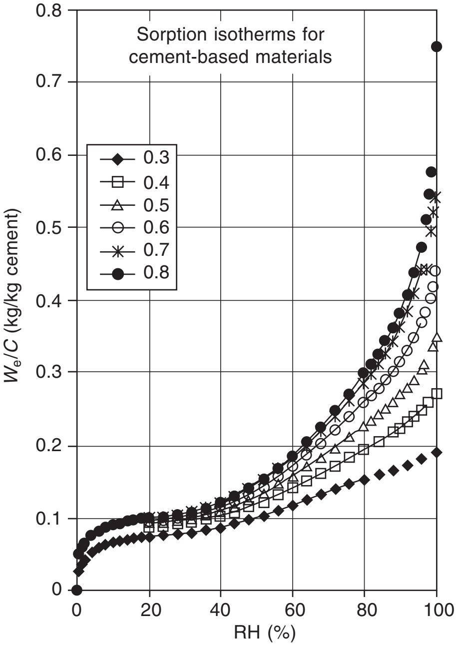

Figure 8.2 Moisture desorption isotherms for cement-based materials with w/c = 0.3–0.8, OPC (Nilsson, 1980).

The moisture sorption isotherm is a function of concrete composition, mainly the w/c and the type of binder, and age, moisture history and, to some extent, temperature (see below). The moisture sorption isotherm is usually determined in a direct way by measuring the moisture content $w_e(RH)$ of samples after they have reached equilibrium with surrounding air of a certain RH. However, the sorption isotherm also works the other way around, giving the local equilibrium in each point of a concrete structure, $RH(w_e)$.

The amount of physically bound water in concrete dominates over the amount of vapour in the air of the empty pores, and over the amount of vapour in connected air spaces. The moisture capacity $\Delta C/\Delta\psi=\Delta w_e/\Delta\varphi$ of concrete is some 100 kg/m^3 but some 1 g/m^3 for vapour in empty pores. Consequently, the moisture capacity of concrete is some 10^5 times larger than for air.

The sorption isotherm is almost independent of temperature. However, a small temperature effect does exist, (see Figure 8.3).

The vapour content in the concrete pores will have a strong influence on the moisture

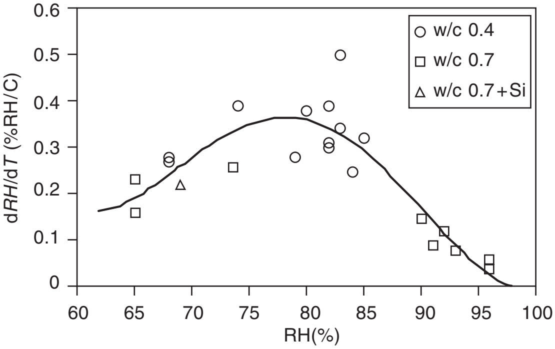

Figure 8.3 The temperature effect, at a constant moisture content, on the moisture sorption isotherms for concrete, +5°C to +20°C (Nilsson, 1987).

flow and the moisture flow direction. From the definition of relative humidity, the relation between the current vapour content v of air and the vapour content at saturation $v_s(T)$, i.e. $RH = v/v_s$, the vapour content in the pores of concrete follows from

$$v(w_e, T) = RH(w_e, T) \cdot v_s(T) \tag{8.5}$$

The dominant effects on the vapour content, according to equation (8.5), obviously are the moisture content that decides the RH and the temperature that decides the v_s. The temperature effect on the sorption isotherm has a much smaller effect, but may be visible when comparing RH measurements at very different temperatures.

When measuring RH in field conditions, where temperature variations are significant, another temperature effect most certainly will cause larger errors in RH measurements (Andrade *et al.,* 1999). Temperature variations may cause a phase difference in the temperature variations of the concrete and the RH probe. From that phase difference a temperature difference between the RH probe and the material will arise. Such a temperature effect will cause an error of some –5%RH/°C (Nilsson, 1987). Condensation on the RH probe may very well occur when concrete is cooling down, contrary to what happens in concrete, where RH somewhat drops when the temperature drops. Consequently, temperature differences must be avoided or measured and corrected for (Nilsson, 1997).

Description of moisture flow

Traditionally, moisture flow in porous materials is regarded as a combination of vapour and liquid flow in the pores. In concrete with low w/c a significant portion of the (small) moisture flow will be ‘physically bound’ water being transferred through the gel due to differences in the state of moisture, a kind of a ‘bound water transport’ similar to what happens in the cell wall of wood (Siau, 1995).

Since the various types of moisture flow cannot easily be separated, the description of moisture transport is determined by what can be measured. For conditions without significant temperature gradients, which is the common situation for concrete, having such a high heat diffusivity rapidly equalizing temperature differences within the concrete, a number of state parameters could be used. *RH, v* or pore water pressure P_w are all uniquely related through the Kelvin equation

$$P_w = \frac{RT_\rho}{M} \ln \frac{v}{v_s(T)} = \frac{RT_\rho}{M} \ln \varphi \tag{8.6}$$

where ρ and M are the density and molar weight of water. Consequently, any one of them would be applicable. Traditionally, the vapour content of the air in the empty pores is used to describe moisture flow:

$$q_w = -\delta(\varphi)\frac{\partial v}{\partial x} \tag{8.7}$$

where δ(φ) is the moisture-dependent moisture flow coefficient giving the total flow of moisture. Consequently, δ(φ) increases significantly with RH, (cf. Figure 8.4).

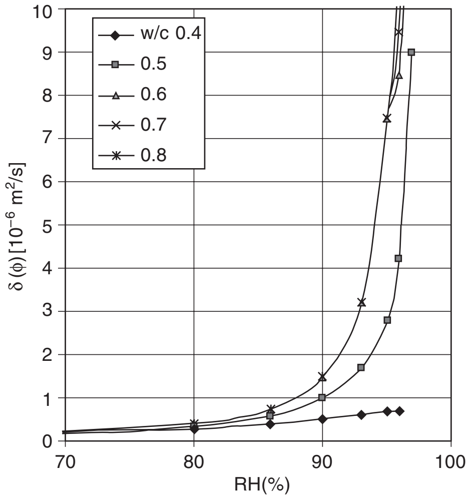

Figure 8.4 The moisture dependency of the moisture flow coefficient δ (φ) for mature concrete (Hedenblad, 1993).

The maximum RH in Figure 8.4 is different for the different concretes and significantly below 100%, because of the pore water being an alkaline solution with a higher concentration for low w/c (Hedenblad, 1993).

The data in Figure 8.4 is for mature concrete, simply because most test methods require steady-state or equilibrium conditions to be achieved. A significant lack of data on moisture transport at early ages exists. A recent study contributes an important method and new knowledge (Vichit-Vadakan, 2000).

Effect of temperature changes

The effect of temperature changes on moisture transport is clearly seen from equations (8.5) and (8.7). The moisture flow coefficient δ (φ) and $RH(w, T)$ are little influenced by the temperature change. Instead, the temperature change will significantly change the vapour content at saturation $v_s(T)$ and, consequently, the driving force v for the moisture transport. A temperature rise will then increase the moisture flow proportionally to the increase in vapour content at saturation. Very large vapour content differences between concrete and the surrounding air, or another material that did not change its temperature as much, may be achieved in this way, i.e. due to solar radiation, long-wave radiation. (radiation from a warm concrete structure), or simply by heating.

Moisture transport in concrete under a temperature gradient still is not understood or quantified. An additional term is added to the flow equation (8.7), but data for the additional flow coefficient for this second term is largely missing.

Steady-state moisture distribution

From the moisture dependency in Figure 8.4 the steady-state moisture distribution can be estimated. It is highly non-linear (cf. Figure 8.5) with a dry upper surface even if the bottom is standing in water.

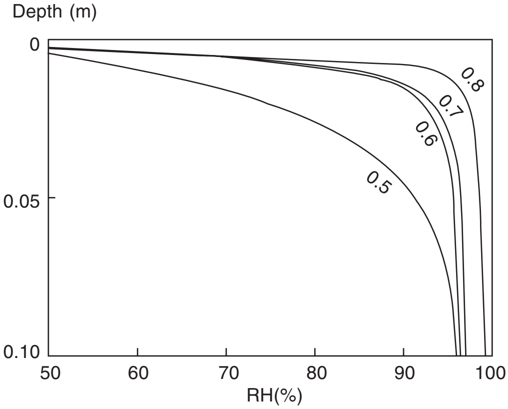

Figure 8.5 RH distributions through a 0.1 m thick concrete slab with the bottom surface standing in water and the top surface in contact with dry air (data from Hedenblad, 1993).

From Figure 8.5 it is seen that concrete is very resistant to moisture flow. Also a very thin concrete structure may very well be dry at a surface in contact with air even though the concrete is saturated at a small depth from that surface. The major resistance to moisture flow is inside the concrete and not at the surface as for many other building materials.

The moisture diffusivity

For moisture variations a moisture diffusivity D_w is the decisive material property. From equations (8.4) and (8.1), with $\psi = v$, $k_\psi = \delta$, $C = w_e$ and c (= v) being insignificant compared to w_e, we obtain

$$D_w(w_e, T) = \frac{\delta(\varphi)}{\frac{\Delta w_e}{\Delta \varphi}(\varphi)} \cdot v_s(T) \tag{8.8}$$

As seen from equation (8.8) the moisture diffusivity D_w is far from being a constant. There is a significant temperature and moisture content dependency.

An estimate of the magnitude of the moisture diffusivity can be derived from Figures 8.4 and 8.2. With a moisture flow coefficient of $\delta = (0.1 - 10).10^{-6}$ m^2/s, a moisture capacity of $\Delta w_e/\Delta\varphi = 100$ kg/m^3 and a vapour content at saturation of 10 g/m^3 (T = + 11°C), D_w equals 10^{-11} to 10^{-9} m^2/s. That is a good measure of the time constant for moisture changes in concrete. The time $t_{0.5}$ to reach half of a moisture change will be (Pihlavaara, 1965)

$$t_{0.5} = \frac{F_o \cdot L^2}{D_w} = \frac{0.2 \cdot L^2}{10^{-10}}(\text{s}) \approx 200 \text{ days } (L = 0.1 \text{ m})$$

where F_o is the Fourier Number and L is the equivalent thickness of the concrete structure. Concrete is obviously a material with a very slow response to moisture changes.

Drying of concrete

When concrete is drying, the moisture transport inside concrete is the decisive factor, as for steady-state flow in Figure 8.5. The evaporation at the concrete surface is usually not a limiting factor, i.e. there is no first drying phase for concrete as for many other materials (Selih *et al.,* 1996). Only for very young concrete, a few days old, and for high w/c above 0.5 such a first drying phase does exist (Nilsson, 1980). Consequently, drying of a particular concrete is mainly affected by the concrete quality, i.e. the moisture diffusivity D_w, the thickness of the structure and the temperature level.

Capillary suction and wetting

When concrete is subjected to capillary suction/wetting by water the amount of absorbed water usually is described by a water absorption coefficient A:

$$Q = A(w_e)\sqrt{t} \tag{8.9}$$

This coefficient is frequently, and easily, determined for very dry concrete with initial conditions far away from most applications. This is simply because the test requires the initial conditions to be controlled and since it takes a very long time to reach equilibrium, fast drying at elevated temperature is used. Such a water absorption coefficient is of little use for practical applications. Instead, tests must be carried out at initial conditions much closer to reality, i.e. equilibrium conditions not too far from saturation. An excellent, but rare, example is shown in Figure 8.6.

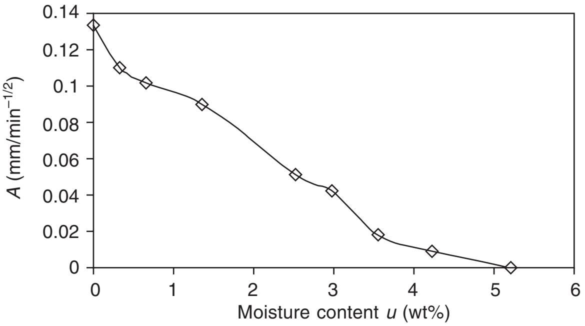

Figure 8.6 The water absorption coefficient A as a function of initial moisture content (De Souza *et al.,* 1998).

The water absorption coefficient A is of course zero at saturation. At a degree of saturation of 50%, the water absorption coefficient is less than a third of what is found from usual tests on dry specimens!

Moisture variations

The response to moisture variations, given by a cosine function with amplitude Δw_e and with a time period t_p, i.e. a 'wave length' of the moisture variation, is shown in Figure 8.7. The 'moisture penetration depth', d_p, is the depth where one third of the moisture variations remain. That depth is a function of the moisture diffusivity and the time period (Lindvall, 1999):

$$d_p = \sqrt{\frac{D_w \cdot t_p}{\pi}} \tag{8.10}$$

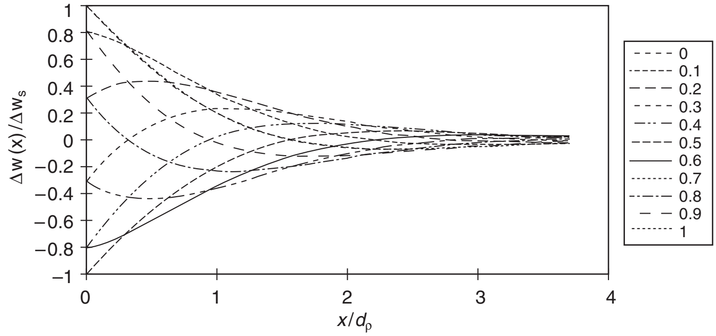

Figure 8.7 Moisture content variations in concrete due to periodic surface moisture variations as a function of relative depth (Lindvall, 1999).

The penetration depth of a moisture variation with different time periods is presented in Figure 8.8. One third of annually varying surface moisture content will penetrate only into a few cm of concrete. Monthly variations will penetrate less than 1–2 cm.

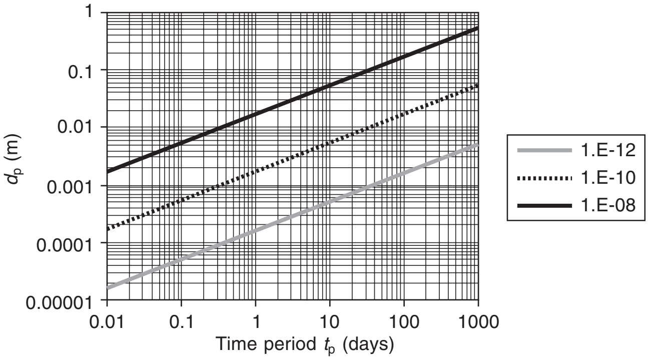

Figure 8.8 Periodic moisture penetration depth d_p for concrete, with different moisture diffusivities $D_w = 10^{-12}$ to 10^{-8} m²/s, as a function of the time period t_p (Lindvall, 1999).

Long-term water absorption

It seems as if the normal moisture flow and capillary suction equations (8.7) and (8.9) are not applicable for low w/c concrete. For w/c 0.40 and lower less than 20 mm depth of penetration was found after two years of exposure in the submerged zone; beneath that only self-desiccation (see Figure 8.9).

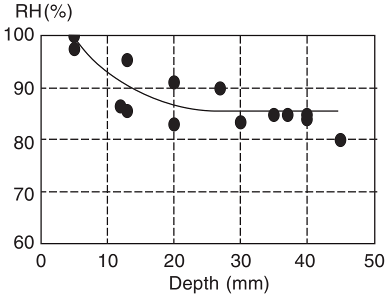

Figure 8.9 Moisture profiles as RH from samples after two years of exposure in sea water. Slab H4, SRPC cement with 5% silica fume, water–binder ratio 0.40 (Rodhe, 1994).

This is yet to be explained. The concrete surface that is continuously exposed to water may be thoroughly cured to a degree of hydration far beyond what is usually achieved in laboratory tests. Alternatively, all the concrete may be so dense that the penetration that is measured is only due to percolation in a small portion of the thickness.

8.4.5 Carbonation

Carbonation is a transport process where diffusion of a gas, CO_2, through a more or less water-filled pore system is combined with a chemical reaction between CO_2 and CaO in the concrete. The decisive rate-determining processes are both of them. A low-w/c concrete, and/or wet concrete, will have a low diffusion coefficient for CO_2. A concrete with large binder content will have a large amount of 'available' CaO that will significantly reduce further penetration of CO_2. Reliable data on both of these processes is required to predict carbonation processes.

However, carbonation in a concrete structure is not only dependent on the material properties. The diffusion of CO_2 is highly affected by the distribution of moisture in the carbonated part of the concrete cover. This moisture distribution will depend on the climatic conditions at the concrete surface, mainly the time of wetness (TOW) and the interval between rains, and the moisture transport and moisture fixation properties of the concrete cover, not only the carbonated part of it.

A complication of the carbonation process is that the local material properties change with time when the carbonation front penetrates inwards. The moisture sorption isotherm is much lower for carbonated concrete, which means that the gas diffusion resistance is lower than for uncarbonated concrete. Relevant properties must therefore be measured on carbonated samples!

In a real environment the moisture conditions in the carbonated concrete constantly vary with time and depth. That makes the carbonation process in field conditions very complicated and much different from the process in laboratory carbonation tests. Recently, a complete carbonation model was presented (Nilsson and Rodhe, 1997) that is based on the moisture transport processes described in section 8.4.4. The depth of carbonation is predicted by numerically solving the following equation:

$$x_{CO_3} = \int_0^t \frac{c_0}{a} \cdot \frac{1}{\displaystyle\int_{X'=0}^{X'=X'_{CO_3}} \frac{dx'}{D_c\,(RH(x'))}}\, dt \qquad (8.11)$$

In the model c_0 is the concentration of carbon dioxide in the outdoor air, a is the 'available' amount of CaO in the concrete and $D_c(RH)$ is the RH-dependent diffusion coefficient for CO_2. The depth of carbonation is calculated stepwise by first predicting the temperature and moisture distribution variation in the concrete cover. From the moisture distribution the resistance to diffusion of carbon dioxide through the carbonated zone is calculated and finally the increment of carbonation depth in each step is added to the previous depth of carbonation.

The model considers the different moisture properties of the carbonated zone, compared to the non-carbonated part of the cover, and keeps track of the continuous change of the thickness of that part of the cover.

The environmental actions at the concrete surface, and the environmental response by the concrete, are decisive for the carbonation process. The actual climate must be translated into times of wetness and intervals of dry periods between rain showers. The water absorption from driving rain must be treated with very short time steps, since that process is very rapid and the amount of rain water is limited, which makes long-term predictions for, say, 50 years very time-consuming.

An example of predicted moisture profiles during a carbonation process is shown in Figure 8.10. The 'jumps' in moisture contents at certain depths correspond to the depth of carbonation.

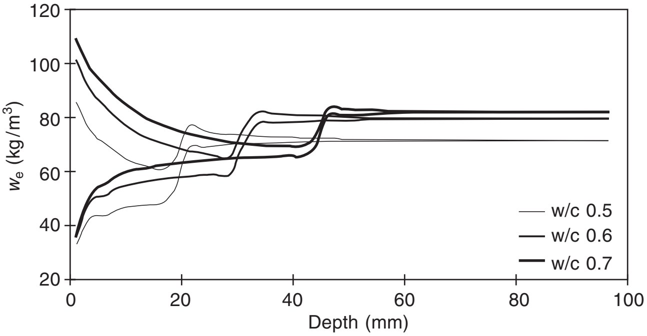

Figure 8.10 Predicted moisture content profiles after 100 years of carbonation in a natural climate. Three concrete qualities (Nilsson and Rodhe, 1997).

The model does not include possible re-alkalization by hydroxides leaching from depths larger than the depth of carbonation. It also gives a distinct carbonation front, which is not what is observed if the degree of carbonation as a function of depth is measured. This is not yet explained. Besides that, most aspects of the carbonation process are included, but the limits in applicability of such a complete model are the lack of data on climatic actions on various concrete surfaces.

8.4.6 Chloride ingress

Chloride ingress, and removal, is a combined transport process of chloride movement and chloride binding/liberation. Traditionally, since chloride transport processes are extremely complicated, an empirical approach has been used for some 30 years to describe chloride ingress. Empirical methods utilize an apparent diffusion coefficient, as in equations (8.3) and (8.4). The development and present lack of understanding are presented in this section. More physical approaches are based on the present understanding of chloride movement and chloride binding.

Chloride binding

Mechanisms of chloride binding are not yet clarified. It is quite clear, however, that the binding is not only chemical and that chemical binding in Friedel's salts, i.e. reaction products of C_3A and chloride, is only a limited portion of the total binding (Nilsson *et al.,* 1996). Physical binding is believed to occur at pore surfaces in electrical double-layers (Chatterji and Kawamura, 1992). Altogether, however, various mechanisms give a 'non-linear binding' not being proportional to the concentration of chloride but more significant at lower concentrations than at higher (Nilsson *et al.,* 1996).

The amount of bound chloride is affected by a number of parameters (Nilsson *et al.,* 1996). The effect of concrete composition is mainly due to the C_3A content and the amount of gel (Tang and Nilsson, 1993). The latter explains part of the effect of w/c, age and type of binder. The alkalinity of the pore water has a large effect on chloride binding, i.e. leaching of alkalis is an important process that changes chloride binding over time (Tang, 1996). It seems as if alkali hydroxide and chloride 'compete' for the same binding 'sites'.

Carbonation reduces the chloride binding to almost zero (Tuutti, 1982). It seems as if chloride binding is also completely reversible, i.e. when the chloride concentration drops in the pore water, the amount of bound chloride also drops. Francy (1998) showed that the surface chloride content after immersion in water 'returned' to close to zero.

Temperature has a similar effect on chloride binding as on moisture fixation, because of slightly temperature dependent binding isotherms in both cases, a rise in temperature decreasing the amount of bound substance. An example of the temperature effect for chloride binding is shown in Figure 8.11.

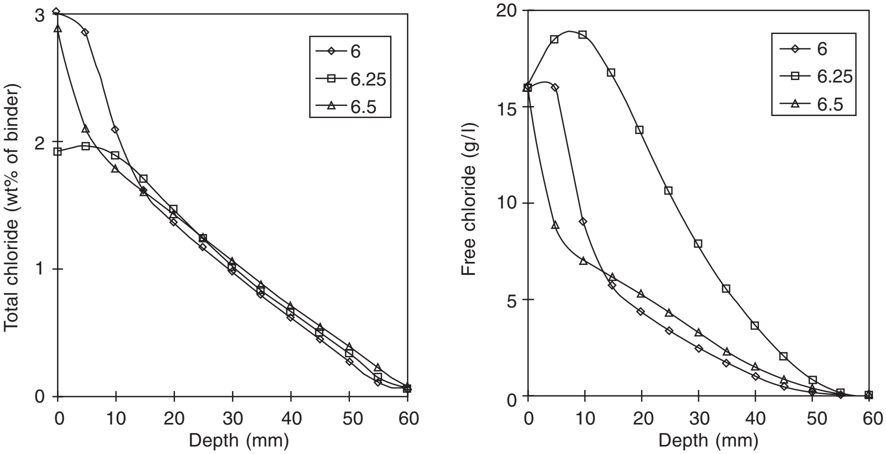

Figure 8.11 Predicted profiles of total (left) and free (right) chloride by ClinConc (Tang, 1996) the sixth year after exposure. The curves are for April (6 years), July (6.25) and October (6.5) SRPC concrete with *w/c* 0.40 exposed to sea water with a chloride concentration of 16 g/l (Nilsson and Tang, 1998).

Temperature variations give a maximum in predicted profiles and a counter-diffusion of chloride during summer periods (cf. Figure 8.11). The strong temperature dependence of chloride binding causes some of the bound chloride to dissolve when the temperature rises, resulting in a large increase in concentration of free chloride in the summer. The concentration at the surface, however, does not change very much during a year. Consequently, the profile of free chloride shows a larger concentration in the concrete pores than in the surrounding seawater. The predicted profiles also show a maximum of free chloride at a certain depth, i.e. a counter-diffusion of chloride out of the concrete, during the summer period (Nilsson and Tang, 1998)!

The temperature dependency of chloride binding has recently been confirmed experimentally by Larsen (1998). He also demonstrated the expected effect of drying and wetting, with an increasing binding at drying because of the rising concentration of free chloride in the pore water.

Recently exposure data strongly indicated that chloride binding increases with time, also in submerged concrete (see Figure 8.12.) This is not yet experimentally confirmed or theoretically understood.

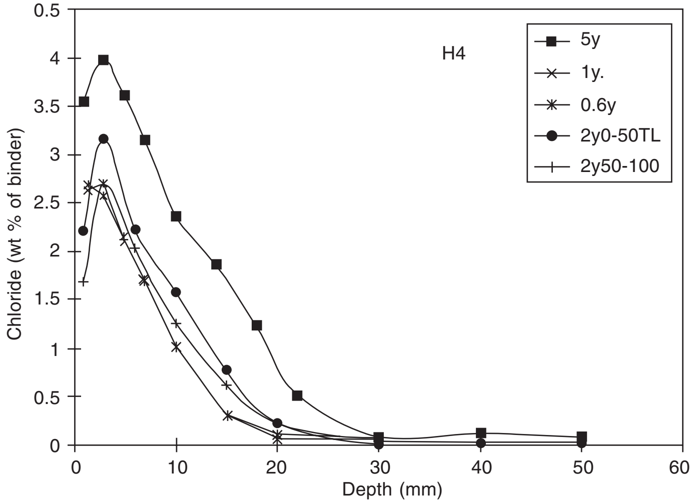

Figure 8.12 Chloride profiles after 0.6 to 5 years exposure, SRPC cement with 5% silica fume, water–binder ratio 0.40 (Tang and Nilsson, 1999).

Chloride transport

The movement of chloride ions, as for other ions, has been treated as if the ions were uncharged particles and that Fick's first law was applicable:

$$q_{\mathrm{Cl}} = -D_{\mathrm{F1}}\frac{\mathrm{d}c}{\mathrm{d}x} \tag{8.12}$$

All kinds of strange experimental results have been found that could not be easily explained. One example is the concentration dependency of the chloride diffusion coefficient in Fick's first law (see Figure 8.13). This is now possible to explain by considering the other ions at the same time (Truc, 2000).

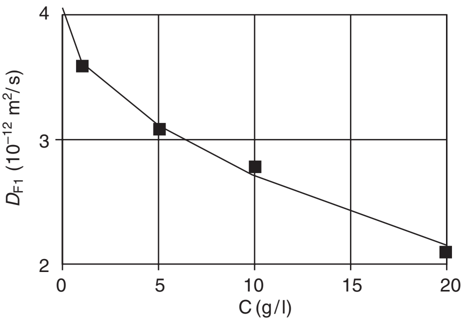

Figure 8.13 Concentration dependent diffusion coefficient in Fick's first law (Nilsson *et al.,* 1996), based on data from Bigas (1994).

The most recent way to describe chloride transport is to consider all ions and for each ion apply the Nernst–Planck equation

$$q_{\text{Cl}} = -D_i\,(w)\left(\frac{\mathrm{d}c_i}{\mathrm{d}x} + c_i\frac{\mathrm{d}\ln a_i}{\mathrm{d}x} + \frac{z_i F}{RT}c_i\frac{\mathrm{d}V}{\mathrm{d}x}\right) + c_i\,.q_l \qquad (8.13)$$

where $D_\mathrm{i}(w)$ is the intrinsic, moisture dependent diffusion coefficient for species i, a_i is the activity, z_i the valence and c_i is the concentration. F, R and T are the Faraday number, the gas constant and the absolute temperature, respectively. V is the electrical potential derived from the interaction between the various ions in the pore water. By including the effect of all ions, a number of chloride transport processes are better understood and quantified (Truc, 2000; Samson, 2000).

Unsaturated chloride transport

The last term in equation (8.13) gives the convection of chloride with the liquid moisture flow q_l. The intrinsic diffusion coefficient D_i (w_e) in equation (8.13) is shown as being moisture dependent. Consequently, to describe chloride transport in unsaturated concrete, i.e. concrete exposed to splash of sea water or de-icing salts, a number of additional material parameters are required. The part of moisture flow that is liquid flow, and the diffusion paths for ions in an unsaturated pore system, must be quantified separately. Until now very few studies have been reported on these transport processes, (i.e. Buenfeld *et al.*, 1995; Francy, 1998 and Nilsson, 2000). Much data and understanding are still missing.

An example of prediction of 'wick action' is shown in Figure 8.14. The depth where chloride is depositing was shown (Nilsson, 2000) to be very sensitive to the division of total moisture flow into liquid and vapour flow. To be able to simulate experimental results, i.e. by Francy (1998), a much larger portion of the total moisture flow than expected, must be regarded as 'vapour flow' or at least a moisture flow that does not 'carry' any ions.

An example of ingress and wash-out of chloride from de-icing salt in a road environment is shown in Figure 8.15. During the first winter a continuous 'build-up' of the surface chloride content occurs. Already in May the early washout is visible. In October the washout is thorough but some chloride continued to penetrate deeper into the concrete during the summer. Comparing the profiles before and after the summer, it is clear that some of the chloride actually leached out of the concrete.

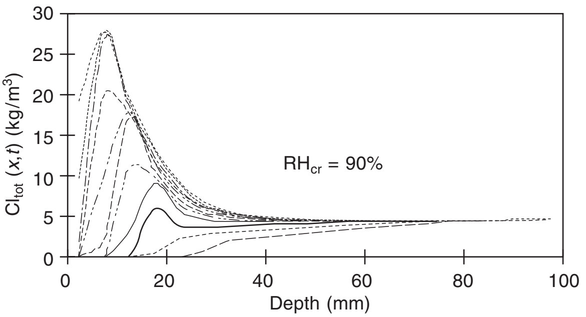

Figure 8.14 Predicted profiles of total chloride during a steady-state experiment with a drying left surface (Nilsson, 2000).

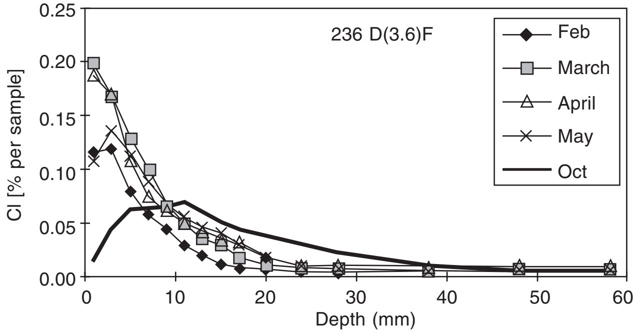

Figure 8.15 Chloride build-up and washout during the first winter and summer of the vertical surface of SRPC-concrete with *w*/c 0.7 (Nilsson *et al.*, 2000).

Figure 8.15 also indicates the complete reversibility of chloride binding. The chloride profile after a summer's splash of pure rain reduces the surface chloride content to almost zero.

Apparent chloride diffusion

Since chloride transport processes, and the boundary conditions that are needed, are so complicated, empirical descriptions are still frequently used. They are all based on curve fitting measured chloride profiles to the error-function solution to Fick's second law, i.e. equation (8.3) (Collepardi *et al.*, 1970);

$$c(x, t) = C_s \cdot \mathrm{erfc}\left(\frac{x}{2\sqrt{D_{app} \cdot t}}\right) \tag{8.14}$$

From the curve-fitting an apparent diffusion coefficient D_{app} is derived as one regression parameter. The other regression parameter is the surface chloride concentration C_s (cf. Figure 8.16). This approach has been rather successful (Frederiksen, 1997), partly because all the things that did not fit the original solution have been used to 'develop' the models for chloride transport processes, usually without any scientific understanding why.

The first observation was that the 'apparent diffusion coefficient' could not be treated as a constant and that it is quite different in a laboratory test and in field exposure. That 'created' a time-dependent $D_{app}(t)$, that significantly decreases with time (Maage *et al.*,

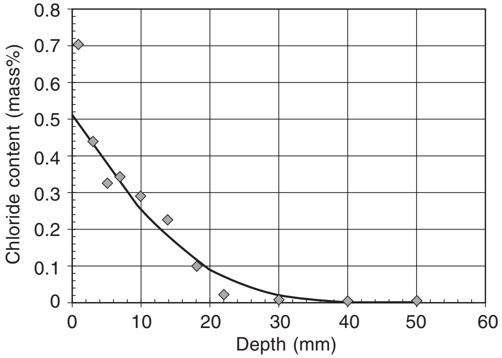

Figure 8.16 Curve-fitting a measured chloride profile to the erfc solution to Fick's second law.

1994). Part of that time-dependency could be explained by continuous densification due to further hydration and binder reactions. It was clear, however, that this could not explain all the 'required' time-dependency!

With a time-dependent D_{app} there is a strong possibility of a misunderstanding since the regression parameter at one exposure time t_i, $D_{\mathrm{app}}(t_i)$, is derived from assuming that it was constant during the time interval $(0, t_i)$!

The next observation was that the surface chloride content is not a constant but increases with time. First this was observed in the splash zone, where continuous wetting and drying might have been the explanation. Now, however, the same observation is done in the submerged zone (cf. Figure 8.12)

8.4.7 Boundary conditions

The required boundary conditions for the various transport processes are very different. Environmental actions on concrete surfaces are complicated, but important to understand and model. Most predictions of transport processes inside concrete are of little value if the surrounding climate is not correctly described.

A simple description of the transport processes, as the empirical curve-fitting of chloride profiles, has a clear advantage in describing the boundary conditions as observed surface chloride contents. However, much good field data is needed to predict the ingress of substances in a new environment. Not only are the environmental conditions different for concrete at different locations, the different locations and orientations of concrete surfaces at a single concrete structure will also have different environmental actions, i.e. different boundary conditions. It is a great challenge for the utilization of prediction models for transport processes to collect data on the variations of boundary conditions in field structures and/or to develop models to describe the boundary conditions from climate data.

One of the most extreme examples is the road environment where the splash of water-containing de-icing salts is mixed with the splash of melting water and pure rain water, direct exposure to rain water from driving rain and occasional drying during the winter season. The rest of the year splash and direct rain will wash out chloride and drying will accumulate chloride at some depth. These conditions will vary with climate, salting frequencies and intensity, traffic type, speed and intensity, vertical and horizontal distance between the concrete surface and the road surface, orientation of the concrete surface etc. A very relevant question is how to consider these aspects for the transport processes inside concrete to be relevant.

8.4.8 Transport properties of site concrete

The concrete mix composition will decide the *potential* permeation properties. The most important parameter for the potential properties is, without doubt, the water–cement ratio w/c or the water–binder ratio w/b. The water–cement ratio will determine the capillary porosity of the cement paste. The lower the w/c, the more cement is originally distributed in the mixing water between the aggregate particles. The lower the w/c, the larger portion of the original space being occupied by the mixing water will be filled by cement reaction products. In modern concrete the w/c is so low that the reaction products have a potential to, in average, fill *all* capillary pores.

Depending on the transport process, the significance of other parameters will vary. For some transported substances the effect of the composition of the binder will be very important, especially those where a chemical binding is involved, such as carbonation and chloride ingress. For other substances, e.g. permeation of gas and liquids, the crack system will be decisive.

An example of the effect of water–binder ratio and type of binder is given in Figure 8.17 for chloride migration.

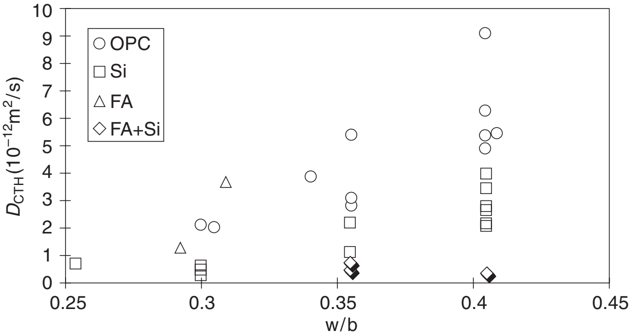

Figure 8.17 The effect of *w/c* and type of binder on the chloride diffusion coefficient, determined by a rapid chloride penetration test method, NT Build 492, data from Tang (1997).

Homogeneity, from mixing, casting and compaction will determine the variations in the potential permeation properties of concrete. The homogeneity depends on the workability of the concrete that will influence its ability to fill the formwork and surround the reinforcement with a limited energy supply, the stability to avoid separation of water and large aggregates, the shape and narrowness of the formwork and the spacing of the reinforcement.

Curing, from procedures applied before and after form stripping, will decide to what extent the potential permeation properties are reached. Curing is important by controlling early evaporation to avoid cracks due to plastic shrinkage, by controlling early-age heat evolution and temperature distribution in the concrete structure to minimize the risk of temperature cracks and by controlling humidity in the concrete surface and reinforcement cover to make the cement reactions continue. The equivalent age, or degree of binder reaction will follow from the concrete composition and the curing procedures, and 'densify' the concrete with time. This densification will be depth dependent to some extent, less the better the concrete.

8.4.9 Methods for measuring transport properties

For every possible transport property a large number of test methods exist. Most of them are well described by Kropp and Hilsdorf (1995). Here only the main principles and applications are pointed out and some comments are given.

Steady-state methods 1: cup methods, cell methods

The main principle is to apply a potential difference over a specimen with a certain thickness and then measure the steady-state flux. Equation (8.1), or one of equations (8.7), (8.12) or (8.13), is then used to evaluate a steady-state transport property. The methods may involve various transport mechanisms depending on the applied conditions on the two sides of the specimen.

The potential difference could be an applied water pressure, air or gas pressure, water vapour or gas concentration, ion concentration, electrical field etc. Special care must be taken to ensure that steady-state conditions really are reached.

The method is widely used for measuring water permeability, gas and air permeability and moisture transport coefficients for concrete. In a few cases a steady-state diffusion cell was used for chloride and other ions, especially for cement paste specimens, but the time to reach steady-state conditions for concrete may be several years.

In recent years a number of migration cell methods are developed where an electrical field is applied to a concrete specimen. In some of them the steady-state flux of ions could be measured within a few weeks. The 'ion diffusion coefficient' D_i in equation (8.13) is, however, difficult to evaluate since all ions involved in the test will affect the flux of a single ion!

Steady-state methods 2: profile

The main principle is to apply a potential difference over a specimen with a certain thickness and measure the steady-state distribution of the substance and the flux (cf. Figure 8.5). Equation (8.1) is then used to evaluate a steady-state transport property from the measured gradients of the distribution of substance.

The method is very time consuming for concrete (years!) since it requires a certain thickness of the specimen to be able to measure the profile. It has been used for moisture in rare cases (Hedenblad, 1993).

Non-steady-state methods 1: ingress profiles

The main principle is to measure the ingress profile of a substance at a certain time or as a function of time (cf. Figure 8.16). Equations (8.3) or (8.15) are then applied for evaluating a 'diffusivity'.

The method is mostly used for determining apparent chloride diffusion coefficients (cf. section 8.4.5) A specimen is immersed in a chloride solution for a certain time of exposure. A ‘chloride profile’ is then determined by profile grinding and analysing the total chloride content at each depth interval.

The same principle could be applied for other substances, i.e. moisture, other ions etc. but equation (8.3) might have to be changed into a non-constant diffusion coefficient (cf. equation 8.4)).

Non-steady-state methods 2: weight gain or loss

The main principle is to measure the consumption/absorption of a substance at a certain time or as a function of time. Equation (8.9), or other more complicated solutions to the mass balance equation, are then applied for evaluating an absorption coefficient.

The typical laboratory test method is the capillary suction test where a concrete specimen is placed in contact with a free water table to let it suck water. The absorption of water is measured by a balance. In other test methods the non-steady-state water or gas flow are determined by measuring the amount of water being absorbed or the drop in gas pressure.

Non-steady-state methods 3: penetration depth

The main principle is to measure penetration depth of a substance at a certain time. An equation similar to equation (8.9) is then applied for evaluating a ‘diffusivity’ or absorption coefficient or ‘permeability’.

The type of method is mostly used for carbonation, accelerated or in field tests, and for water penetration. In recent years a number of migration cell methods are developed where an electrical field is applied to a concrete specimen. In some of them the depth of penetration is measured within a few days (cf. Figure 8.18). The ‘ion diffusion coefficient’ D_i in equation (8.13) is, however, difficult to evaluate since all ions involved in the test will affect the flux of a single ion. Additionally, in a non-steady-state test, the binding capacity will be decisive for the depth of penetration.

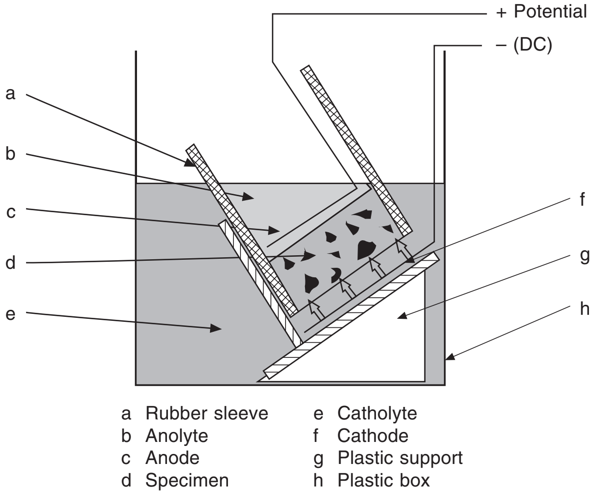

Figure 8.18 The non-steady state electrical migration test method NT Build 492 for rapid chloride penetration test.

8.5 Summary and conclusions

The service life and long-term behaviour of concrete structures, and concrete repair materials, are to a large extent controlled by a number of transport processes in the concrete pore system. Those transport processes will affect the time-dependency of the performance of concrete and reinforcement. A brief overview has been given of some significant transport processes in concrete, mainly moisture variations, carbonation and chloride ingress. Today's understanding of a number of transport processes is excellent. Decisive material properties for some flow and binding processes are still lacking to a large extent and some observations are still quite difficult to explain.

The transport processes in real life, in field structures, are, however, far from being correctly quantified and predicted. A major reason for this lack of relevance is the very complicated boundary conditions that physical models require. Real climatic conditions are extremely difficult to 'translate' into time-varying surface humidity and surface chloride concentration.

Until better models for environmental actions are available, empirical models for transport processes that are based on curve-fitting measured substance profiles will be prevalent.

Moisture transport

Most knowledge and quantification of moisture transport processes in concrete structures are based on fundamental theory and experimental studies under controlled laboratory conditions. In spite of this, moisture transport under a temperature gradient is hardly studied, which adds much uncertainty in understanding and describing moisture conditions in outdoor structures. Very little practical verification of theoretical models in field structures exists. One reason is the difficulties in performing such measurements. Another is the great scatter and variations in climatic actions.

Carbonation

Excellent models for carbonation are available today where the variations in the environmental actions can be considered. The most important factor that determines the rate of carbonation is the humidity in the carbonated zone. A better description of the carbonation process requires significant research on predicting the duration of wet and dry periods at the concrete surface and the moisture variations in the concrete that follows.

Chloride transport

Physical prediction models for chloride transport processes are now very promising since they consider the interaction of all ions, not only chloride. In this way a number of observations have been possible to explain. However, the time effect in chloride binding is still not understood. Non-saturated flow, including the convection of chloride with liquid moisture, still lacks much data on material properties and a lot of data and models for describing the boundary conditions.

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9

Reinforcement corrosion

G.K. Glass

9.1 Introduction

Corrosion of reinforcing steel in concrete is the most significant deterioration process affecting reinforced concrete structures. It has, for example been estimated that the remediation of concrete bridges in the USA undertaken as a direct result of corrosion of the reinforcing steel cost the US state departments $5 billion in the year 2000 (Federal Highway Administration, 1999). In addition to bridges, other highway structures, buildings and marine facilities are affected. An estimate of the cost incurred in the UK in 1997 as the result of all forms of corrosion damage to concrete was £750 million (BRE, 2001).

The two most important causes of corrosion of the reinforcing steel are carbonation and chloride contamination of the concrete. Carbonation is a problem that mainly affects buildings. Chloride contamination affects structures that are exposed to de-icing salts or marine environments. Other less common causes of corrosion are acidic gases such as sulphur dioxide, aggressive ions such as sulfates, fluorides and bromides and stray electrical currents.

Corrosion is accompanied by a loss of rebar cross-section and a build-up of corrosion products. The corrosion products occupy a larger volume than the original metal from which they were derived (Figure 9.1). This generates the tensile stresses causing cracking and spalling of the concrete cover. Very often the first indication of a problem is the appearance of a crack following the line of reinforcement (Figure 9.2).

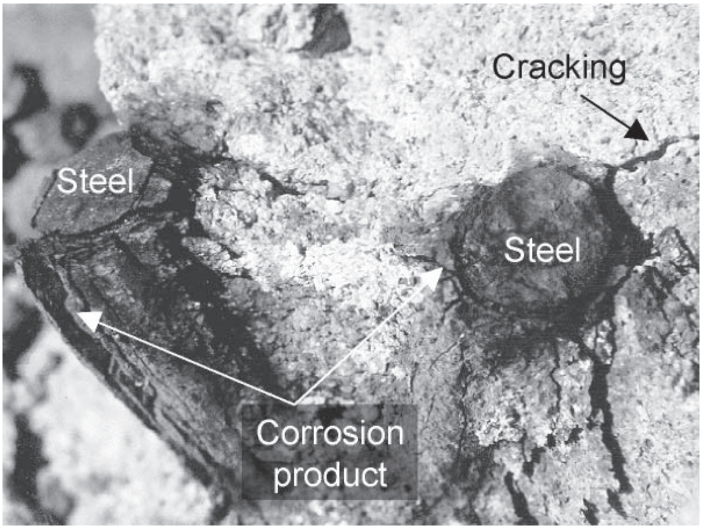

Figure 9.1 Expansive corrosion products on steel in chloride contaminated concrete.

Figure 9.2 Corrosion induced by carbonation of a concrete beam which has resulted in a crack in the concrete cover following the line of the reinforcement.

9.2 The corrosion process

Corrosion may be defined as the deterioration of a metal by reaction with species in the environment to form chemical compounds. At room temperatures it is an electrochemical process; that is, the chemical reactions occur in conjunction with the flow of an electric current. This is illustrated in Figure 9.3.

The metal dissolves at one site to form positive ions leaving free electrons in the metal. A reaction that results in a more positive product is termed **oxidation**, and the location

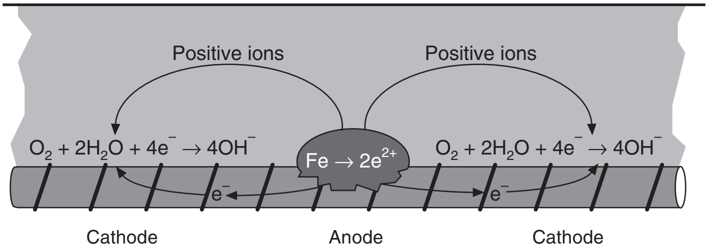

Figure 9.3 Schematic diagram of the electrochemical processes occurring during corrosion.

where oxidation occurs is called an **anode**. For example, iron dissolves (is oxidized) to form positive iron ions. The residual electrons in the metal are consumed by a reaction which produces a more negative product. Such a reaction is termed **reduction** and the location where reduction occurs is called the **cathode**. For example, oxygen is reduced to form negative hydroxyl ions. An electronic conductor on which an oxidation or reduction reaction occurs is termed an **electrode**.

Between the anode and cathode, ionic and electronic current must flow to prevent the build-up of charge. The ionic conductor is termed an **electrolyte** and is usually an aqueous solution of ions (e.g. Fe^{2+}, OH^-, Na^+, Cl^-). The positive ions are termed **cations** and the negative ions are **anions**. The metal provides the electronic conductor. If there is no external source of electrons (provided by an external power supply), the anodic oxidation reaction must generate electrons at exactly the same rate as the cathodic reduction reaction consumes them and the flow of electrons in the metal and ions in the environment between the anode and cathode must prevent any accumulation of electric charge.

Two important ionic species present in all aqueous environments are hydrogen ions (H^+) and hydroxyl ions (OH^-). When combined these ions produce water. Most common cathodic reactions result in the generation of hydroxyl ions. An excess of hydroxyl ions is termed **alkalinity** (a more alkaline environment is produced at the cathode). By contrast, the iron ions produced at the anode may react with water (hydrolysis) to produce hydrogen ions (**acidity**) which will lead to acidification of the local environment at the anode. The processes involved in the formation of rust include the precipitation of iron ions as iron hydroxide and the oxidation of the iron hydroxide by further reaction with oxygen.

A potential difference (voltage) is the driving force for an electrochemical reaction. A more positive potential results in a higher oxidation state and the release of more electrons (positive ions are repelled from a more positive electrode surface). Likewise, a more negative potential will drive negative ions away from an electrode. An oxygen electrode has a more positive potential than an iron electrode. Thus, iron dissolves as positive ions because its potential is raised by the couple to an oxygen electrode, and oxygen is reduced to negative hydroxyl ions because its potential is lowered by the same couple. The potential of an electrode supporting an electrochemical reaction can be measured relative to a stable standard reference electrode.

The stability of compounds in a given environment and potential range is determined by thermodynamics. In the case of iron, a number of possible electrochemical reactions exist. The most stable products as a function of potential and pH for a fixed set of environmental conditions are given in a Pourbaix diagram illustrated in Figure 9.4. This diagram does not indicate the rate at which the most stable state will be achieved. However, if the most stable products at a given electrode potential and pH are insoluble oxides, the risk of high corrosion rates is lower than if the most stable products are soluble ions. If the most stable product is the metal then corrosion is thermodynamically impossible. Also included on this diagram is the potential – pH relationship of the oxygen reduction (line b) and hydrogen evolution (line a) reactions. The oxygen reduction reaction is the most important cathodic reaction stimulating the corrosion of reinforcing steel in concrete.

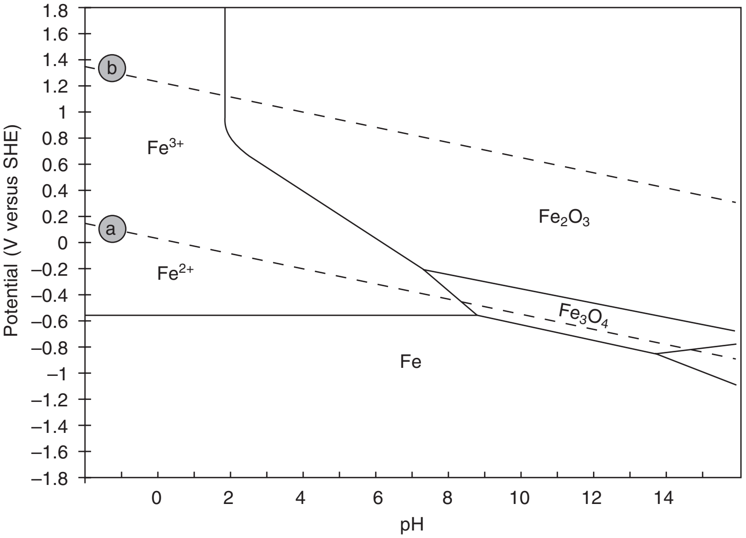

Figure 9.4 Simplified Pourbaix diagram for iron showing the most stable products at a given pH and potential.

9.3 The concrete environment

Concrete consists of a graded mix of aggregate particles in a cement paste matrix. The cement paste consists of unhydrated cement, hydration products and the residue of the water-filled space which gives rise to capillary porosity. The hydration products consist mainly of calcium silicate hydrate gel and calcium hydroxide. The various phases present are evident in the backscattered electron image of a polished cross-section of concrete containing a steel ribbon given in Figure 9.5. The grey scale in these images depends on the electron density of the material. The phases of interest, graded in terms of their brightness, are the steel (lightest) > unhydrated cement grains > calcium hydroxide (CH) > gel (predominantly calcium silicate hydrate (C–S–H)) ~ aluminate-bearing hydrates ~ aggregate > porosity and voids (darkest). The hydration products consist mainly of calcium silicate hydrate gel, which has a porosity of around 28 per cent (the gel pores), and calcium hydroxide.

Capillary pores are up to 1 μm in diameter, whereas gel pores are around 2 nm. Concrete may also contain entrained air, entrapped air and other voids. Intentionally entrained air voids are bubbles typically 0.1 mm in diameter and are distributed evenly throughout the cement paste. Accidentally entrapped air usually forms very much larger voids, often up to several millimetres in diameter. This will typically account for 2 per cent of the volume of the concrete. Other defects include internal cracks, voids under

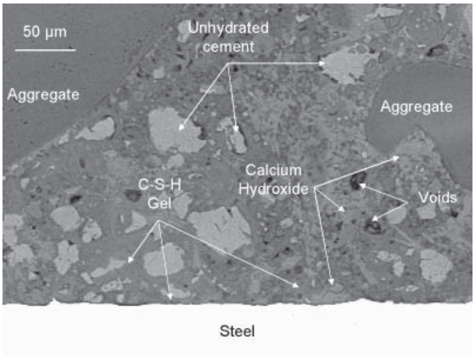

Figure 9.5 Backscattered electron image of a polished cross-section of steel in concrete (Grass *et al.*, 2001).

aggregate particles created by bleed water in concrete, and 'honeycomb' voids (Glass and Buenfeld, 2000a).

The **pore solution** is an aqueous electrolyte that may be present in the pores and larger defects. It contains ions such as sodium (Na^+), potassium (K^+), calcium (Ca^{2+}), hydroxyl (OH^-) and sulphate (SO_4^{2-}), as well as dissolved oxygen. Thus, the necessary reactants are present to permit corrosion of the embedded steel. However, a significant feature of cement hydration is that the aqueous phase rapidly acquires a high pH. Furthermore, the material contains a substantial portion of reserve alkalinity in the form of sparingly soluble hydroxides that resist downward pH changes at values above 10. When steel is in contact with such an alkaline solution, the thermodynamically most stable products of any reaction between steel, water and oxygen are insoluble oxides (cf. Figure 9.4). A film consisting of a metal-oxide layer covers the steel and presents a barrier to further metal dissolution. The steel is said to be **passive** and the oxide film is termed a passive film.

Passivation is thus the primary mechanism of corrosion protection for steel in concrete. No significant corrosion will occur if this environment remains intact. However, concrete is exposed to a wide variety of external environments and the ingress of aggressive species may render it corrosive.

9.4 Stages in the deterioration process

A schematic diagram of the process of corrosion-induced deterioration is given in Figure 9.6. Deterioration starts with the loss of protection provided by the concrete cover. This is followed by corrosion initiation and then propagation.

The loss of protection provided by the concrete cover occurs as the result of the ingress of aggressive species such as chloride ions and carbon dioxide (see aggressive species listed in section 9.1). These species are transported through the pore system in the cement paste. At low porosities, entrained and entrapped air voids are distinct cavities isolated from one another. This will restrict their influence on the transport of aggressive species. When they are not filled with solution, they will act to block the solution transport of ions while promoting transport in the gaseous phase. Cracks and other large voids may be oriented to produce a continuous network through the concrete thereby significantly

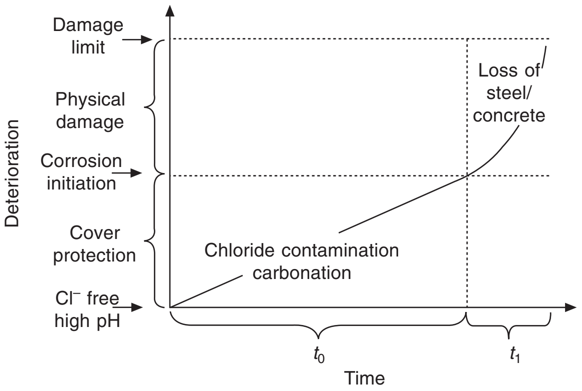

Figure 9.6 Schematic diagram of corrosion-induced deterioration of reinforced concrete.

reducing resistance to transport. If such large defects are absent, capillary pores will have the dominant effect on the transport of aggressive species into concrete.

The rate of any transport process will depend on the volume fraction, tortuosity and connectivity of the pores. This is determined by factors such as the water/cement ratio (w/c), cement content, cement fineness, cement type, use of cement replacement materials (for example ground granulated blast furnace slag (ggbs), pulverized-fuel ash (pfa) and silica fume (sf)), concrete compaction, and degree of hydration (Atkinson and Nickerson, 1984). Reducing w/c (which controls the original spacing of the cement grains) and prolonged hydration may, for example, result in the capillary pores becoming blocked by gel so that they are interconnected solely by gel pores.

It may be noted that no physical damage occurs while aggressive species contaminate the concrete cover. Indeed a gain in concrete strength may occur. Nevertheless the loss of protection provided by the concrete cover is a deterioration process that reduces the remaining maintenance-free service life. The period of this first stage normally forms a substantial proportion of the service life before the first maintenance is necessary. As indicated above, this period is strongly dependent on the cover depth and the properties of the cover concrete affecting the rate of transport of the aggressive species.

The second stage of the deterioration process involves corrosion propagation. The transition between the first and second stages is precipitated by the breakdown of the passive film resulting in corrosion initiation. During the second stage a loss of steel section occurs. Furthermore, because the high volume corrosion products exert tensile forces, spalling of the concrete cover may occur (cf. Figure 9.1). This may affect the integrity of the structure by reducing the tensile and bond strength of the steel. In addition a safety hazard may result from the falling debris.

More detail on the stages in the deterioration process is given in the sections below.

9.5 Carbonation-induced corrosion

9.5.1 Concrete carbonation

When atmospheric carbon dioxide (CO_2) dissolves in the cement pore solution, carbonic acid (H_2CO_2) is formed. A reduction in the pH of the concrete pore solution then occurs and some of the alkaline solid phases are neutralized. For example, calcium hydroxide ($Ca(OH)_2$) will be converted into calcium carbonate ($CaCO_3$). As the reserve levels of the alkaline solid phases are depleted, a zone of low pH (the carbonated zone) extends from the surface into concrete. This process is termed **carbonation**. A fall in pH to values below 10 at the steel may render the steel passive film thermodynamically unstable Figure 9.4.

Factors associated with both the concrete and the external environment affect the rate of carbonation. The nature of the porosity and the alkaline reserves of the cement hydration products are the main factors associated with the concrete that affect carbonation. Thus, for example, a high water/cement ratio will increase the capillary porosity and the rate of carbonation (Figure 9.7). Cracks resulting from tensile stresses in the concrete will also increase the carbonation depth.

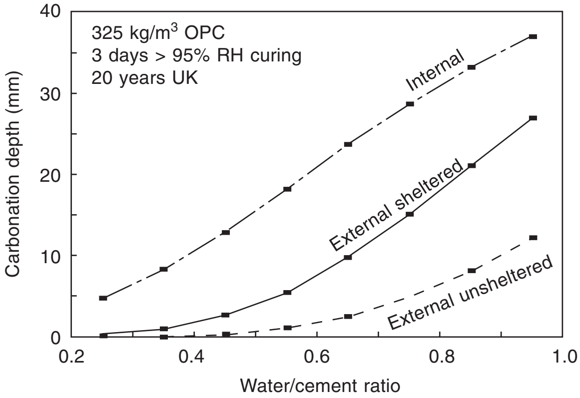

Figure 9.7 The effect of free water/cement ratio and external exposure environment on the depth of carbonation in concrete (Hassanein, 1997).

Carbon dioxide reacts with all the major constituents of hydrated cement. This includes both calcium hydroxide and the calcium silicate hydrate gel. Many solid phases will release hydroxyl ions as the pH is reduced. These alkaline reserves in hydrated cement depend on the cement type (Sergi and Glass, 2000; Glass *et al.,* 2000a). Table 9.1 shows the acid required to reduce the pH of aqueous suspensions of ground cement paste and concrete to a pH of approximately 10. It is apparent that blending ordinary Portland cement (opc) with pfa or ggbs may substantially reduce the alkaline reserves in the hydrated paste and therefore reduce the resistance to carbonation.

Table 9.1 Acid (moles H^+ / kg binder) required to reduce the pH to the phenolphthalein endpoint together with values extrapolated from the titration data at pH 10

Cementitious binder	Acid neutralization capacity (moles/kg)		
	Cement paste (Sergi and Glass, 2000)		Concrete (Glass *et al*., 2000a) (pH 10)
	(Phenolphthalein)	(pH 10)	
OPC	19.8	19.8	18.9
SRPC	19.1	18.6	17.5
30%PFA	13.1	–	15.4
65%GGBS	13.0	13.0	14.5

A very important factor affecting the rate of carbonation is the moisture content of the concrete. This is determined by the environment to which the concrete is exposed. Water is necessary for the carbonation reaction, but if the pores are filled, the ingress of carbon dioxide is severely hindered (Hassanein, 1997). Thus concrete exposed in dryer environments carbonates more quickly (Figure 9.7).

Models of the carbonation process may be formulated by assuming that it is controlled by diffusion of carbon dioxide through concrete and reaction with the constituents of hydrated cement. However, the diffusion coefficient for carbon dioxide is difficult to measure as the carbonation process is slow and results in a change in the pore structure. Simple models of the rate of carbonation of concrete often assume that the carbonation depth is proportional to the square root of time (t).

$$\text{depth} = k\sqrt{t} \tag{9.1}$$

The coefficient of proportionality (k) has to be determined for each concrete/environment combination. Examples of measured relationships are given in Figure 9.8.

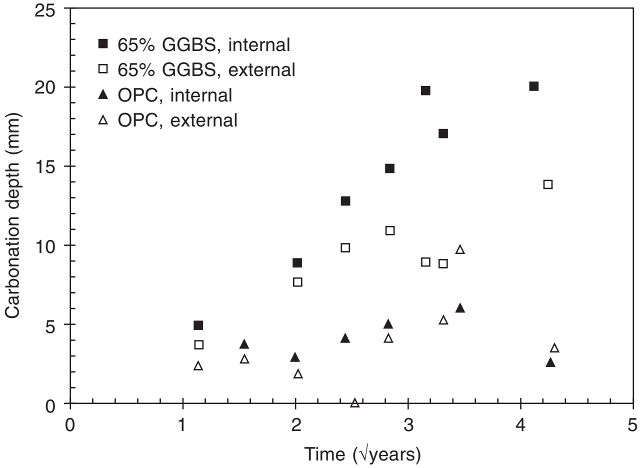

Figure 9.8 Measured natural carbonation depths over a 20-year period in 65 per cent GGBS and OPC concretes plotted as a function of the root of time (Hassanein, 1997).

9.5.2 Corrosion initiation and propagation

After the depth of carbonation has reached the reinforcing steel, the passive film is no longer stable. However, significant rates of corrosion are not automatic. As indicated in Figure 9.3, corrosion requires the presence of an electrolyte to conduct ions between the anodic and cathodic sites. This is normally provided by moisture. In dry concrete the resistance of the environment to the flow of ionic current may be high. Increasing the moisture content increases the risk of significant corrosion occurring (Figure 9.9) but it may reduce the carbonation depth (Glass *et al.,* 1991).

The binding capacity for aggressive ions such as chloride is also affected by carbonation. Bound chloride ions are released as the pH of the concrete is reduced. This reduces the concrete resistivity and helps to retain moisture. Thus the presence of low levels of chlorides exacerbates the carbonation problem. Severe corrosion problems are encountered

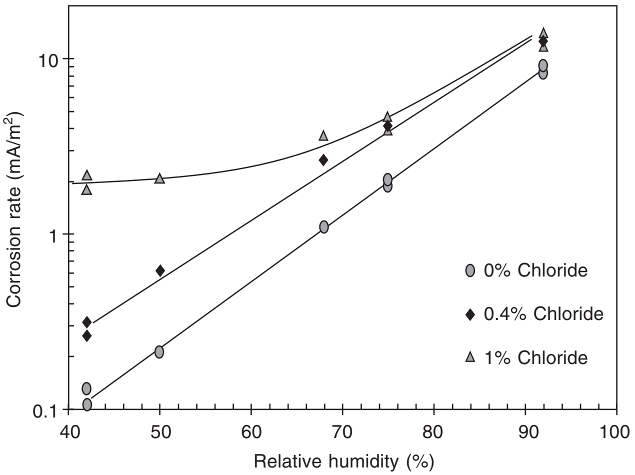

Figure 9.9 Steel corrosion rate as a function of chloride content and relative humidity in carbonated OPC mortar (Glass *et al.*, 1991).

when carbonation is accompanied by high levels of chloride contamination. However, this is unusual as the dry conditions, which promote high carbonation rates, are not compatible with wet conditions which promote high rates of chloride contamination.

An example of carbonation-induced corrosion is given in Figure 9.10 which shows a beam removed from a building. The depth of carbonation is shown by the indicator phenolphthalein. This changes colour (from pink to colourless – dark to light grey in Figure 9.10) when the pH decreases below 9.

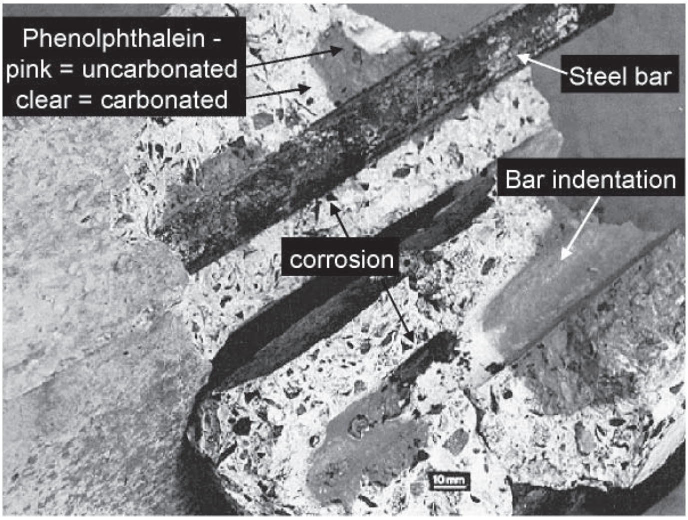

Figure 9.10 A carbonated concrete beam showing areas of corrosion.

9.6 Chloride-induced corrosion

9.6.1 Chloride contamination

Chloride contamination of concrete may arise from both internal and external sources. Internal sources of chloride include contamination of the mix materials and the use of calcium chloride as a set accelerator in construction. Limitations are placed by current codes of practice on the acceptable levels of chloride contamination resulting from the use of contaminated mix materials, while the use of chloride containing admixtures for reinforced concrete is generally not permitted (BSI, 1988). Thus internal chlorides tend to only affect older existing structures. External sources of chlorides include de-icing salts and sea salt in marine environments (Bamforth *et al.*, 1997).

Chloride ingress into concrete can occur by a number of mechanisms. This includes **diffusion** due to a concentration gradient, **migration** in an electric field, and **water flow** (Bamforth *et al.*, 1997). The rate of diffusion is described by the diffusion coefficient. This parameter gives the flux of a species (quantity passing through a unit area per unit of time) per unit of concentration gradient. Migration is determined by the ionic mobility. This gives the average velocity per unit of electric field. Water flow may result from a pressure gradient, absorption into partially dry concrete, wick action (absorption at one location with drying at another location) or electro-osmosis (the movement of water under the influence of an electric field) each of which may have its own transport coefficients (Glass and Buenfeld, 2000a).

As indicated above, chloride transport is affected by the pore structure. The physical effect of the pore structure is generally incorporated into the measured values of the transport coefficients (e.g. the diffusion coefficient or ionic mobility) defining the various transport processes. In addition, chemical interactions between ions in the pore solution and with the pore walls affect the movement of chloride ions through concrete.

The two main effects on chloride transport arising from interactions between ions in the pore solution, are the effective concentration of a species may be altered and, the separation of charge is constrained (the condition of electroneutrality). The effective concentration of an ion resulting from its non-ideal behaviour is termed its **activity**. The theoretical description of the transport of a species may, to a good approximation, be expressed in terms of its concentration only when the concentration is low.

The condition of electroneutrality implies that, when no ions are produced or consumed, the net quantity of positive charge transported by cations must be equal to the negative charge transported by anions. If this were not the case a local charge would continue to build up. The transport of an excess of ions of one particular type (e.g. Cl^-) in a given direction may therefore only occur if other ions of the same sign (e.g. OH^-) are transported in the opposite direction (Yu *et al.*, 1993). Thus, when sodium chloride diffuses into concrete, chloride diffusion into the concrete might be retarded by its association with a slower charge balancing sodium ion if no significant counter diffusion of hydroxyl or sulphate ions out of the concrete occurs.

The two main interactions between ions and the pore walls are chloride binding and membrane effects (Glass and Buenfeld, 2000b; Zhang and Buenfeld, 2000). **Chloride binding** in concrete may be defined as the interaction between the porous matrix and chloride ions which results in their effective removal from the pore solution. All cements bind a proportion of the chloride present. This will remove chloride from the transport process as well as alter the pore solution concentration and therefore the concentration gradient driving diffusion. While there are many factors associated with the constituents of the concrete, the composition of the pore solution and the external environment that affect chloride binding, the most important factor is the cement type (Glass *et al.*, 1997).

Ion exchange membranes are produced by a surface charge on the pore walls. This charge on the pore walls is balanced by an equal and opposite charge in the pore solution. For narrow pores or low concentrations, only charge balancing ions may be present in the pore solution. An ion exchange membrane therefore favours the transport of the charge balancing species through its pore system (Kyrota, 1992). As a result, when both cations and anions attempt to diffuse through a membrane, the preferential diffusion of either anions or cations will produce a potential difference across the membrane. This is termed a **membrane potential**. The sign and value of the surface charge on the pore walls in concrete will depend on the electrolyte composition and pH, as well as on the binder type and composition of the hydration products (Glass and Buenfeld, 2000a; Zhang and Buenfeld, 2000). It may be noted that, even in the absence of membrane effects, a difference in the mobilities of the anions and cations will produce a small potential difference termed a **junction potential** if diffusion occurs.

9.6.2 Modelling chloride penetration

Models of chloride penetration into concrete may be based on mathematical models of the physical transport processes or an analysis of empirical data. A physical model considers the **flux** of an ion. This is defined as the quantity passing through a unit area per unit of time. In an ideal solution the flux (J) is given by a form of the Nernst–Planck equation:

$$J = -D_{\mathrm{p}}\frac{\partial C}{\partial x} - \upsilon C\frac{\partial \phi}{\partial x} + Cu \tag{9.2}$$

where C, D_{p}, υ, ϕ, and u are the concentration, diffusion coefficient, ionic mobility, electric potential and average mass velocity respectively (Weber *et al.*, 1991). The three terms on the right-hand side of equation (9.2) represent the flux due to diffusion, migration and water movement respectively. The time dependent spatial distribution of the ion is then given by a solution of the equation:

$$\frac{\partial C}{\partial t} = \frac{\partial J}{\partial x} + r \tag{9.3}$$

where r is the rate of production/consumption of the species (Glass and Buenfeld, 2000a).

The model may be simplified if one transport process dominates chloride ingress. If, for example, this is diffusion and the surface chloride concentration (C_0) and diffusion coefficient are constant (independent of time), and chloride binding may be neglected ($r = 0$), equations (9.2) and (9.3) have an exact solution defined by the error function (erfc). For diffusion into a semi-infinite medium this is given by:

$$C(x, t) = C_0 \operatorname{erfc}\left(\frac{x}{2\sqrt{D_{\mathrm{p}}t}}\right) \tag{9.4}$$

It should be noted that the diffusion coefficient may take various forms. The pore system diffusion coefficient (D_p) in equation (9.4) characterizes the velocity of ions in the pore system under the influence of a concentration gradient. The ionic flux is determined relative to the cross-sectional area of the pores carrying the ions (Glass and Buenfeld, 1998). However, as noted above, all concrete will bind some chloride thereby changing the chloride profile. If the effects of linear chloride binding are included in equations (9.2) and (9.3), the pore system diffusion coefficient will be replaced by the apparent diffusion coefficient (D_a). This is widely used to model the chloride profile as shown in Figure 9.11 (Bamforth *et al.*, 1997).

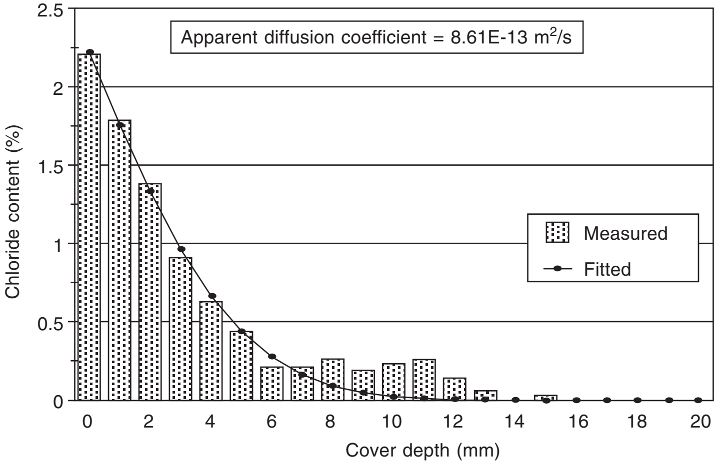

Figure 9.11 Chloride profile in concrete.

Empirical models are based on an analysis of measured data. For example, many chloride profiles follow the form given by equation (9.4) even though the underlying assumptions are incorrect. This provides an empirical basis for the use of equation (9.4) to describe the chloride profile. It is important to note that, even though the chloride profile is characteristic of a diffusion process, it is not necessary to assume that it was caused solely by diffusion (Engelund and Sorensen, 1998).

Equation (9.4) has also been used in an empirical manner in the development of probabilistic models of chloride ingress into concrete. The ingress of chloride, corrosion initiation and the subsequent propagation of corrosion damage depends on a number of factors that may only be assessed with substantial uncertainty. The uncertainty arises not only from uncertainties in the material characteristics and environmental conditions, but also from uncertainties in the mechanism of the deterioration process. A probabilistic model attempts to address these uncertainties. Such models are gaining acceptance as a method of assessing risk of failure (Faber *et al.*, 1999).

9.6.3 Chloride-induced corrosion initiation

The oxides which make up the passive film on iron are thermodynamically stable in the alkaline environment in concrete even when chloride ions are present (Page and Treadaway, 1982). In this situation corrosion tends to be localized and chloride induced-corrosion initiation in concrete follows the model of **pitting corrosion**. It is a two-stage process in which pit nucleation is followed by pit growth (Glass *et al.,* 2000b). The causes of pit nucleation are still subject to much debate. However, it is widely recognized that pit nucleation is usually followed by repassivation. If the pits are to grow, pit nucleation must be accompanied by a local fall in pH and increase in chloride content at the pit nucleation site. The local fall in pH occurs as the result of the hydrolysis of dissolved iron ions. An example of such a reaction is given by:

$$Fe^{2+} + 2H_2O \rightarrow Fe(OH)_2 + 2H^+ \tag{9.5}$$

The presence of an excess of chloride ions provides the charge balancing anion to stabilize the local reduction in pH. Hydrochloric acid (HCl) is effectively formed (Figure 9.12). The presence of chloride ions promotes the continued dissolution of iron.

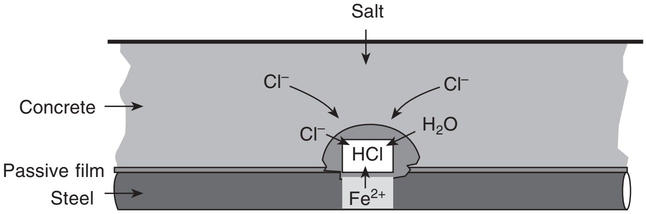

Figure 9.12 Schematic representation of the process of corrosion initiation arising from chloride contamination.

The effect of this mechanism is evident at voids at the steel-concrete interface. It was noted that corrosion tends to initiate at the location of such defects (Figure 9.13). This has been attributed to the absence of solid phases on the steel at these locations that would otherwise release hydroxyl ions to prevent a local fall in pH thereby inhibiting corrosion initiation (Page, 1975). Titrametric methods have been developed to determine the resistance to pH reduction that gives rise to this inhibitive property of the solid phases. In one method, termed differential acid neutralization analysis, the resistance to pH reduction is given by the acid added per unit of pH reduction of an aqueous suspension of ground

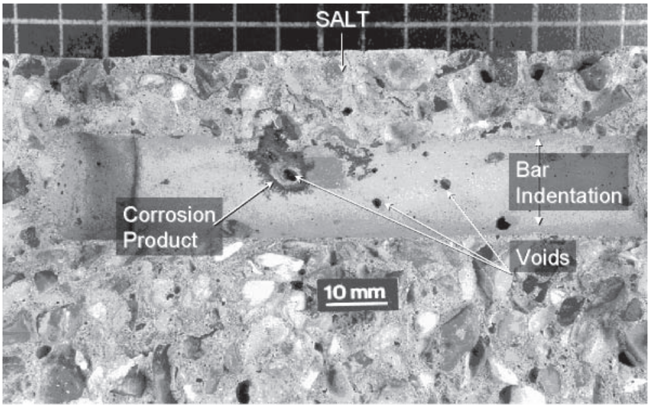

Figure 9.13 Corrosion initiation evident from the corrosion product on the concrete surface at the location of a void at the steel–concrete interface.

solid. An example of the data obtained on an aqueous suspension of ground OPC concrete that had previously been exposed to a neutral chloride solution is given in Figure 9.14. The dissolution of the solid phases gives rise to peaks in the data at various pH values. On the basis of such data, inhibitive effects have been attributed to many hydration products of cement that release hydroxyl ions into the pore solution at high pH values (Glass *et al.*, 2000a, 2001a).

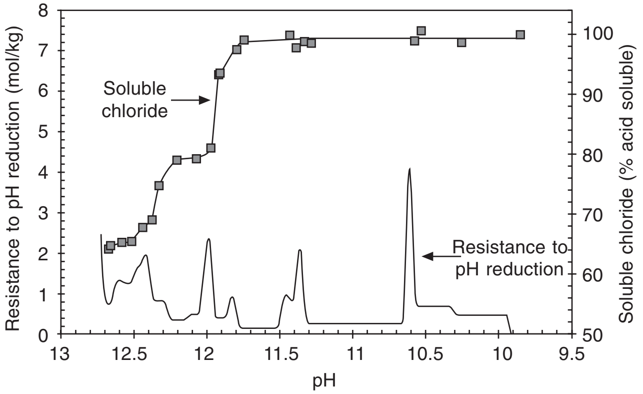

Figure 9.14 The resistance to a reduction in pH and soluble chloride content determined on chloride contaminated OPC concrete (Glass *et al.*, 2000).

Chloride ions bound in the solid phase may act to promote corrosion via the same mechanism that operates when hydroxyl ions bound in the solid phase act to inhibit corrosion (Glass *et al.*, 2000b; Glass and Buenfeld, 1997a). The pH-dependent dissolution characteristics of chloride are included in Figure 9.14. It was noted that, for this sample, virtually all the chloride that was bound in the solid phases was released before the pH reduced to 11. As the passive film is still thermodynamically stable at this pH, it might be assumed that all the chloride released will be available to sustain pit growth. Evidence of the participation of bound chloride in corrosion initiation comes from the absence of any significant correlation between chloride binding and chloride threshold level (Glass *et al.*, 1999). However, there is substantially more bound hydroxide than bound chloride which gives the solid phases a net inhibitive effect.

The **chloride threshold level** may be defined as the quantity of chloride at the steel that is necessary to sustain local passive film breakdown, and hence initiate the corrosion process. It is usually presented as a ratio of the chloride to cement content of concrete (expressed as a weight percentage). Typical values range between 0.2 per cent and 2.5 per cent by weight of cement (Glass and Buenfeld, 1997a). While the chloride content is relatively easy to determine, it should be noted that the cement content is often only estimated as laboratory verification of this is more difficult. The acid neutralization capacity of a suspension of ground concrete can give an indication of the cement content or it may be directly used in an alternative representation of the chloride threshold level that has been proposed (Sergi and Glass, 2000).

Low chloride threshold levels are often observed under site conditions. It is believed that these are the result of the defects such as voids, millscale and rust on the steel. The presence of such defects combined with moisture (the electrolyte) and oxygen, which raises the steel potential to positive values will limit the amount of chloride that is necessary to cause passive film breakdown. In technical terms such defects are said to lower the **pitting potential** of steel. This is the potential at which local breakdown of the passive film occurs. It generally shifts to more negative values as the chloride content increases. Because the condition of the steel–concrete interface is seldom characterized, the associated variations in the chloride threshold level appear to be random. Thus the chloride threshold level may be regarded as representing a risk of corrosion initiation. The risk of corrosion initiation occurring on UK bridges as a function of the chloride content may be estimated from the data in Figure 9.15 (Vassie, 1984).

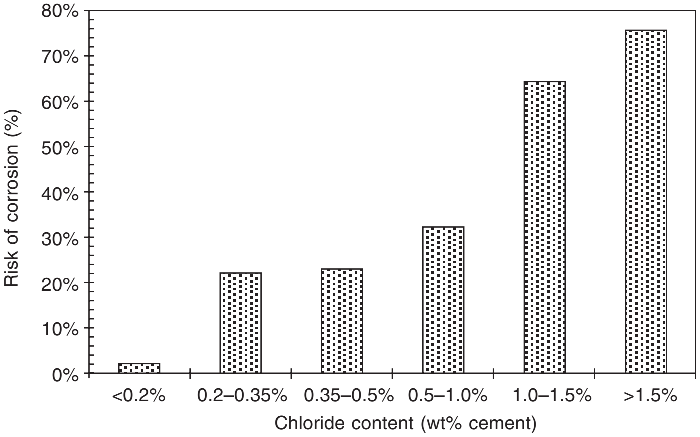

Figure 9.15 Risk of corrosion initiation as a function of the chloride content (Vassie, 1984).

In addition to the condition of the steel–concrete interface, other factors may affect the chloride threshold levels to a smaller extent (Glass and Buenfeld, 1997b). These include factors associated with the external environment (moisture, temperature and chloride source), the barrier properties of the concrete cover (w/c and cover depth) and the chemical properties of the cementitious binder. Corrosion initiation may be prevented if the concrete is too dry, while in very wet environments the corrosion rate following initiation may be limited by the restricted access of oxygen. One of the effects of a dry environment may be to prevent moisture entering the entrapped air voids at the steel.

9.7 Other causes of corrosion

As noted above, the two most important causes of passive film breakdown, and therefore corrosion of steel in concrete, are a reduction in the local pH resulting from carbonation, and the disruption of the passive film caused by chloride contamination. Other contaminants may similarly induce corrosion. For example, acidic gases such as sulphur dioxide may give rise to corrosion via a similar mechanism to that operating when carbonation occurs. Aggressive ions such as sulphates, fluorides and bromides, may render the passive film unstable via a similar mechanism to that operating when chloride contamination occurs. In addition stray electrical currents may cause corrosion by forcing some areas of the steel to become anodic. These causes of corrosion induced deterioration are however not common and are restricted to environments such as concrete chimney stacks, chemical plant and concrete adjacent to DC traction railways and trams.

9.8 Corrosion rate

9.8.1 Typical values

Corrosion rates may be expressed as a current density, a rate of weight loss or a rate of section loss. For steel, a corrosion current of 1 mA/m^2 approximates a weight loss of 10 g/m^{2}yr which in turn approximates a section loss of 1 μm/yr. Reinforcing steel corrosion rates of 1 mA/m^2 or less are considered to be negligible and are not likely to result in disruption of the concrete cover. The corrosion rate of passive steel in concrete is approximately 0.1 mA/m^2.

Once passive film breakdown has occurred, a corrosion cell is established (Figure 9.3). An ionic current flows between the anodic areas, where metal dissolution is occurring, and the cathodic areas where oxygen is being reduced. The increase in chloride content and the production of acid at the anodic areas results in further disruption of the adjacent passive film. If oxygen has relatively easy access to the steel (the pore system is not saturated with water), the general corrosion rate may exceed 100 mA/m^2. Localized corrosion at rates of up to 1000 mA/m^2 may occur.

The essential processes in a corrosion cell on steel which result in the production of corrosion products are an anodic reaction (metal dissolution), a cathodic reaction (typically oxygen reduction), the movement of electrons through the metal between the anodic and cathodic sites and the movement of ions through the environment between these sites. The rate of corrosion may be controlled by any one of these processes. However, the resistance to electronic current flow within a rebar is usually very small. Thus in practice the rate at which a metal is consumed can be controlled by the kinetics of either the anodic or cathodic reactions, or by the resistance to current flow between the anodic and cathodic sites through the concrete. The associated controlling mechanisms are termed anodic control, cathodic control or resistive control. Reinforced concrete is exposed to a wide variety of environments and any one of these three mechanisms may dominate depending on the exposure conditions (Glass and Buenfeld, 1996).

9.8.2 Anodic control

When the concrete is periodically allowed to dry out, oxygen has relatively easy access to the steel. The kinetics of the principal cathodic reaction, oxygen reduction, are said to be **weakly polarized** because this reaction can occur relatively easily. The presence of a passive film may restrict the overall rate by limiting the rate of metal dissolution (the anodic reaction kinetics). The anodic reaction kinetics are said to be **strongly polarized** and the corrosion rate is under anodic control. To increase the corrosion rate passive film breakdown must occur.

A typical relationship observed between the corrosion potential and the corrosion rate for a system under anodic control is given in Figure 9.16 (Glass *et al.*, 2001b). More negative corrosion potentials are associated with a higher corrosion risk. To explain this relationship, it should be noted that the corrosion potential is a **mixed potential** containing the effects of both the anodic and cathodic reactions. These two reactions are sometimes referred to as the **half reactions** making up the corrosion process. If the cathodic reaction kinetics remain unchanged, the cathodic reaction rate will only increase if the electrode potential is shifted to more negative values. This is because the reaction products are more negative than the reactants (e.g. O_2 is converted into OH^-). By contrast, if the kinetics of the anodic reaction remain unchanged, the rate of anodic dissolution will only increase if the electrode potential is shifted to more positive values. This is because the reaction products are more positive than the reactants (e.g. Fe is converted to Fe^{2+}). The natural potential difference that exists between the negative iron electrode and the positive oxygen electrode provides the driving force for the spontaneous corrosion process.

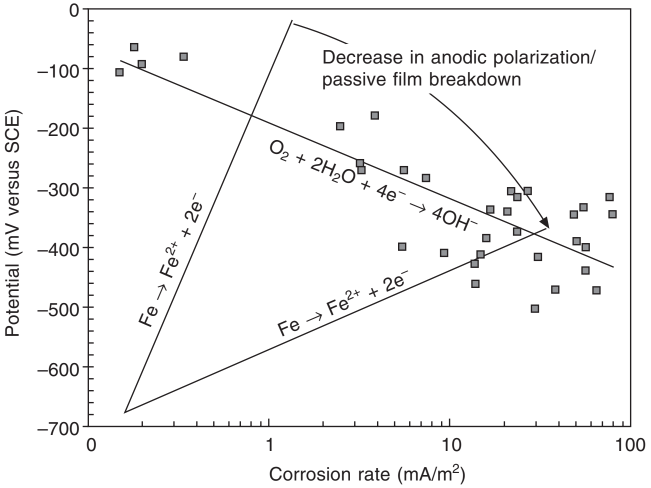

Figure 9.16 The typical relationship observed between the corrosion potential and the corrosion rate in reinforced concrete when the corrosion rate that results from a change in the resistance to the anodic reaction (Glass *et al.*, 2001).

The corrosion potential–corrosion rate relationship in Figure 9.16 may be explained by a change in the anodic reaction kinetics. The kinetics of the cathodic reaction, oxygen reduction, are assumed to be constant. When only the anodic reaction kinetics change, more negative corrosion potentials will be associated with higher corrosion rates. Such a change may be brought about by an increase in the chloride content at the steel which results in passive film breakdown.

9.8.3 Cathodic and resistive control

While anodic control is common for steel in concrete, it is also possible that the corrosion rate may be under cathodic or resistive control. Cathodic control arises when the passive film on the steel has largely been destroyed and the corrosion rate is controlled by the access of oxygen to the steel. Oxygen may, for example, have to diffuse through a waterlogged pore system. As it is an uncharged species this diffusion process is independent of the electrode potential. Its rate of diffusion corresponds to a **limiting current** for oxygen reduction on the steel.

When the cathodic reaction kinetics remain unchanged, an increase in the corrosion rate is accompanied by an increase in the corrosion potential. Such a relationship may be observed in waterlogged concrete that is allowed to dry out or when the reinforcing steel is coated with an insulating barrier coating (e.g. an epoxy coating) that restricts oxygen reduction and this coating degrades with time.

Resistance control may occur in dry or carbonated concrete. In this case the flow of ionic current between the anodic and cathodic sites on the steel restricts the overall corrosion rate. No clear corrosion rate–corrosion potential relationship exists as a potential difference equal to the voltage drop through the electrolyte occurs between the anodic and cathodic sites. However, in practice the measured potential tends to follow that of the electrode on which the cathodic reaction occurs. It is suggested that resistivity affects the anodic reaction kinetics more than the cathodic reaction kinetics and the controlling process is sometimes referred to as anodic resistance control.

9.8.4 Factors affecting the corrosion rate

As indicated above, the corrosion rate may be controlled by the anodic or cathodic reaction kinetics, or the resistance of the electrolyte in the concrete pores. The resistance presented by each of the controlling processes is termed **polarization**. A process is described as being strongly polarized if a large potential change only induces a low reaction rate equivalent to a small current (it presents a large resistance to the flow of current). In a spontaneous corrosion process, anodic, cathodic and resistance polarization will occur.

Factors affecting the corrosion rate include the chloride content, the access of oxygen to the steel and the resistivity of the environment. These are controlled by factors such as the waterlogged nature of the concrete, its porosity, other macroscopic defects in the concrete and the temperature and relative humidity of the external environment. For example, low levels of chloride contamination will affect the corrosion rate in dry or carbonated concrete via its direct effect on the resistivity of the environment and its indirect effect on restricting the drying process (Figure 9.9) (Glass *et al*., 1991).

To refine models of the risks associated with corrosion-induced deterioration of reinforced concrete, it is necessary to relate the corrosion rate to the risk of corrosion induced cracking of the concrete cover. This will depend on the volume of the corrosion product, the geometry of the concrete section and its reaction to the tensile stresses (Schiessl and Raupach, 1997).

9.9 Monitoring corrosion

Monitoring may be used to assess existing deterioration and predict future performance. An assessment of existing deterioration is a standard requirement in the repair of corrosion-damaged concrete (Concrete Bridge Development Group, 2000). The current condition of reinforced concrete may be defined by the state of the reinforcement and the deterioration and effective loss of the cover concrete. If the steel is corroding, spalling of the concrete cover can be a safety hazard, while the loss of steel cross-section and bond strength may affect load carrying capacity.

Visual inspection may be used to assess a problem in its advanced state, i.e. after cracking of the concrete cover has occurred. A flow diagram illustrating this process is given in Figure 9.17. It may be followed by further non-destructive and destructive testing.

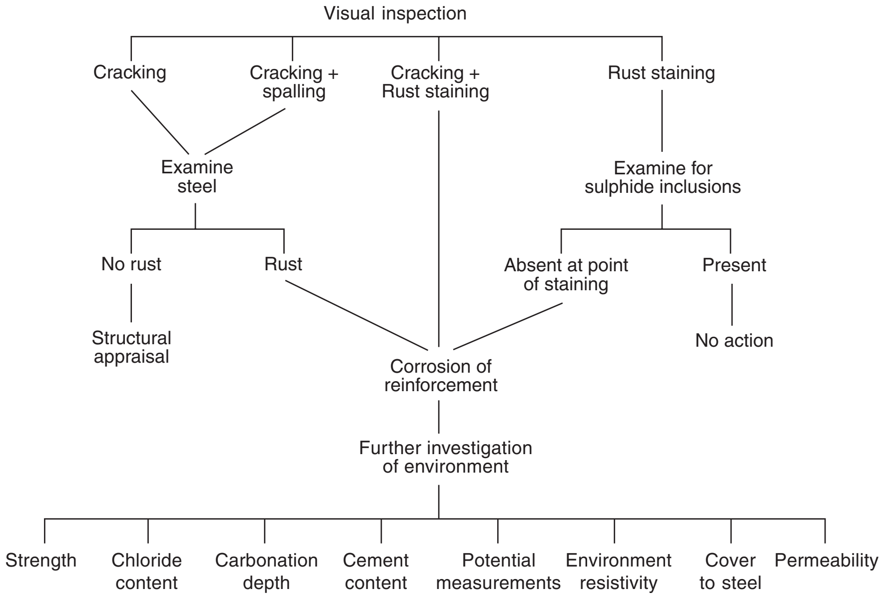

Figure 9.17 A flow diagram illustrating the visual assessment of cracks on reinforced concrete (BRE, 2000a).

Traditional non-destructive, *in-situ* techniques to monitor corrosion and its extent include delamination testing, corrosion potential mapping, corrosion rate measurement, and concrete resistivity measurement. A delamination survey may be conducted by tapping the structure with a hammer. Delaminated areas will then be associated with a hollow sound and may be recorded directly on the structure.

Steel corrosion potential may be measured using a portable reference electrode and a voltmeter. The absolute value of the measurement gives some indication of corrosion risk with more negative measurements indicating a higher corrosion risk (cf. Figure 9.16). For example, on bridge structures, rebar corrosion potential measurements determined relative to a silver/silver chloride portable reference electrode that are more positive than – 100 mV are considered to represent a negligible corrosion risk, while potentials more negative than – 250 mV represent a significant corrosion risk. Many factors may affect the measured values of the corrosion potential. As a result a more negative potential does not always indicate a higher corrosion risk. Another method of interpreting the data is to plot the data as a map and to use the map to identify the location of anodes and cathodes. An example of such a map is given in Figure 9.18. Local areas of negative potential are more likely to be the anodes of the corrosion cells which form in concrete. A limited amount of concrete breakout may be undertaken to calibrate the map.

Corrosion potential and rate measurements give an indication of the condition of the

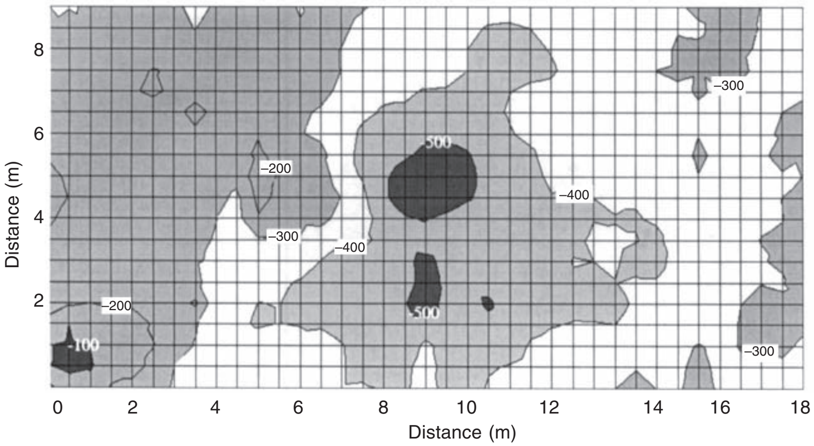

Figure 9.18 An example of a corrosion potential map obtained on steel in concrete. The potentials are given relative to the silver chloride electrode.

passive film on the steel (Gowers and Millard, 1999a). Corrosion rates are difficult to measure for a number of reasons. A small perturbation is applied to the steel and the response is measured in terms of an effect on the steel potential or applied current. As a result, any instability in the corrosion potential will result in measurement errors. The area of the steel affected by the measurement is another unknown variable that is required to quantify the measurements. The history of the local corrosion rate can give an indication of any local loss in steel section.

Resistivity measurements give an indication of the aggressive nature of the cover concrete (Gowers and Millard, 1999b). A small current is passed through the concrete and the potential generated between two probes is measured. Resistivity measurements greater than 20 kΩ cm are usually considered to represent a non-aggressive concrete cover, while measurements less than 10 kΩ cm would represent an aggressive concrete cover. The relationship between resistivity and corrosion risk is again complicated by the large number of factors that affect resistivity. For example, resistivity will vary significantly if the concrete is wetted and then allowed to dry out.

Destructive tests include removing samples for analysis to determine chloride profiles or take carbonation depth measurements (CEB, 1992). It is generally advised that non-destructive techniques should be calibrated with a limited amount of destructive assessment if possible (BRE 2000a).

To predict future performance, the rate at which the corrosion protection afforded by the cover is lost and the rate of corrosion-induced deterioration following passive film breakdown are required.

The rate of loss of protection provided by the cover is determined by the source of chloride and its rate of ingress. Approximate transport coefficients that characterize diffusion, migration, the movement of water and the capacity of the cement to immobilize chloride may be extracted from samples removed for laboratory analysis. However, factors such as temperature and degree of saturation, which are likely to be time dependent, will affect the transport processes occurring and should be monitored on-site (McCarter *et al.*, 2000).

Some innovation is necessary to extract transport coefficients using non-destructive techniques. Recent developments have been made in impedance and resistivity measurements (McCarter and Chrisp, 2000). These contain information on the moisture content, pore structure and degree of contamination of the concrete. Such techniques have already been used to study wetting and drying of concrete (McCarter *et al.*, 1998). Other methods of predicting deterioration have used galvanic corrosion probes (Raupach and Schiessl, 1997) and chloride ion sensors (Atkins *et al.*, 1996). However, the long-term stability of chloride ion sensors is unproven and galvanic corrosion probes should be installed prior to casting the concrete to accurately reflect the changing local environmental conditions.

Once corrosion initiates, the corrosion rate will determine the subsequent rate of deterioration. As noted above, average data are required as this varies with time. A popular method of obtaining the corrosion rate is to measure the polarization resistance (Figure 9.19).

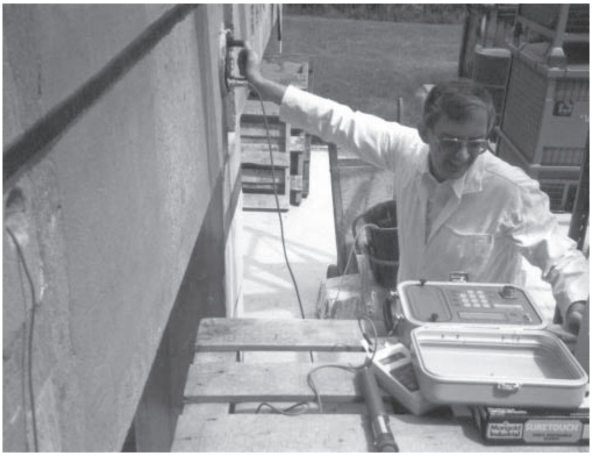

Figure 9.19 A device for measuring the corrosion rate of steel in concrete.

9.10 Repair of corrosion-damaged concrete

The repair of concrete structures is becoming increasingly important due to the long service lives required and the high cost of building and maintaining the infrastructure. There are many stages leading to a successful repair. These include:

- Assess the cause and extent of the damage
- Identify functional requirements of the repair and the repair options
- Design, specify and install the selected option
- Monitor the performance

9.10.1 Diagnostic approach

The choice of a successful repair system depends on an accurate identification of the cause of the problem. Thus, for example, if carbonation is the cause of corrosion-induced deterioration, an anti-carbonation coating may be considered when specifying the repairs.

As noted above, a number of inspection methods are available. Such an assessment is aimed at identifying whether repairs are needed, what repair options are available, and providing quantities for contract documentation. Thus, for example, one may need to determine whether a paint coating can be applied to the concrete surface and the surface preparation which would be most appropriate to obtain the durability required. In this case trials may be undertaken to provide the relevant data. Visual examination is the most common method used to determine the extent of the deterioration and provide a measure of the associated quantities. However, other methods such as a delamination survey or a potential survey may also be appropriate.

The rate of deterioration will also be useful in assessing the future serviceable life and maintenance requirements of the structure as well as the current need for and cost effectiveness of repairs. Methods which may be used include modelling the rate of penetration of aggressive species and the determination of corrosion rates.

9.10.2 Functional requirements

The functional requirement of the repair system will depend on the rate and extent of the deterioration together with the maintenance costs and design life of the structure. Repairs may be considered to be part of the maintenance required to achieve the design life. Factors which may affect its function include its compatibility with the existing structure and its durability in the service environment. As an example of compatibility, a coating system may not be applied to an existing friable concrete surface which may have resulted from some form of acid attack. Besides adhesion other factors which may affect compatibility include thermal expansion and shrinkage.

The durability of the repair system depends on both its own ability to resist deterioration in the environment to which it is exposed and on its ability to prevent further deterioration to the neighbouring or underlying concrete. Thus, for example, while conventional patch repairs may prevent further corrosion-induced deterioration in the repaired area, they may not be able to prevent further corrosion in adjacent concrete particularly if the cause of corrosion is chloride contamination because of the localized nature of the corrosion. Active corrosion sites provide a form of electrochemical protection (cathodic protection) for adjacent passive steel that may also be exposed to chloride contaminated concrete. The traditional patch repair of corroding areas therefore triggers corrosion initiation in these passive areas as the local protection provided is removed (Page and Sergi, 2000). The expansive forces resulting from this subsequent corrosion may cause the bond between the patch repair and the original concrete to break, or induce cracks within the repair material (Figure 9.20). Thus, when conventional patch repairs are used to inhibit chloride induced corrosion deterioration, all contaminated concrete should be removed if further deterioration is to be avoided (BRE, 2000b).

9.10.3 Repair options

A number of repair options are available for deteriorated structures. These include:

- Do nothing and monitor
- Patching

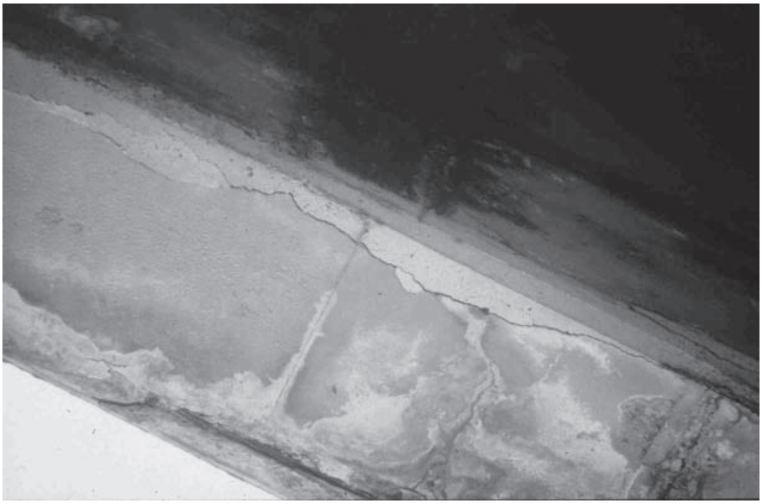

Figure 9.20 A failed patched repair in chloride-contaminated concrete.

- Concrete replacement
- Surface treatment of concrete
- Electrochemical protection

These may be divided into those in which active steps are taken to improve the environment within the concrete by removal of aggressive species, those in which the aim is to exclude any further contamination from the environment and those in which the effects of the deterioration are minimized. Various repair principles and examples of methods based on the principle are given in Table 9.2 (ENV 1504-9: 1997).

Methods which improve the environment include concrete replacement together with the electrochemical methods of cathodic protection, chloride removal and re-alkalization. Such methods may be appropriate when the aggressive species are contained within the concrete and any further deterioration must be minimized. Figure 9.21 illustrates the environment changes which may occur when an electric current flows through an electrolyte. The change in the environment at the steel interface promotes steel passivity.

Exclusion of the environment is primarily achieved through surface treatment of the concrete. The aim in this case is to prevent further ingress of aggressive species which may cause future deterioration. However, some coatings also allow the concrete to dry out while greatly reducing further moisture ingress. Thus this may also be viewed as improving the internal environment.

In some cases the aim may be to do the minimum necessary to maintain the function of the structure until the end of its useful life determined by other factors. Methods which can then be considered include simply monitoring the structure and installing holding repairs.

9.10.4 Technical requirements

The design, specification, drawings and bills of quantities contain the technical requirements of the repair system and form the principle means of communication between the consultants,

Table 9.2 Repair principles for reinforcement corrosion induced deterioration (ENV 1504-9)

Principle and its definition	Some examples of methods based on the principle
Restoring passivity Creating conditions in which the reinforcement is protected by a film of ferric oxide	**Replacing contaminated or carbonated concrete** Physically removing contaminated or carbonated concrete and replacing it with uncontaminated, uncarbonated concrete **Restoring alkalinity to carbonated concrete** Re-alkalizing carbonated concrete *in-situ* ☠ May cause hydrogen embrittlement and force discontinuous steel to corrode **Chloride extraction** Removing chloride contamination from concrete *in-situ* ☠ May cause hydrogen embrittlement, bond strength reduction, alkali aggregate reaction and force discontinuous steel to corrode **Cathodic protection** Moving the potential of the steel into the passive region ☠ May force discontinuous steel to corrode
Increasing resistivity Increasing the electrical resistivity of the concrete	**Limiting moisture content with surface treatments, coatings or by sheltering** ☠ Reducing moisture content may increase the rate of carbonation
Cathodic control Creating conditions in which potentially cathodic areas of reinforcement are unable to take part in the corrosion reaction	**Limiting oxygen content (at the cathode) by coating the concrete** **Applying cathodic protection to remove oxygen at the steel surface** ☠ All electrically connected potentially cathodic areas must be included in the saturated area

☠ denotes a warning

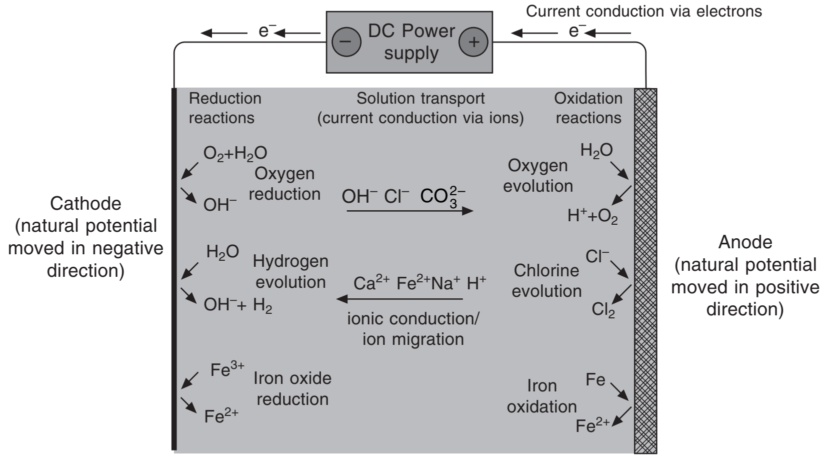

Figure 9.21 Electrochemical processes occurring when a current flows from an anode to a cathode through an electrolyte.

owners and installers. The specification may be in the form of a simple method statement, or be related to the performance required. In the latter case a specialist contractor may be required to interpret the specification in terms of a method statement which may be submitted during the tender process.

The design contains the basis for all the decisions and choices made concerning a particular repair system. This includes any calculations that may be required. The aim of the specification and drawings is to translate the design requirements into an understandable document. Within the specification, a bill of quantities is usually provided to give the units by which the work may be measured.

9.11 Summary

The corrosion of steel reinforcement in concrete structures is a major problem. Deterioration starts with the loss of protection provided by the concrete cover. Physical damage does not occur during this stage. The high pH of the concrete environment maintains a stable passive oxide film on the reinforcing steel. This is followed by corrosion initiation and then propagation.

The two most important causes of corrosion initiation are carbonation and chloride contamination. Contamination of the concrete cover can occur via diffusion, migration and water flow. It is affected by the pore structure, interactions with the pore walls and interactions with other ions in the solution.

Once passive film breakdown has occurred, the corrosion rate is controlled by either the kinetics of the anodic or cathodic reactions, or by the resistance to current flow between the anodic and cathodic sites through the concrete. Reinforced concrete is exposed to a wide variety of environments and any one of these three mechanisms may dominate depending on the exposure conditions.

Monitoring may be used to assess existing deterioration and predict the future performance. Corrosion potential and resistivity measurements may provide information on the state of the reinforcing steel and the condition of the cover concrete.

The repair of corrosion damage has always presented problems particularly in the case of chloride induced corrosion. This results from the localized nature of the corrosion which provides a form of electrochemical protection to the steel in the adjacent chloride contaminated concrete.

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10

Concrete and fire exposure

Bob Cather

Concrete is generally considered to perform well when exposed to fire since it is inorganic, non-combustible and does not give off noxious fumes. Moreover, as well as being non-combustible, concrete is a poor conductor of heat and has a relatively high specific heat capacity, which facilitates its use as a protection material to other elements and even as a heat storage medium. There are, however, several issues that make the real behaviour of concretes in fire conditions less straightforward and these are related to compositional and phase changes within concrete constituents and the behaviour of absorbed moisture.

Although this chapter is targeted at concrete as a material, it is difficult to discuss this meaningfully without considering the manner of its incorporation in structures and the temperature response of steel, either because of the use of concrete with embedded steel reinforcement or because concrete is used to provide fire protection for steel structures.

This chapter considers the performance of ‘structural concrete’ in typical buildings or other structures subject to various fire excursions. Applications such as concrete for extreme heat resisting elements in foundries or refractory facilities are outside the scope.

10.1 Essentials of concrete behaviour

There are two principal effects of fires on structural concrete:

- Loss in strength of matrix by degradation of hydrate structure. This occurs at various stages from 300°C upwards but the main losses are seen at 500°C plus.

- Spalling and ‘shelling’ of the outermost concrete. This can occur with most concretes but the extent and rate is influenced by aggregate type, moisture content, concrete quality, fire severity and imposed stress condition.

The overall behaviour of concrete in a fire is the result of the complex interaction of the mechanisms of strength loss and spalling. Although there has been considerable research on concrete in fires and individual mechanisms identified and understood, the complexity of the interactions makes precise prediction of behaviour of concrete in structures extremely difficult.

10.2 Strength loss in the cement matrix

Strength loss in the concrete matrix has been researched but some divergence in the detailed conclusions are found in references. While individual mechanisms can be identified in specific matrix components, e.g. the breakdown of hydrates in any one type of cement, the variety of processes creating change in materials properties and their interactions result in different specific values and consequences from the different researches. The broad temperature ranges over which the changes occur and their effects on concrete properties, discussed below, should, however, be sufficient to estimate overall performance and evaluate damage.

On initial heating concrete will first lose absorbed, free or ‘evaporable’ water then bound or adsorbed water. This loss of water may induce microcracking and some consequent loss in compressive strength, possibly up to 10%. From 150°C upwards some degradation loss of water from silicate hydrates and from Portlandite (calcium hydroxide) can occur but above 300°C the loss of bound water from the hydration products becomes more prominent and further strength loss will occur. With increasing temperature the strength loss continues in the silicate hydrates and, at 350–400°C, in the calcium hydroxide by dehydration to form calcium oxide. It is also suggested that the formation of calcium oxide can result in post-fire damage should the calcium oxide react with water, such as from fire extinguishing efforts, causing swelling and cracking. By approximately 500°C a considerable loss in strength has occurred – variously recorded as 50 to 75% of original strength – and temperatures in the range 550–600°C have variously been taken as the upper limit for retention of any useful strength in the concrete. However, degradation processes and losses continue to take place up to 850–900°C. The strength loss does not appear to be uniquely defined and research outputs vary in the extent of loss recorded, but reductions of the order of 70–80% are quoted where the concrete becomes loose and friable (Smith, 1994; Neville, 1995).

A similar scale of change is found in concrete compressive modulus, over the same temperature ranges described above in relation to strength.

The type of cement is thought to have some influence on strength loss. For cements with fly ash or ground granulated blast furnace slag there is some suggestion that the lower quantities of free calcium hydroxide in the hydrated microstructure give reduced losses on heating. However, for most Portland type cements, these differences are sufficiently small as to not affect the practical performance of the concrete and therefore cement types are not explicitly selected for fire resistance.

There are two exceptions to this general conclusion on cement type. The first is concretes with microsilica where the very low permeability paste produced is thought to significantly increases spalling risk – see below on high strength concrete. The second exception is concrete made with calcium aluminate cements. These have greater resistance to strength loss at high temperatures and are used for specialist non-structural aplications such as refractory linings or industrial floor toppings in foundries. Such applications are outside the scope of this chapter.

10.3 Spalling

Spalling of concrete in fires is the breaking-off of layers of the concrete surface in response to the applied heat. Spalling can be either localized or widespread depending upon the fire and/or concrete condition, particularly moisture content, and the susceptibility to break-up of heat-unstable aggregate particles. On prolonged heating areas of concrete cover can also just fall away, a process that is sometimes called 'sloughing'. The processes causing sloughing are not generally reported, although it is noted that it occurs from corners of beams and slabs and seems to spread along a plane of weakness parallel to the outer surface. Because 'sloughing' occurs late in a fire exposure it is considered by some as being of less concern than explosive spalling that occurs earlier upon exposure to fire. Understanding explosive spalling is important because of the potential for loss in section of the concrete element, the depth of fire affected concrete and the reduced protection to embedded steel.

Spalling is a frequently observed phenomenon in fire; more prominently on soffits of slabs and on beams because of the greater exposure to heat and possibly heat 'entrapment'. It is not certain that this frequent observation is fully anticipated by design codes and this is discussed in more detail below, in the section on design codes.

The prediction of risk of spalling occurrence has not proven easy despite considerable research. The propensity to spall is influenced by the moisture content of the concrete, the permeability of the concrete, the rate of heating, the nature of the aggregate and the load applied to the concrete. Although these separate contributing mechanisms have been identified, their relative contribution and their interaction is less well understood. There are, however, general trends that can be established. Concretes in a moist or saturated condition will spall faster and more extensively the drier the concrete. Some guidance (Malhotra, 1984; Concrete Society, 1998) suggests that moisture contents greater than 3% by weight will lead to spalling, although this 'limiting' value may be affected by the permeability of the concrete; the lower the permeability the higher the risk of spalling, a higher temperature also increases the risk. Some research also shows that elements under higher structural load during the fire have increased risk of spalling. These issues are discussed in greater detail in following sections.

There are mixed views and experiences expressed in the literature on the influence of reinforcement bars and concrete cover depth on spalling. These are briefly discussed later in the section on design codes. In general the presence of normal structural bars does not seem to influence spalling of concrete until after the cover has spalled off, although the greater the depth of cover (it is believed), the greater the risk of spalling. There is some evidence that using a smaller non-structural mesh in the cover zone reduces the ability of spalled concrete to fall away.

10.4 The influence of aggregate type

The contribution of aggregate type to spalling and section loss is both from the nature of the aggregate itself and the differences in temperature-related properties between aggregates and the surrounding matrix. It is commonly found that siliceous aggregates such as flint gravels give the poorest resistance to spalling. This is explained by being partly the result of markedly different coefficients of thermal expansion between the aggregate and cement paste, particularly at higher temperatures, and partly the result of a volume increase phase transformation (at approximately 570°C) from α-quartz to β-quartz.

Limestone aggregates have generally been shown to give good fire resisting performance but not all design codes have found the evidence consistent enough to give design guidance differentiating that performance. There are several reasons why limestone type aggregates can be expected to give improved resistance to degradation. First, the aggregates typically have lower coefficient of thermal expansion than siliceous aggregates and they are closer to that of cement paste, giving lower internal stresses on heating. There are also no solid-state phase changes in limestone aggregates within fire exposure conditions. On heating to temperatures in excess of 660°C calcium carbonates begin to break down, similarly above 740°C for magnesium carbonates. On breaking down the minerals release carbon dioxide, in itself an endothermic reaction, but the released carbon dioxide is claimed to give blanketing protection against heat transfer. The residual aggregate particles also have lower thermal conductivity, further reducing heat transfer into the concrete.

Synthetic, sintered, lightweight aggregates can demonstrate very good performance in ‘dry’ building fires (Concrete Society, 1995; BSI, 1985). The performance derives from the coefficient of thermal expansion compatibility with cement paste, the inherently high temperature stability of the aggregate and the good cement aggregate bond giving high strain to cracking failure. These lightweight aggregates have, however, shown poor performance in laboratory tests if they are kept saturated up to the test. It is not completely clear why this is, but is likely to be due to a combination of a very high retained water content and the good-quality, low-permeability, cement matrix. This can lead to high internal pressure build-up due to steam generation, which is unable to dissipate sufficiently quickly due to the low permeability of the matrix.

Evidence for other types of lightweight aggregate, particularly from natural sources, is rather dispersed and insufficiently conclusive to give general guidance. Specific research and testing would be needed to assess performance.

10.5 High-strength concrete

High-strength concrete is increasingly used for building structures worldwide. Although there is no fully agreed definition of when ‘high strength’ begins, many people take this to mean concrete of higher than grade C80 cube strength. The response of high-strength concrete to fire exposure appears to differ from that of lower strength in several ways.

Some research (Neville, 1995) indicates that higher strength concrete in fires tends to show greater strength loss earlier than other concretes. Although not fully researched this may be, as shown previously, due to the higher strength concretes having a higher cement matrix proportion that is affected to a greater degree at lower temperatures than the aggregate fraction.

Of greater concern is that high-strength concrete appears more prone to explosive spalling than ‘normal’ concrete with a more rapid loss of section (Smith, 1994; Concrete Society, 1998). The precise mechanisms for this behaviour are not fully reported. The understanding of concrete behaviour is less well reported than for normal concrete in building fires but it is thought probable that there is greater difficulty in vaporized moisture escaping from the fine pore structure concrete. Leading researchers claim that it takes a very long time for high-strength concrete to become dry enough, even inside a completed building, for spalling not to coccur and in practice it may never achieve this conditin. It has been suggested also that moisture released within the concrete by the breakdown of hydration products by heating may be sufficient to cause spalling.

10.6 Essentials of steel behaviour

All steels will lose strength with increasing temperature. Figure 10.1 shows the relationship between strength and temperature for normal structural steel, and a typical concrete (with no allowance for spalling). Typically, the static load on a steel element in a structure is designed to be approximately 55% of the yield strength. The strength of the steel at 550°C is about 55% of the room temperature yield strength. Therefore, structures that are typically loaded will just remain stable at 550°C. This is known as the critical temperature. If the static load is lower, the critical temperature will be higher and the structure will be stable at higher temperatures than 550°C. These temperatures are much lower than the melting point of steel (~1450°C) and thus slowing or preventing the dangerous extent of weakening of steel, rather than prevention of melting, is the main concern with concrete acting as fire protection. Note: These temperatures and parameters relate to ‘normal structural steel’. Other special steels may have different temperature sensitivities or may be loaded to a higher level and thus require different parameters.

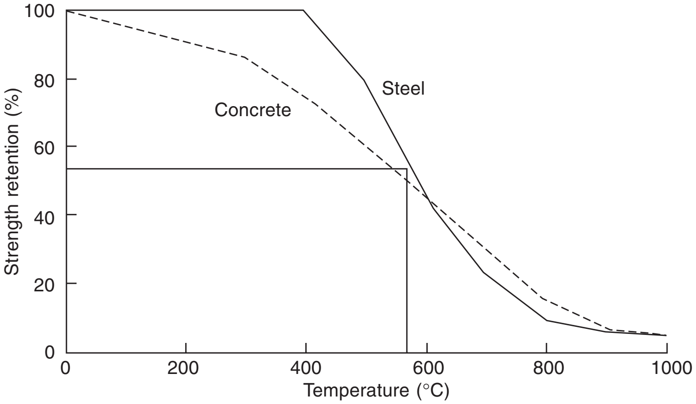

Figure 10.1 Strength retention with temperature for concrete and steel.

10.7 Fire behaviour and design codes

The performance of concrete in a fire and the design of structures to provide adequate resistance must take into account much more than the behaviour of concrete as a material. The approach should be to consider the likely extent of degradation, protection of reinforcement from high temperatures and provision of minimum section sizes to prevent fire spread through separating elements such as walls or floor slabs. The basis of current conventional structural design (Bailey, 2002) is to consider elements of a structure such as beams, columns, slabs etc. and their connectivity. This approach has proven reasonably successful in that it is rare for concrete structures to suffer collapse during a fire. However, it is increasingly seen as conservative and incomplete with resulting extra cost and experiencing greater damage than might otherwise be the case, because of insufficient accommodation of temperature movement between combinations of structural elements. A detailed discussion of the structural design bases for fire resistance are outside the scope of this chapter, but some explanation will aid understanding of the role of concrete materials and concretes in contributing to adequate fire resistance.

The primary degradation effects that fire resistance design considers are loss in strength of the matrix, heat transfer and spalling – both in the context of loss of concrete cross-section and the exposure of reinforcement to the fire. Matrix strength loss and heat transfer are compensated by the provision of minimum concrete section sizes dependent upon the degree of fire resistance required, which may commonly be $^1/_2$ hour, 1, 2, 3 or 4 hours, depending upon the type and use of that part of the structure. The cover of concrete to be provided is related to the effects of heat on reinforcement and assumptions regarding spalling. The main Code of Practice in the UK for design for fire resistance is BS 8110 (BSI, 1985). In BS 8110 Part 1, there are ‘simple’ prescriptive rules for both section size and cover to reinforcement that are suitable for the majority of design situations. Part 2 of BS 8110 (BSI, 1985) has more comprehensive design bases for structures considered special cases where there is potential benefit in using a more detailed approach than that contained in Part 1. BS 8110 Part 2 (BSI, 1985) describes three methods, namely from tabulated data, direct fire tests on elements of strucure and fire engineering calculations. In a fire engineering approach – applicable in BS 8110 only to elements subject to flexure, e.g. beams and slabs – calculation of structural behaviour is made based upon first principles using the assumption that failure in a fire is governed by yielding of main tensile reinforcement. Other UK codes use a similar basis to BS 8110 and it is likely that the Eurocodes will be similar in principle but may differ in detail. The Eurocode (CEN, 2002) also has developed a middle route between tabulated data and a full Fire Engineering approach. In this method a series of isotherms (at 500°C) have been developed for various concrete section sizes under differing fire exposure periods. From these the analysis of the residual structural capacity can be performed under various scenarios.

The prescriptive rules in Part 1 of BS 8110 make no allowance for different aggregate or concrete types and they assume that no spalling will occur if the cover to reinforcement is 40 mm or less. The more comprehensive methods for design for fire resistance in Part 2 of BS 8110 recognize differences in performance between different concrete types in respect of different coarse aggregate types. However, while recognizing that in practice concretes made from limestone aggregates are less susceptible to spalling than concretes containing a higher proportion of silica, such as flint or granite, guidance is not given on taking advantage of this benefit due to the lack of consistent data. The tabulated design method in BS 8110: Part 2 does, however, acknowledge the improved performance of lightweight aggregates that are assumed to be in a non-saturated condition (see influences of aggregates above).

For covers in excess of 40 mm, where spalling is considered likely by design codes, supplementary protection is required. In earlier UK structural codes, one recommendation for this protection was to use a light steel mesh in the outer concrete cover zone. It is now recognized by codes that such a method is very difficult to achieve with satisfactory levels of quality and the approach is rarely used. Other supplementary methods described in BS 8110 include the application of plaster or lightweight fire sprays or cladding.

10.8 Fire types and heat exposure

Buildings are normally designed in the context of resistance to a ‘cellulosic fire’ which essentially is the combustion of building finishings and contents that typically comprise largely cellulose-based materials. To simulate the effects of fire on structural elements of a building, standard fire tests have been developed and adopted. These fire tests expose an element or an assembly of elements to a standard time–temperature curve in a furnace. Various standard tests can be used e.g. BS 476 Part 20 (BSI, 1987) and ISO 834 that have slightly different characteristics but the overall nature of the test is similar and the test conditions are therefore, generally considered to be the same. These tests typically expose elements or assemblies to temperatures of approximately 900°C after 45 minutes and 1050°C after 2 hours. Typically, buildings will be required to show resistance in the standard test for 60 minutes although there are specific situations where shorter or longer periods are required.

A hydrocarbon fire arises from the combustion of hydrocarbon fuels – liquids or gases – and compared with a cellulosic fire creates more extreme conditions in several ways. The principal differences are the rate of temperature rise and peak temperature. For both the BS and ISO hydrocarbon curves a temperature of 900°C is reached after only 4 minutes and the peak temperature of 1100°C after approximately 20 minutes.

These fire tests create standardized conditions, but specific situations may create real fires more or less onerous than standard test conditions. When a hydrocarbon fire occurs in a closely confined space, such as in a tunnel, it can be more intense than the standard hydrocharbon fire test. Standardized time–temperature curves have been developed to allow for this, an example being the Dutch RWS (Rijswaterstaat) specification that peaks at 1350°C after 1 hour. Similarly, the actual severity of exposure of a concrete element to heat and thus the consequent damage in a building will be influenced by the location of the fire relative to the element and the degree of enclosure of the fire and consequent heat build-up characteristics. For example, the soffitt of slabs and beams directly above the fire might be expected to suffer greater damage than the top of the slab upon which the fire is located because of the greater upward convection of heat.

To complement the hydrocarbon fire tests, particularly for the petrochemical industry, jet fire testing has been developed. This reproduces the effect of burning pressurized oil or gas released at constant rate from, for example, a ruptured pipe. As well as a higher heat flux, the sample will be subjected to the erosive effects of the jet stream.

Typical standard time–temperature curves for cellulosic and hydrocarbon fires are shown in Figure 10.2.

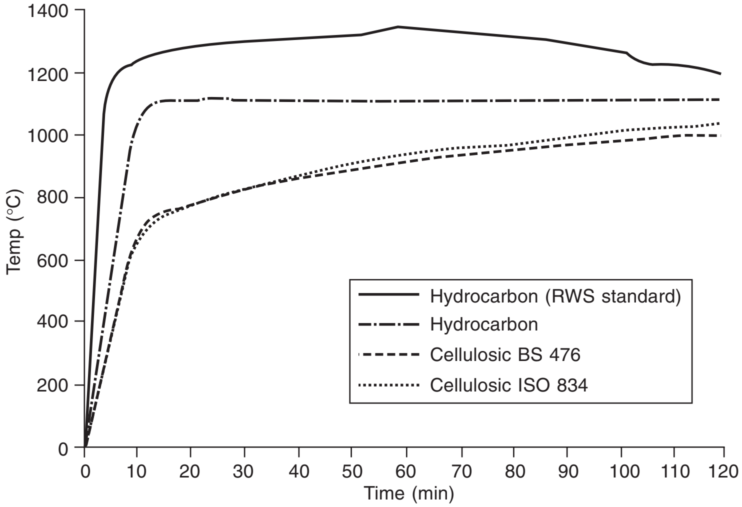

Figure 10.2 Comparison of typical 'cellulosic' and 'hydrocarbon' time/temperature curves.

Other non-furnace tests, e.g. of the wood crib type, have been developed and adopted by specific bodies because they are believed to better simulate real fires on complete construction assemblies. Although such tests are useful for comparative purposes, they are not directly recognized by UK Standards and Building Regulations and they are not discussed in detail in this chapter. Similarly, other fire-performance-related standard tests such as surface spread of flame and combustion product toxicity are not considered here because they are not relevant to structural concrete.

10.9 Behaviour of concrete in extreme fires

Concrete does not 'melt' in the majority of extreme fire conditions but it could do so in conditions such as created by, for example, a thermic lance (steel burning in a pure oxygen environment). However, this is exceptional and is not normally considered in the design of reinforced concrete subject to hydrocarbon fires.

The reported experience for this combination of materials and hydrocarbon fire is limited and there is little published informaton or guidance on this aspect compared to laboratory fire tests.

One of the most helpful early reports was that on the consequences of a tanker explosion and fire while moored to a concrete jetty in Bantry Bay (Maling, 1987), Ireland. The limited reports showed a very rapid loss of cover concrete (50–75 mm) but that spalling beneath the reinforcement was limited. More recently the records of the condition of the Channel Tunnel concrete linings after a vehicle fire (NCE, 1996) appear to show localized complete penetration of the concrete lining. It is possible that the confined tunnel environment created more aggressive temperature conditions than those for the 'open' jetty at Bantry Bay.

Currently, although there are data available from both real extreme fires and tests, it is fragmented and is somewhat contradictory. However, it is possible to derive some general views and mechanisms that illustrate the likely performance under such conditions.

Rapid heat rise in concrete causes evaporation of free and physically bound water and, at higher temperatures, moisture loss by dehydration of cement hydrates. If the permeability of the concrete is insufficient to allow an adequate rate of dissipation then the vapour pressure in the pores of the concrete will rise. A contribution to the low apparent permeability-resisting vapour dissipation, is the vapour condensation further inside the concrete away from the fire. Once the vapour pressure rises to a critical level cracking and explosive spalling will occur.

This explosive spalling can occur after only a few minutes and rates are quoted in some reports of up to 3 mm/min for normal-weight aggregate concrete and up to 8 mm/min for lightweight aggregate concrete. More information is needed on these rates and the contribution from the various parameters.

Other types of spalling such as local spalling and sloughing-off (gradual reduction of a cross-section) that have been observed in cellulosic fires are possible but explosive spalling seems to be the dominant form in an extreme hydrocarbon fire.

As was described earlier, concrete subjected to high temperature will suffer loss of strength. The strength loss increases as the temperature increases and both the aggregate and the cement hydrates are affected. At the very high peak temperatures in a hydrocarbon fire the aggregate and cement hydrates may be completely destroyed.

Many factors have an influence on the performance of concrete in hydrocarbon fire but those with a primary influence are:

- the rate of temperature rise in the concrete
- the moisture content of the concrete
- the permeability of the concrete

These factors are interlinked but there is some evidence to suggest that, at least for low-permeability concrete, sufficient vapour pressure for damage to occur can be generated by decomposition of cement hydration products alone, even where there is little or no free water within the concrete pores. This would mean that indoor concrete would never dry sufficiently for spalling not to be a problem and, that self-desiccation in concrete with a very low water/cement ratio would not alleviate the problem.

The definition of satisfactory performance will be dependent on individual circumstances. Nevertheless, it is likely that the very high rate of temperature rise in a hydrocarbon fire will cause explosive spalling and loss of section at a high rate, particularly in high-strength concrete and lightweight aggregate concrete. Reinforcement could thus be exposed to high temperatures in less than approximately 20 minutes depending on depth of cover and other factors.

10.10 Improving the fire resistance of concrete

For many fire incidents concrete can be considered to be sufficiently resistant to damage by fire that, on its own, it needs no further protection. There may, however, be situations where more severe design fires are envisaged, or the existing concrete construction is considered inadequate for new conditions or where concern about the inherent performance exist, e.g. for high-strength concrete. For these conditions enhanced fire protection or resistance may be required.

Where a simple increase in the thickness of concrete cover would satisfy the design requirements, the design Codes allow the use of equivalent thicknesses of other passive materials such as sand cement render or gypsum plaster. Alternatively, fire protection boards can be mechanically fixed around the concrete sections. Steel structures are frequently protected against fire by the application of relatively thin intumescent coatings, which foam upon exposure to produce a protective char. Although attempts have been made to use these materials on concrete, they have not been very successful to date. One particular problem is that, even with the protection in place, the temperature in the concrete is sufficient to vaporize moisture which then disrupts the coating.

One method for improving the fire performance of high strength concrete in cellulosic fires and all types of concrete in more extreme fires, is the incorporation of fine polypropylene fibres within the concrete (HSE, 2001). At present this route is only adopted for new concretes where the fibres can be incorporated into the mix but it may be possible to develop overlays for some existing structures. The action of polypropylene fibres was initially believed to be by fibre melting and vaporization upon heating providing escape pathways for moisture. It was known from tests that such fibres could be effective but not all fibres were found to be equally effective and there was some uncertainty about the influence of fibre geometry and quantity. Research in France (Kalifa *et al*., 2001) has provided significantly better insight into the mechanism of beneficial action by the fibres.

The research measured the pressure build-up in the porous network within the concrete for additions of different polypropylene fibres close to the ‘conventional’ level of approximately 2 kg/m^3. The data obtained showed that lower dosages than previously thought can be beneficial and that the fibres are already contributing to preventative action at temperatures well below that necessary for vaporization. It was concluded that the polypropylene on melting – at approximately 170°C – was absorbed into the cement paste of the matrix, creating the necessary pathways. Further research on this aspect would be helpful in increasing understanding.

10.11 Evaluation of concrete structures exposed to fire

On first inspection a structure or building that has experienced a fire event can appear to have suffered extensive damage such that it may have to be demolished and replaced. The widespread deposition of smoke and soot, possibly extensive flooding from fire fighting, partly combusted residues from the fire source and remains of other building contents can combine to make a grim picture. Often the real damage to the concrete structure may be much less severe than at first it seems and thus investigation and evaluation need to be carried out carefully and methodically. Investigation of fire damage to concrete is covered comprehensively in specialist references (Concrete Society, 1995; Smith, 1994). The following is aimed at presenting the essential aspects in sufficient detail to give a broad understanding of the issues and approach. It is useful to remember that real fires are likely to be quite different to the standard ‘Design’ fire conditions used in structural design and each fire event should be treated as having individual fire characteristics.

First, it is useful to try to build up a picture of the nature of fire and thus the likely nature and extent of any damage. An assessment of the materials burnt and the disposition of the fire can give information about the likely temperatures developed and the duration at any location. Fire debris can also give useful guidance as to temperatures experienced by evaluating which types of materials, e.g. plastics, glass, aluminium, or timber, have deformed, melted or burnt. These observations are rarely enough to evaluate the extent of damage directly but are a useful guide in planning more specific examination and testing. Consideration of the fire characteristics may also prompt other specific issues, such as whether toxic or deleterious combustion products have been given off. The burning of extensive quantities of PVC, for example, may give off enough hydrogen chloride to initiate corrosion of steel elements or reinforcement.

The three principal concerns in evaluating the effect of the fire on a concrete structure are:

- depth of damage (spalling) or loss in strength of the concrete matrix
- loss in strength of steel reinforcement or embedded structural steel elements
- damage or distress to the structure from movement, settlement or imposed loads

Outwardly, damage to the concrete will be seen most obviously as spalling. This will vary depending upon location in the structure and the severity of the fire. Typically, soffits and thin ribbed slabs show more damage than tops of slabs and lower portions of columns. The absence of spalling, however, does not necessarily signify that no damage or loss in strength has occurred. As discussed earlier, up to temperatures of approximately 300°C the strength loss for normal concretes is modest and is usually taken as insignificant in evaluating future performance. Above 300°C the loss is significant and it is most simply assumed that the concrete has insufficient strength.

A useful guide to temperature and strength loss is to use the temperature-induced colour change in the matrix of concretes made with siliceous aggregates. At around 300°C these concretes start to show a pink colour derived from iron compounds in the concrete. The colour is not always consistent with all concrete but, where visible, is a very useful guide to temperature exposure. With increasing temperature the pink colour will be degraded as other strength-reducing changes in the matrix take place, typically at around 600°C and the concrete may once again appear grey. Thus, the examination of a small diameter core, after splitting may reveal an outer grey, weak or friable zone, beneath which the pink layer can be seen and beneath that, normal concrete. A simple level of assessment of the extent of damage is by removal of concrete to the inner side of the pink zone. The siliceous aggregates needed to give the pink colour can be either the coarse or fine fractions and thus even concretes made with limestone or manufactured lightweight aggregates but using a siliceous sand, can show the pink coloration.

Other methods of evaluating the depth of damage to concrete have been used either separately or in combination with colour change. Simply taking cores and crushing for strength has not been very useful in assessment because of the difficulty in assessing the gradation of property change with depth. Non-destructive evaluation with a rebound hammer has been used on a comparative basis to make an approximate estimate of areas of potential damage but the information is of limited value as residual strength, or the depth of influence, cannot be determined. More successfully ultrasonic pulse velocity measurements in direct mode through complete elements has been used to evaluate depth of damage. A more specialist but very specific test method uses changes in the thermoluminescence of aggregate particles drilled from small holes in the concrete (Placido, 1980). The magnitude of the effect reduces significantly at similar temperatures to those creating strength loss in the concrete. The ability of this method to allow for the duration of heat exposure has, however, been questioned by some researchers. Because of its specialist nature the method has not become common for the evaluation of strength compared to colour change.

Evaluation of the effects of the fire on steel reinforcement is important for overall structural assessment. The effects of a fire event on the steel will be determined by the nature of the steel, e.g. hot rolled, cold rolled, cold drawn prestressing strand or other more specialist types, and the degree of exposure to heat related to depth of cover and extent of spalling.

The steel will exhibit loss in strength while at elevated temperatures but may recover some strength on cooling. The strength recovery for cold-formed steel bars may be substantial if the peak temperature exposure is less than 450°C. For hot rolled steel the majority of the strength will be recovered if exposure is limited to 600°C. Prestressing strand is likely to show more strength loss at lower temperatures than bar reinforcement and typically experiences a 50% loss at 400°C. Special steel bar reinforcement products may exhibit a different behaviour to typical steel and specific manufacturer guidance should be sought or mechanical tests performed on bars taken from the damaged structure. For normal reinforcement, making assessment from generalized guidance may be suffcient but if doubt exists or special requirements need to be assessed then direct mechanical testing may be appropriate.

Damage or distress to the structure from movement, settlement or imposed loads can only be assessed by suitably qualified and experienced engineers. Issues of deflection, cracking, restraint to temperature-induced movement, can all be relevant. For prestressed concrete, the loss in modulus of the concrete and/or the relaxation in the prestressing strand may require evaluation of residual prestress capacity. Detailed discussion of this is outside the scope of this book.

The approach to the repair of fire damaged concrete will depend upon the nature and extent of damage and the conclusions drawn from the structural evaluation. The repairs might be limited to cutting out affected concrete and replacing with hand-applied polymer modified mortars, essentially to reinstate the cover to reinforcement. Some of these proprietary products have been tested for fire resistance and appear to be accepted by authorities as consistent with the protection provided by normal concrete. More extensive repairs might be undertaken with sprayed concrete or, for deeper repairs, by shuttering and re-casting with concrete. In extreme cases whole element replacement might be the most effective and economic solution. Any steel reinforcement found to have suffered unacceptable strength loss might be replaced by lapping new bars in or perhaps consideration could be given to externally bonded or fixed reinforcement including resin fibre composites. The fire performance of these approaches will be structure specific and is thus outside the scope of the discussion here. Fire damaged structures can, in principle, be reinstated to fulfil normal service life expectations.

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11

Freeze/thaw resistance

Michel Pigeon, Bruno Zuber and Jacques Marchand

11.1 Introduction

The basic mechanisms of frost damage are now fairly well known. The formation of ice crystals can create pressure on the pore walls, and also a flow of water out of the pores through the paste to the nearest air void. In addition, unfrozen water is attracted to the ice crystals during periods of freezing. This causes the slow growth of the crystals (ice accretion), which in turn increases the internal stresses developed. The main parameters that influence these mechanisms are also fairly well known. They are the characteristics of the pore system, i.e. total porosity and pore size distribution.

The use of de-icer salts can increase the surface damage due to frost, a phenomenon known as surface scaling. Unfortunately, although it has been extensively studied, this phenomenon is still not clearly understood (Marchand *et al*., 1997). It is, however, very clear that the microstructure of the surface layer can be more porous, and that saturation and salt penetration are easier close to the surface.

In this chapter, the basic mechanisms of frost action will first be described, and then typical test results will be presented to illustrate the influence of the various parameters. The particular problem of surface scaling due to freezing in the presence of de-icer salts will be the subject of a separate section, and another section will be devoted to the question of air entrainment, emphasizing the importance of the air void spacing factor, which is not always closely related to the air content. Laboratory tests and, more important, field performance have shown conclusively that the solution to obtain good-frost and de-icer salt scaling resistance is the use of properly air-entrained good-quality concrete. At the end of the chapter, the frost resistance of special concretes, such as high-performance and roller-compacted concrete, will be briefly discussed, and the information obtained through the controlled observation of field performance will be summarized. This last section will emphasize the importance of access to water. The basic reason for the severity of most laboratory tests is most probably access to water at all times, which is not always the situation in the field.

11.2 Mechanisms of ice formation in cementitious materials

At atmospheric pressure, the formation of ice, i.e. the solidification of liquid water, is possible when the ambient temperature is reduced to below 0°C. Experimental results have shown that ice crystals formed in microporous bodies (such as the hydrated cement paste) have essentially the same physical characteristics as ordinary ice. In both cases, the density and properties of the crystals are those of type I_h ice. Ice crystals present several allotropic forms depending on temperature and pressure history. Type I_h ice is the form commonly encountered in the field (having an hexagonal lattice). Furthermore, as for 'free' water, the solidification process in a porous solid comes with a 9 per cent volumetric expansion as well as a release of heat (latent heat of solidification). Both phenomena are related to the change in the molecular structure of water.

Ice formation in cementitious materials can be investigated with several techniques among which low temperature calorimetry is probably the easiest to understand. A small sample of material is placed in an adiabatic container. As the temperature of the cell is slowly reduced, the heat flow is measured.

Figure 11.1 shows typical low-temperature calorimetry test results. These experiments were performed in the 0 to – 60°C range, at atmospheric pressure. As ice formation is an exothermic process, the deviation of the apparent heat capacity curve from the baseline (dashed line) corresponds to an energy release, which can be related to the solidification of a given mass of ice.

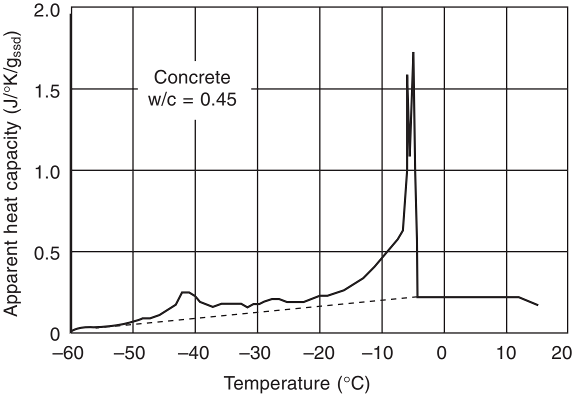

Figure 11.1 Typical low-temperature calorimetry data (subscript ssd stands for saturated surface dry). A heat release (surface between the plain and the dotted curves) during a given temperature decrease is related to the amount of ice formed between these temperatures. Ice formation in cementitious materials is a continuous process between 0°C and –55°C.

Test results clearly indicate that:

- Ice does not form exactly at 0°C but only at lower temperatures;
- All water does not freeze at the same temperature. On the contrary, the interstitial fluid freezes progressively in the porosity as the temperature is decreased (a continuous heat release corresponds to a continuous ice formation process, see Figure 11.1).

These phenomena cannot be associated with any kinetic effects since they are observed even when the experiment is conducted at low freezing rates.

Low-temperature calorimetry experiments clearly indicate that liquid water and ice co-exist in a wet porous body kept at sub-zero temperatures. From thermodynamic considerations, it can be shown that at a temperature below the normal freezing point of water (0°C at atmospheric pressure), ice and water can be in equilibrium (i.e. some water molecules remain in a liquid state while others can be found in a solid state). These equilibrium conditions correspond to the equality of the chemical potentials of the various phases (ice and water in a saturated medium; ice, water and vapour in a partially saturated body) present in the system. The chemical potential of a given pure phase is, in general, a function of temperature and pressure. Thus, two phases at the same temperature can co-exist providing that the pressure in each of them is, in general, different. In that case the mechanical equilibrium between the two phases involves the characteristics of the interface.

The two co-existent phases are necessarily separated by a surface called the interface. Laplace's law describes the mechanical equilibrium between two phases subjected to a hydrostatic stress and separated by a curved interface (Defay *et al*., 1966). A pressure difference is then created between the two phases. This difference in pressure is related to the curvature and the superficial tension of the interface (which corresponds to the energy required to increase the interface by a unit). For a spherical interface, the thermodynamic equilibrium at temperature T is reached when the principal radius of curvature of the interface between ice and liquid water, R_{eq} is equal to:

$$R_{\mathrm{eq}} = \frac{2 \cdot \gamma_{\mathrm{ls}}}{\displaystyle\int_{T_0}^{T} \frac{\Delta s^{\mathrm{ls}}}{v^{\mathrm{l}}}} \tag{11.1}$$

γ_{ls} is the superficial tension between ice and liquid water, Δs^{ls} is the entropy of ice formation (negative for the solidification of water), v^{l} is the molar volume of liquid water and T_0 is the equilibrium temperature for a flat interface (0°C for liquid water/ice).

According to equation (11.1), at a temperature T, below the normal freezing point T_0, a curved interface having a radius of curvature R_{ed} is required to allow ice to co-exist with liquid water in the system. Conversely, the formation of ice is subjected to geometrical limitations. For instance, at a temperature T, ice is only able to penetrate spaces for which the radius of the curved interface is greater or equal to R_{eq}.

This last case is observed when a porous medium is considered. Figure 11.2 shows a saturated porous medium exposed to decreasing temperatures (from the normal freezing point to lower temperatures). At the beginning (Figure 11.2(a)), ice is present only where the existence of a flat interface is possible (exterior surfaces or in large air bubbles). Then, as the temperature is lowered, the radius of curvature of the solid/liquid interface at equilibrium is decreased according to equation (11.1). As a result, the lower the temperature, the smaller the pores which ice can penetrate.

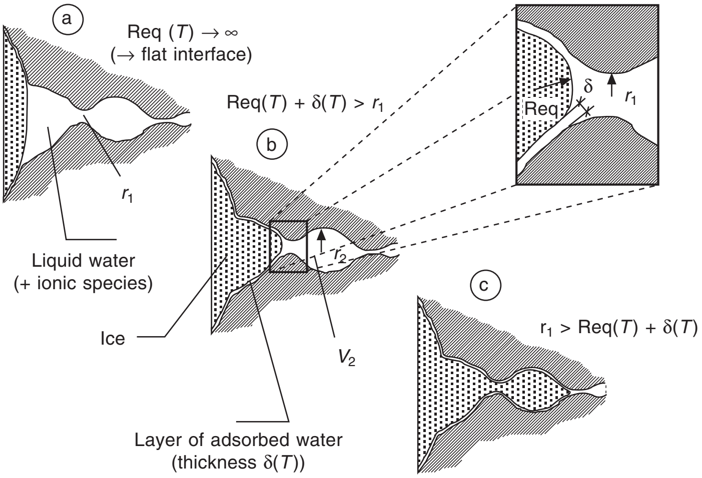

Figure 11.2 Progressive penetration of ice/liquid water interface in a complex porous network. The lower the temperature, the smaller the pores in which ice/liquid interface can penetrate.

This process of ice formation is clearly determined by the radius of the pore entryways (and therefore the pore size distribution of the material). As can be seen in Figure 11.2(c), the volume V_2 can only freeze if ice has propagated through the smaller pore of radius r_1. This process is very similar to the phenomenon of capillary condensation and to the penetration of mercury during a porosimetry experiment. Indeed in these two cases the external loading parameters are respectively the relative humidity and the mercury pressure. In the first case, relative humidity controls the curvature of the liquid–vapour interface. In the second case, pressure controls the curvature of the mercury–vapour interface. In our case, the loading parameter is temperature which, through equation (11.1), controls the curvature (and thus a pressure difference) of the interface between ice and liquid water.

It should be emphasized that the interstitial fluid present in the porosity is not pure water but a concentrated ionic solution. The properties of this solution (such as its superficial tension and its chemical potential) tend to vary as the temperature is decreased and ice is formed in the system. This point is important since the temperature of solidification of a given solution is directly related to its concentration: the more concentrated the solution, the lower the temperature of solidification.

A difference appears when water is in contact with a solid surface. Due to surface forces, some water molecules are adsorbed on the pore walls. This adsorbed water has almost acquired the structure of ice although its physical behaviour remains that of liquid water. This liquid-like layer of adsorbed water has a thickness which depends on temperature.

Experiments show that the adsorbed water is not frozen at – 60°C. It freezes at lower temperatures as shown by dynamic measurements (Radjy, 1968). In addition to their influence on the properties of the interstitial fluid, ions in solution can also affect the forces acting at the surface of the porous solid and therefore influence the behaviour of the adsorbed phase. The adsorbed water may play a very important role in the basic mechanisms of degradation.

The previous discussion mostly focused on the behaviour of saturated systems. The mechanisms involved in the formation of ice in an undersaturated porous solid are similar to those previously described. However, the presence of a second interface (a solid/vapour interface) may contribute to modify the kinetics of the solidification process.

The depression of the freezing temperature of water has often been associated with nucleation (supercooling) effects. The nucleation of ice is the result of a thermostatistic process (involving molecular shocks and energy concepts) leading to the appearance of an ice crystal from the liquid state. Although supercooling is likely to account for a fraction of the freezing point depression of water, nucleation effects play an important role at very low temperatures.

11.3 Mechanisms induced by ice formation

Many processes can, at least in theory, be at the origin of the degradation of concrete by frost. In fact, any phenomenon able to generate stresses or strains exceeding the mechanical properties of the solid skeleton during the exposure of the material to sub-zero temperatures can induce irreversible damage. The mechanisms of frost deterioration of concrete remain imperfectly understood, and are still the subject of a great deal of interest from researchers and engineers.

Ice formation results in a 9 per cent volume expansion of water. Therefore, if ice formation is initiated in a pore, water will be expelled from this cavity. If this water flow is hindered, hydraulic pressures will be generated within the solid matrix. This phenomenon explains why dilations are observed during freezing. It is also consistent with the fact that air entrainment can improve the frost resistance of concrete. The presence of air bubbles reduces the distance that water has to travel and contributes to ease the dissipation of internal pressures.

For saturated materials (or materials near their saturation point), the generation of hydraulic pressures is commonly considered to be the most important cause of frost deterioration. However, experiments can also lead to contractions much greater due to the thermal shrinkage of the material. The contraction of the material upon freezing cannot be explained on the basis of the hydraulic pressure theory.

Nevertheless, some explanations can be provided. Some authors have linked the contraction to the presence of ice crystals in some pores and liquid water in others. If pores are considered to have a cylindrical shape, the ice crystal/pore wall interface is thus also cylindrical (even with the presence of an unfrozen layer of (liquid-like) adsorbed water between ice and the pore walls). However, (according to equation (11.1)), the interface between the ice crystal and the unfrozen water (present in the smaller pores) is spherical. On the basis of thermodynamic and mechanical equilibrium considerations, a relation between the hydrostatic pressures developed in the solid and liquid phases and the stress applied to the pore walls can be derived. As can be seen in Figure 11.3, this relation is only valid for a given pore geometry. If the ice crystal is assumed to be subjected to a uniform internal stress at a value near that of the atmospheric pressure (for instance, if the crystal is in equilibrium with vapour through a flat interface), the stresses generated on the pore walls by the presence of ice and water globally lead to a depression, resulting in a contraction of the porous body (Zuber and Marchand, 2000; Zuber, 2002).

Another mechanism is often mentioned to explain the dilations and contractions observed during a freezing experiment. If, for any reason, a certain fraction of unfrozen

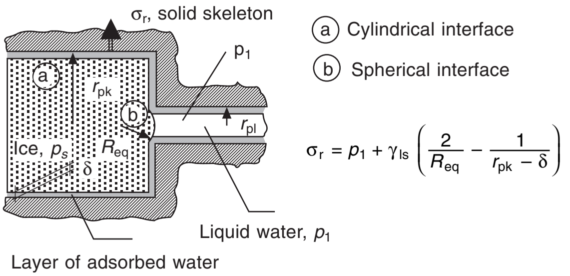

Figure 11.3 Hydrostatic stress generated in the solid skeleton when ice is present in the porous network.

water is maintained in a supercooled state*, this volume of liquid water is, therefore, not in thermodynamic equilibrium with the solid phase already formed in the system. The gradient in chemical potential initiates the transport of the unfrozen water to the freezing sites. These movements can create dilations in two ways, first, by generating hydraulic pressures during the flow of water, second, by supplying the growth of existing ice crystals (which can create crystal pressures as seen during ice-lensing phenomenon in frozen soils). Contractions are, in this context, explained by tensions generated by the suction of the water from pores where it cannot freeze. In partially saturated media, these mass transfers occur by diffusion of the vapour phase (Litvan, 1978).

Mixture characteristics can affect the frost resistance of the material in various ways. The porosity of the material directly influences the amount of freezable water and transfer properties (permeability and diffusivity) of the solid. Admixtures, mineral additions and the physical and chemical properties of the cement can influence the frost behaviour of the resulting material in two ways. On the one hand, they can influence the mechanisms of air entrainment (and therefore affect the spacing factor of the material). On the other, they can modify viscosity, surface tension and thermodynamic properties of the pore fluid as well as the surface forces.

Conditions of exposure are also of great importance. Saturation clearly plays a role in controlling the amount of freezable water and, in a certain way, the efficiency of the air bubble network. De-icing salts could play a role in the physical characteristics of surface forces and of the pore fluid. Even if the rate of cooling does not appear to have any influence of the amount of water that can freeze at a given temperature (at least not for the cooling rates usually found in practice), it tends to affect the hydraulic pressures generated within the solid upon freezing. The duration of the freezing period is also found to play an important role in the behaviour of concrete exposed to frost action.

As emphasized in the previous paragraphs, the frost durability of concrete is affected by both the characteristics of the mixture and the exposure conditions. Field and laboratory performances will be critically reviewed in the following sections.

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*A supercooled phase is a phase that could freeze according to thermodynamic considerations but that has not frozen.

11.4 Laboratory testing and influence of various parameters

The frost resistance of concrete is generally determined by subjecting specimens prepared in the laboratory to a number of cycles of freezing and thawing in water, or of freezing in air and thawing in water, in the temperature range of +4°C to –18°C or –20°C. In order to obtain results in a relatively short period, the specimens are generally subjected to five or more cycles per day, since, as in the ASTM C666 Standard procedure, the number of cycles is often fixed at 300. To evaluate the degree of internal cracking, and thus the damage due to frost action, the two most common procedures are length change measurements (ASTM C671) and dynamic modulus of elasticity measurements. A length change of more than 200 μm/m (approximately) or a loss of modulus of elasticity normally indicates that the internal structure of concrete has been significantly damaged by microcracking. Loss of mass can also be measured, but it is more related to the resistance to surface scaling than to internal cracking, and scaling resistance is a property normally determined with de-icer salt scaling tests, as described in the following section.

Laboratory testing has shown conclusively that there exists, for almost all types of concrete, a critical value of the air void spacing factor. If the spacing factor is higher than this critical value, the concrete specimen being tested is destroyed very rapidly by the cycles. Microcracking occurs, and there is a rapid loss of mechanical properties. If the spacing factor is lower than this critical value, the concrete specimen can withstand a very large number of cycles without any significant damage. Figure 11.4 shows the results of a series of freezing and thawing cycle tests carried out on a typical Portland cement concrete. All mixtures were prepared at a constant free water to cement ratio of 0.5, but with different air-void networks. As the results show, there exists, for this concrete, a

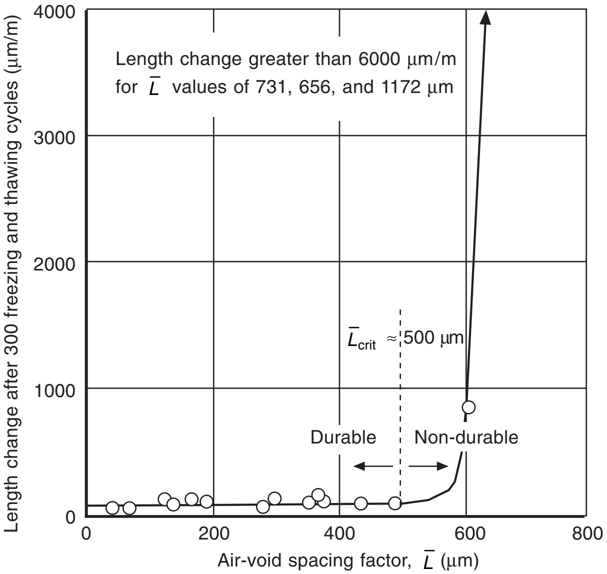

Figure 11.4 Freeze–thaw critical spacing factor (for a standard concrete of w/c: 0.5).

critical value of the air-void spacing factor. All mixtures with a spacing factor significantly higher than 500 μm were very quickly destroyed by the cycles. This behaviour is typical of what is observed in the laboratory: the frost resistance of a concrete specimen is generally either very good or very poor. As Figure 11.4 shows, a moderate degree of deterioration is not often observed.

The critical value of the air-void spacing factor varies with many parameters, but mainly with those that influence the porosity: the water to binder ratio, the type of binder, the length of curing, and the use of certain admixtures. It also, of course, depends on the test conditions, i.e. mainly the freezing rate, the minimum temperature, the length of period at minimum temperature, and the availability of water. It has been shown experimentally that the critical value of the spacing factor decreases as the rate of freezing during the tests increases. Interestingly, for most good quality concretes with a water-to-binder ratio of 0.6 or less, irrespective of the type of binder (and even for sprayed concretes or latex modified concretes) tested according to one of two ASTM C666 procedures (freezing and thawing in water or freezing in air and thawing in water), with perhaps the exception of certain high-performance concretes (see section 11.7), the critical value of the air-void spacing factor varies between 200 μm and 600 μm. The 200 μm value is typical of properly air-entrained concrete, and 600 μm corresponds to the lower end of the range for non-air-entrained concrete. In view of this, it is hardly surprising that most codes of practice (see, for instance, CSA-A23.1/A23.2) recommend a maximum value of the air-void spacing factor of 200 μm, especially since, as will be seen in the next section, this value is also that required for good resistance to scaling due to freezing in the presence of de-icer salts. As early as 1949, on the basis of laboratory testing, Powers suggested a value of 250 μm.

In order to evaluate the influence of any given variable on the frost resistance of concrete, it is necessary to determine the critical air-void spacing factor for the concrete under consideration, and then to compare it with that of a reference mixture. A higher critical value indicates a better performance, since the concrete requires a lower degree of protection against frost, and a lower value, a poorer performance. Very often, investigators are misled because the critical spacing factor is not determined. It is therefore quite possible that the observed positive influence of a given admixture on frost resistance, for instance, is not due to an improved microstructure, but simply to an improved air-void system!

Aggregates are an important component of any concrete, and they must, of course, always be selected properly to ensure that they will not influence negatively the frost resistance of concrete. Certain aggregates, generally characterized by a high porosity and a low average pore size, are simply not resistant to frost. Due to their fine pore structure, they easily become saturated, and the pressures due to the movement of water when ice forms are higher than the tensile strength of the aggregate. This is particularly the case with large aggregate particles, since the distance that water must travel during freezing is then high. Other types of aggregates, even if frost resistant, can have a negative influence by expelling water in the surrounding paste upon freezing. A high porosity, a 2 per cent absorption generally being considered an upper limit, is an indication of potential problems. Obviously, access to water is again a very important condition, and a low paste porosity therefore helps to reduce the degree of saturation of the aggregates at the time of freezing. Air-entrainment is also important, since the air voids close to the paste–aggregate interface can help to reduce the pressures that are due to the expulsion of water by the aggregate into the surrounding paste.

A relatively common type of frost deterioration is what is called in North America D-line cracking (deterioration-line cracking). As previously mentioned, the availability of moisture is a basic condition for frost deterioration, and such is often the case close to joints in concrete pavements. If the concrete is not sufficiently protected by air entrainment, or if certain types of aggregates are used, the frost damage creates a pattern of cracks close to and parallel to the joints.

Considering the importance of the degree of saturation as regards frost resistance, Fagerlund (1975) developed the critical degree of saturation concept. For any concrete, there exists a critical degree of saturation such that frost damage will inevitably occur if the concrete freezes while the degree of saturation is higher than the critical value (see Figure 11.5). The longer it takes for a given concrete to reach the critical degree of saturation, the better is the frost resistance. Obviously, good-quality properly air-entrained concretes take a very long time to reach critical saturation, particularly since the capillary forces are very small in the air voids (most air voids have a diameter larger than 25 µm). This concept underlines the importance of access to water and can be used to predict service life, i.e. the time required to reach critical saturation under field conditions.

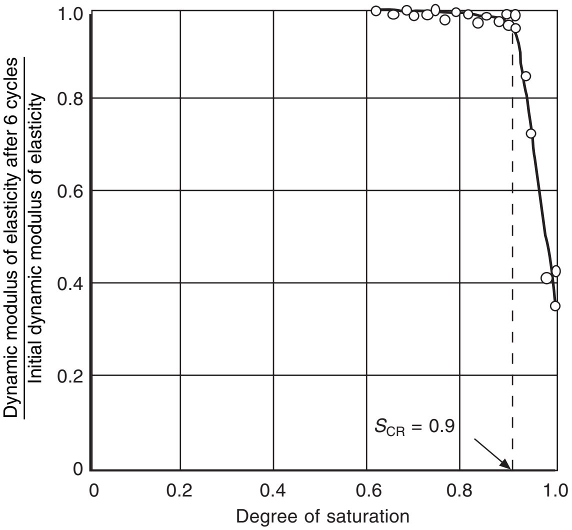

Figure 11.5 Relationship between the relative dynamic modulus of elasticity and the degree of saturation of concrete.

11.5 De-icer salt scaling

Experimental evidence (from both laboratory tests and field observations) has clearly demonstrated that the deterioration due to frost can be amplified by the use of de-icer salts. When concrete freezes in the presence of a salt solution, surface scaling can occur if the concrete is not protected by an adequate air-void network, or if the microstructure of the surface layers is not sufficiently dense and homogeneous. Small particles of paste and mortar are broken from the concrete, and the mass of broken particles increases with the number of cycles, gradually exposing the coarse aggregates.

Although the basic mechanisms still require further investigation, it has been shown that many types of chemicals can cause scaling (which indicates that it is a physical, not a chemical, process), and also that there is a pessimum concentration at which the surface deterioration is maximized (see Figure 11.6). This concentration, typically around 3 per cent for sodium chloride, is now that used in most laboratory tests such as ASTM C672. Testing is therefore carried out under very severe conditions. However, as can be seen in Figure 11.7, very severe scaling can also be observed in the field, even if concrete does not freeze with a saline solution on the surface, but simply freezes while the top layers have been contaminated with large amounts of salt.

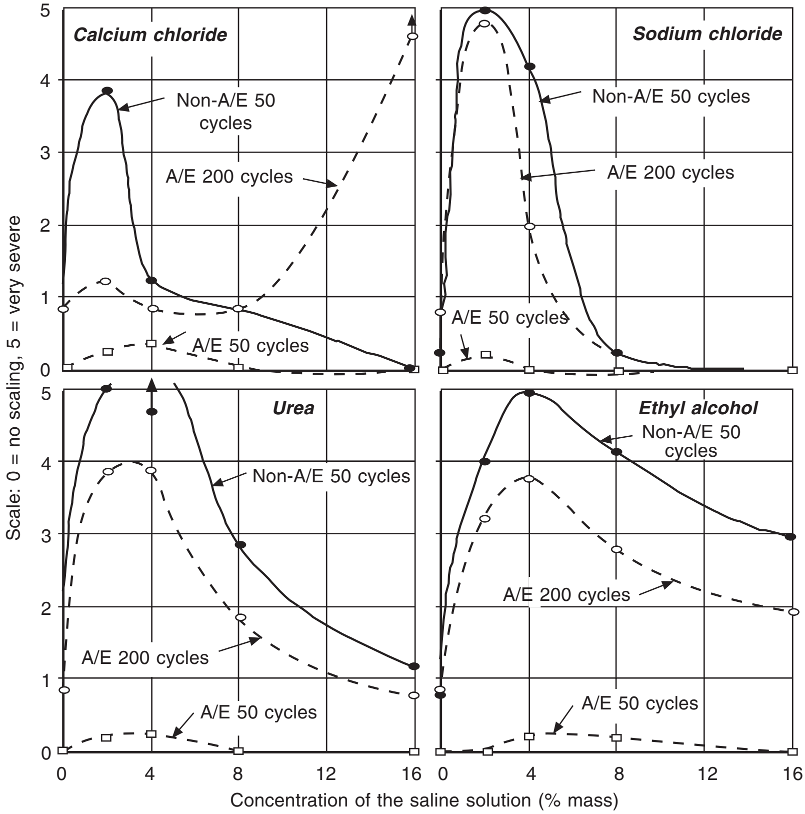

Figure 11.6 Influence of the concentration of the solution and the type of chemical on the degree of surface scaling due to freezing and thawing cycles in the pesence of a de-icer solution.

One of the most important aspects of the scaling problem, which was briefly mentioned in the Introduction, is the microstructure of the surface layers. The surface of a concrete element can be quite different from the rest. It can be more porous, or even, in certain cases, more dense. When a test is carried out, the relationship between the mass of residues and the number of cycles is therefore not always linear (see Figure 11.8). A porous layer is destroyed very rapidly by scaling, and then the process becomes very slow if the concrete has a dense and homogeneous microstructure and is properly protected by air entrainment. If the concrete is not well protected, the mass of residues keeps on

Figure 11.7 Scaling on a wall exposed to de-icing salts during winter. In addition to architectural damage, severe scaling can lead to progressive structural deterioration.

increasing with the number of cycles, as Figure 11.8 shows. If the concrete is well protected and has no porous layer, the mass of residues increases extremely slowly with the number of cycles.

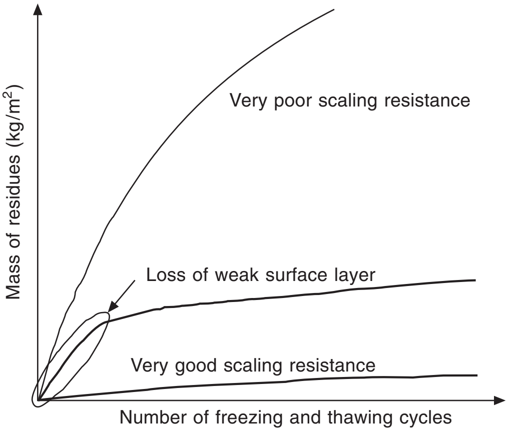

Figure 11.8 Typical de-icer salt scaling test results. Between poor and good scaling resistance, one can observe an initial deterioration of the weak surface layer followed by a good residual scaling resistance.

The conditions that can lead to the formation of a porous layer are not all clear, but porous surface layers are apparently more common in laboratory specimens cast in moulds and subjected to bleeding which probably tends to dilute the surface of the element before setting occurs. In the field, sidewalks, for instance, are normally cast on a granular base (which reduces bleeding), and the bleed water on top generally evaporates as soon as it reaches the surface. Recent testing has shown that the top surface of a sidewalk can be as scaling resistant as the inner concrete. This was verified by testing sawed surfaces for scaling resistance. In fact, the best sidewalk (for scaling resistance) is that which has a top surface as resistant to scaling as the inner concrete. To study the basic mechanisms of scaling, or to analyse the influence of any parameter (type of cementitious material, curing technique, etc.), it is thus recommended to test sawn surfaces, and in addition, for a more complete understanding, troweled or moulded surfaces.

The basic parameters that influence scaling due to freezing in the presence of de-icer salts are the same as those that influence frost deterioration: the characteristics of the capillary pore system and those of the air-void network. It should be mentioned, however, that a good scaling resistance generally requires a better microstructure (reduced porosity and increased homogeneity) than the simple resistance to freezing and thawing cycles (presence of water, of salts, higher number of cycles and decreased minimum temperature). Many concretes are frost resistant, but not de-icer salt scaling resistant.

The use of fly ash and slag generally tends to reduce somewhat the resistance to scaling, since it decreases the permeability while the total amount of freezable water remains more or less constant. Recent unpublished results indicate that this would be due to a lower paste porosity but a higher interfacial zone porosity. Concretes containing fly ash and slag therefore require a better protection, i.e. a closer system of air voids. This is not the case for microsilica which generally improves the resistance to scaling. The characteristics of the coarse aggregates can also have an influence. Certain types of aggregates can cause pop-outs, either because they can be broken during freezing if they are close to the surface or because they expel too much water upon freezing, which tends to break the mortar cover over the aggregate particle.

11.6 Air entrainment

As explained by Powers (1975), air is entrained in the fresh concrete by the mixing process, due either to the creation of a vortex (stirring) or simply to concrete falling on itself (kneading). This explains the importance of the type of mixer, as well as of the characteristics of the aggregates, since the relatively large entrapped air voids are split into smaller voids by the movement of aggregates in the mixer. The role of air-entraining admixtures is not only to facilitate this process of air entrainment, but also to stabilize the air voids that are formed. Air entraining admixtures tend to concentrate at the air–water interface and to bind the air voids to the surrounding clinker grains through various processes.

Energy is an important consideration in air entrainment, since it takes energy to form surfaces (the air–water interfaces), and thus to form air voids and split them into smaller voids. In addition, the normal tendency of small air voids is to coalesce into larger voids with a lower specific surface, and therefore a lower energy. Large voids can be more easily expelled from the paste by buoyancy.

Once the concrete has been cast, the air-void network is normally fixed, and it stays fixed until the concrete hardens. Only if the dormant period is very long can a significant number of small air bubbles be dissolved into the surrounding water. However, under normal conditions, the spacing factor stays relatively constant from the time of casting until setting occurs.

The common procedure to determine the air-void spacing factor (and the other characteristics of the air-void network) is described in ASTM Standard C457. It relies on microscopical examinations of polished concrete sections (at a magnification which can range between 50× and 125×), and the calculations are based on simplifying assumptions concerning the size distribution of the air voids. This factor is therefore not a very precise indication of the protection offered against frost, and more precise factors have been suggested.

Many parameters can affect the spacing factor obtained through air entrainment. For obvious reasons, the type and dosage of the air-entraining admixture is the most important. The characteristics of the mixer also play a significant role. For instance, high-energy laboratory mixers with sharp blades are generally more efficient than the typical mixers used in concrete plants, particularly when the viscosity of the cement paste is high. It is more difficult to entrain air in a very viscous paste (such as, for instance, low water to binder ratio pastes containing silica fume). In fact, if the water content is too low, it is simply impossible to entrain air, since air bubbles must form in water.

The influence of the characteristics and grading of the aggregates on air entrainment can be quite significant. In pastes, air entrainment is almost impossible, since most of the bubbles formed are expelled by buoyancy. In mortars and concretes, however, the fine aggregates form a grid which traps the air voids and prevents them from rising to the surface.

All the parameters that influence the spacing factor also influence the total volume of air obtained. But many other variables, such as the temperature of the mixture, for instance, also influence the air content. Most of the air volume is in the voids larger than 100 µm. However, the spacing of the air voids is basically determined by the voids smaller than 100 µm, which are much more numerous (more than 100 times). It is therefore very important not to be misled by the air content when designing a mixture for frost resistance. Figure 11.9 shows the relationship between the air volume and the spacing factor determined on the hardened concrete for a very large number of different concretes. It can be seen that, at a given air content, the spacing factor can be extremely variable, since the number and size of the voids smaller than 100 µm can be quite variable. Frost-resistant mixtures must thus be properly designed, which requires the determination of the spacing factor on samples cast from the mixture after about one hour after the first water–cement contact. As many publications have emphasized in recent years, the characteristics of the air-void

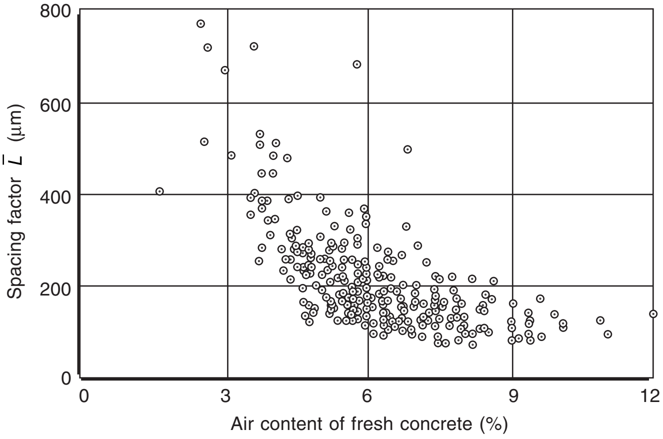

Figure 11.9 Relation between air content versus spacing factor. Spacing factor is not directly linked to air content.

network can vary with the time of casting, particularly if concrete is continuously agitated in a readymix truck, and it is very important to ensure that the concrete being cast still has adequate air-void characteristics.

The most difficult problem concerning air entrainment is to ensure that an adequate air-void spacing factor will be obtained with only a limited volume of air. Sometimes, for reasons which are not evident, a properly designed mixture will have a very high air content. Since this reduces strength, most producers will automatically reduce the dosage of the air-entraining admixture to decrease the air content. Unfortunately, this can result in a spacing factor larger than the desired value. In such situations, the only possibility is to use a trial-and-error procedure to determine the cause of the problem.

Normal placement procedures, such as vibration with needles, do not significantly affect the air-void spacing factor of a properly designed mixture, since only large voids are expelled form the mixture. However, if the air-void network is of poor quality, or if the mixture is unduly vibrated, the in-place concrete can have a spacing factor much higher than that required for good frost resistance. Pumping can also influence the air-void system, since the pressure that is used tends to dissolve the smaller (and most important) air voids. If pumping is used, tests should always be carried out to determine the characteristics of the air-void system on specimens cast after pumping.

11.7 Special concretes

In recent years, the use of special concretes, such as high-performance concrete, self-levelling concrete, roller-compacted concrete, sprayed concrete, or fibre-reinforced concrete, has become quite common. The normal concretes of today are in many ways different from the normal concretes of the past. It is thus important to consider the frost resistance of these concretes, and to try to understand how the basic principles described in this chapter can be applied to them.

High-performance concretes are globally characterized by a reduced paste porosity, a finer pore size distribution, and an increased tensile strength. They are also particularly affected by self-desiccation. This phenomenon tends to reduce the degree of saturation during freezing, first, because hydration rapidly reduces the amount of water available in the capillary pores, and second, because the ingress of water into the body of the concrete is very slow in such a dense material. It is possible, on the basis of laboratory tests, to determine a critical water-to-binder ratio below which air entrainment is not necessary as a protection against frost or de-icer salt scaling. At a certain point, the amount of freezable water simply becomes too small to generate significant water movements and stresses, and, as previously pointed out, the freezing point of water decreases with the size of the pores. The critical water-to-binder ratio, of course, varies with the test conditions, the type of cement, the type of curing, and the mixture characteristics. As a general guide, and based on the available information, it can be stated that air entrainment is generally not required when the water-to-binder ratio is lower than 0.25. It is, however, generally required if the ratio is higher than 0.35. For intermediate values, air entrainment should be used unless testing has shown it is not required, for both frost and de-icer salt scaling resistance.

Roller-compacted concretes and other types of dry-cast concretes have a reduced paste content (compared to normal cast in-place concrete), and air entrainment can be very difficult in this type of mixture. In addition, it has been shown that the paste is often not as homogeneous as in normal concretes, although the use of supplementary cementitious materials apparently tends to increase the homogeneity of the paste. Considering in addition the presence of the voids due to incomplete compaction which can act as air bubbles to relieve the pressures due to freezing, it can be seen that the frost resistance of these concretes can be a very complex problem, and it is therefore impossible with the present state of the knowledge to give very precise and usable information on this subject. The frost and de-icer salt scaling resistance of dry-cast concretes should therefore be verified by tests, and microscopic examinations should be performed to verify the characteristics of the air-void system obtained with a given air-entraining admixture and a given mixing procedure. Recent results indicate that roller-compacted concretes made with silica fume cement at a water-to-binder ratio of approximately 0.35 to 0.40 can be frost and de-icer salt scaling resistant without the use of air entrainment (Marchand *et al.*, 1992; Pigeon and Marchand, 1996).

Self-levelling concretes are characterized in the fresh state by a low yield stress and a high viscosity. This is generally obtained with a high paste content and the use of viscosity-modifying admixtures. The high mobility or fluidity of these mixtures raises the question of air entrainment. Is it possible to entrain, and retain until setting, a proper network of air voids in these mixtures, and obtain a good resistance to frost and de-icer salts? Recent research (Lacombe, 1997) has shown that the answer to this question is yes, but this requires particular mixing procedures and the use of very carefully selected admixtures, and the result in terms of the air-void spacing factor should always be verified. Similar comments can be made concerning fibre-reinforced concretes. It can be difficult to produce properly air-entrained fibre-reinforced concrete, since the loss of workability due to the fibres is generally compensated with a higher fluidity of the mortar fraction, and tests to verify the air-void spacing factor in the hardened concrete should also always be made.

Sprayed concrete (the word ‘shotcrete’ is more common in North America) is now quite frequently used for the repair of superficially damaged structures in areas where the exposure conditions are severe (numerous freezing and thawing cycles and frequent use of de-icer salts). For dry-mix shotcrete mixtures (i.e. which are produced by adding water to the dry materials at the nozzle), the use of large dosages of an air-entraining admixture, either in powder form or mixed with the water used for shooting, has been shown to be successful (Lamontagne *et al.*, 1996) and these shotcretes can thus be made frost- and de-icer salt scaling-resistant. Wet-mix shotcrete can also be made quite frost resistant through the use of air entrainment, but the loss of some of the small air voids during pumping can apparently reduce the de-icer salt scaling resistance (which normally requires a lower spacing factor than simple resistance to freezing and thawing cycles).

11.8 Field performance

There are many significant differences between the exposure conditions in typical laboratory tests and those in the field. The temperature during laboratory freezing and thawing cycles generally varies from 4°C to –18°C or –20°C in only a few hours, which is uncommon in most areas where severe winter conditions exist. However, very long freezing periods are not typical of laboratory tests (for obvious reasons), but are quite normal under natural exposure conditions. The degree of saturation of concrete can also be very different in the laboratory and in the field. In laboratory tests, concrete is normally not only prevented from drying but also has access to water at each cycle which increases very significantly the possibility of frost damage, as recent testing has shown (Pigeon and Pleau, 1995). It is therefore extremely important to analyse very carefully the results obtained by observation of the field performance of concretes of known characteristics.

The most important lesson learned from field studies, in North America as well as in Europe, is the importance of a proper network of air voids if the concrete element under consideration can have access to water and is subjected to freezing temperatures significantly below 0°C. For normal good-quality concrete, on the basis of a vast amount of field data, an air-void spacing factor not higher than 200 μm can be considered as a safe design value, particularly if exposure to de-icer salts is expected. Field observations have very clearly demonstrated the highly detrimental influence of de-icer salts which tend to increase very significantly the surface damage due to frost.

Field studies have also shown that laboratory testing is very severe and can often, but not always, significantly underestimate field performance. However, considering the very low cost of air entrainment in most cases as well as the protection that it offers against possible damage due to frost, it is sound engineering practice to use air-entrained concretes in areas where severe winter conditions prevail. As explained in a previous section, only certain types of high-performance concretes can be considered to be frost resistant without the use of air entrainment.

Another important lesson learned from field studies is that air entrainment in a laboratory mixer is very different from air entrainment in a plant or in a readymix truck (i.e. in a so-called dry batch plant). Mixtures designed for frost resistance should therefore be tested under field conditions (large batches, long waiting periods before casting) to determine if the cast-in-place concrete will have the proper air-void spacing factor. It is quite easy, for instance, to obtain an unstable air-void network if many different admixtures are used.

Of course, as always, it must be remembered that good curing is a basic condition to obtain concrete with the desired properties of strength and durability. In this respect, a significant information has been obtained recently (Pigeon and Pleau, 1995) through many field (and laboratory) studies. Although the reasons for this are not clear, it appears that the use of a curing compound is better for good scaling resistance than water curing. Luckily for researchers, concrete still holds many secrets!

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12

Acid, soft water and sulfate attack

Steve Kelham

12.1 Aqueous solutions

12.1.1 Pure water, pH, acids, bases

Pure water does not consist of purely H_2O molecules but dissociates to a small extent into charged ions.

$$H_2O = H^+(\text{hydrogen ion}) + OH^-(\text{hydroxyl ion}) \tag{12.1}$$

In fact the free proton H^+ will be associated with one or more water molecules and for practical purposes equation (12.1) can be written

$$H_2O + H_2O = H_3O^+ + OH^-$$

The product of the molar concentrations (in moles/l) is the ion product constant for water, which has the measured value 10^{-14} at 25°C.

$$[H_3O^+][OH^-] = 10^{-14} \tag{12.2}$$

where square brackets '[ ]' indicate molar concentrations. The small value indicates that there is little self-ionization and the equilibrium in equation (1) is very much to the left, i.e. most of the water is not dissociated. Since the concentrations of H_3O^+ and OH^- must be equal in pure water it follows from equations (12.1) and (12.2) that each is equal to 10^{-7}.

The pH of a solution is defined by

$$pH = -\log_{10}([H_3O^+])$$

and therefore the pH of pure water is 7.

Acids may be defined as substances that increase the H_3O^+ concentration when dissolved in water and therefore reduce the pH. Since the ion product is constant, this implies that the OH^- concentration must be decreased. Similarly, a base is a substance that reduces the H_3O^+ concentration (increases the pH) and increases the OH^- concentration. For example:

$$\text{Acid } HCl + H_2O = H_3O^+ + Cl^- \tag{12.3}$$

$$\text{Base } NaOH = Na^+ + OH^- \tag{12.4}$$

12.1.2 Strong acids, weak acids, soft water

In hydrochloric acid the dissociation given in equation (12.3) is essentially complete and the molar concentration of H_3O^+ is equal to the concentration of the HCl solution. A 0.1 molar solution will give a H_3O^+ concentration of 0.1 (= 10^{-1}) or a pH of 1. Most inorganic acids give similar high degrees of dissociation and are termed 'strong' acids.

For most organic acids, such as acetic acid or humic acid, the degree of dissociation is low and the pH cannot be simply calculated from the concentration. Acetic acid gives

$$CH_3COOH + H_2O = H_3O^+ + CH_3COO^- \tag{12.5}$$

The dissociation constant K is defined by

$$K = [H_3O^+][CH_3COO^-]/[CH_3COOH] \tag{12.6}$$

and has the value 1.85×10^{-5}. The H_3O^+ concentration in 0.1 molar solution of acetic acid can be approximated by assuming that dissociation is small so that $[CH_3COOH]$ is ~0.1 and that the H_3O^+ concentration is sufficiently large that the self-ionization of the water can be neglected. The result is ~10^{-3}, i.e. ph 3, about 100 times less acid than the 0.1 molar HCl. Acids giving a low degree of dissociation are termed 'weak' acids.

The term soft water is used for water that contains dissolved CO_2. The atmosphere contains ~0.03% CO_2 and water in equilibrium with this is a weak acid with a pH of 5.6 due to the formation of carbonic acid.

$$CO_2 + H_2O = H_2CO_3 \tag{12.7}$$

$$H_2CO_3 + H_2O = H_3O^+ + HCO_3^- \tag{12.8}$$

$$HCO_3^- + H_2O = H_3O^+ + CO_3^{2-} \tag{12.9}$$

The dissociation constant for equation (12.8) is ~10^{-7} and for equation (12.9) ~10^{-11}. The concentration of HCO_3^- is thus small and that of CO_3^{2-} very small. The solution is, however, much more aggressive than expected because of the influence of equation (12.7). The equilibrium constant for this reaction is ~10^{-3}, i.e. very little of the dissolved CO_2 forms carbonic acid, but if HCO_3^- is removed by reaction more carbonic acid can be formed from the reservoir of dissolved CO_2. In underground waters that have been under high pressure and waters in contact with vegetable matter the quantities of dissolved (free) CO_2 can be large. The actual aggressiveness of the water depends not only on this free CO_2 but on the concentrations of calcium and other ions.

12.1.3 Sulfate solutions, sea water

The main chemical attack associated with concrete in contact with aqueous solutions is that associated with sulfates. The main sulfates encountered in ground water are calcium, sodium, potassium and magnesium. Concentrations of gypsum are normally low due to its low solubility but solid gypsum is often also present. Sulfates are found in clays and other soils throughout the world. Sulfates are also present in sea water but NaCl is the main salt present and this interferes with the sulfate reactions.

In solution the sulfates are ionized, e.g.

$$K_2SO_4 = 2K^+ + SO_4^{2-}$$

and reactions with both ions need to be considered, particularly in the case of magnesium.

12.2 Reactions of water and acids with concrete/mortar

12.2.1 Leaching

Pure water can damage concrete by dissolving out those hydration products with a significant solubility. The main hydrates will disappear in the order CH, monosulfate, ettringite, C-S-H, although the C-S-H will suffer gradual decalcification once the CH is unable to maintain the Ca ion concentration.

12.2.2 Reactions of hydration products with acids

Acids also attack concrete by dissolving the hydration products but the rate of attack is usually much greater than for pure water. Acids that form relatively insoluble calcium salts, such as H_2SO_4, are less aggressive. Acids can also attack some aggregates, notably limestone.

12.3 Factors affecting rate of attack by water and acids

12.3.1 Solution chemistry, solution availability

The aggressiveness of solutions is not directly related to the pH or concentration of dissolved ions, although it is important to know these values when assessing exposure conditions. The solubility of associated calcium salts and concentraton of dissolved CO_2 also affect the rate of reaction as discussed above.

As important as the solution chemistry is the availability of the solution for reaction with the concrete. Flowing or percolating water will cause more rapid deterioration than small quantities of still water, which will gradually approach equilibrium with the concrete. Flow can carry away loosened aggregate and precipitated reaction products as well as dissolved solids.

In the case of water in soil or clay the important parameter is the permeability of the soil. A low permeability clay with an apparently very aggressive water chemistry will lead to little damage to a concrete surface in contact with it because the water at the surface will be rapidly neutralized.

12.3.2 Concrete quality

No concrete is resistant to acid but dense, low w/c (water/cement) concrete will reduce the rate of attack.

12.3.3 Cement type

The main effect of the use of different cements is likely to be to change the permeability and hence the rate of attack. The use of pozzolanic materials to reduce the CH content also reduces the rate.

12.3.4 Aggregates

The type of aggregate does not have a large effect on the rate of reaction but in some cases, such as concrete pipes, it is important that the deterioration should not lead to debris which could be carried away and cause further problems. In these cases limestone aggregates are used so that they too are dissolved and the concrete surface remains smooth. If the supply of acid is restricted limestone aggregate will also assist in neutralizing it.

12.4 Reactions of sulfate solutions with concrete

12.4.1 Aluminate hydrates, ettringite (AFt), monosulfate (AFm)

Portland cements contain sulfates to control the reaction rate of the C_3A. Alkali sulfates from the clinker and the added gypsum dissolve rapidly and in the presence of this high concentration of sulfate ions the C_3A reacts to form ettringite.

$$C_3A + 3CaSO_4(aq.) = C_3A\cdot 3CaSO_4\cdot 32H_2O$$

In cement hydration pure phases are rarely found and the ettringite phase is often referred to as AFt (alumino-ferrite-tri). If the cement contains sufficient C_3A to react with more sulfate than is available, further reaction converts the AFt to AFm (alumino-ferrite-mono) or monosulfate.

$$C_3A \cdot 3CaSO_4 \cdot 32H_2O + 2C_3A = 3(C_3A \cdot CaSO_4 \cdot 12H_2O)$$

The ettringite crystal contains a lot of water and its formation involves a large increase in solid volume. However, in the early stages of hydration this is easily accommodated.

A reasonably mature concrete will contain a mixture of AFt, AFm, some unhydrated C_3A, rather more unhydrated C_4AF and some other aluminate and alumino-ferrite hydrates, such as hydrogarnet phases (C_3AH_6 and related solid solutions).

12.4.2 Delayed ettringite formation

In the alkaline environment of hydrating Portland cement ettringite is unstable at temperatures above ~70°C. If hydration takes place at higher temperatures, through the use of steam curing of precast production or through self-heating in large pours, then ettringite is not formed. Studies of mortars immediately after heat treatment indicate either no sulfate containing phases or small quantities of monosulfate. Much of the sulfate is absorbed by the C-S-H and some is in solution in the pore fluid, particularly in high alkali systems. Similarly, no aluminate hydrates are visible. The alumina content of the C-S-H is higher than usual and some hydrogarnet may be detected.

When the concrete returns to ambient temperature this assembly of phases is not stable and ettringite begins to form and crystals grow in available pore spaces in the paste structure. In some cases this leads to expansion of the paste and cracks form through the paste and particularly around aggregate particles. These cracks are then also filled with ettringite crystals. This damage mechanism is termed Delayed Ettringite Formation (DEF) and is a form of ‘internal’ sulfate attack. Although no external sulfate or other ions are involved damage only occurs in wet concrete.

12.4.3 Reactions with external sulfate

External sulfate ions can react to form the same phases as the internal sulfate. In particular, reaction with unreacted C_3A or AFm to form AFt will lead to a solid volume increase that may be disruptive. An additional reaction with external sulfates is with CH to form gypsum.

$$Ca(OH)_2 + SO_4^{2-} = CaSO_4 \cdot 2H_2O + 2OH^-$$

If all the CH is consumed decalcification of C-S-H can take place. Thus, in addition to the expansive formation of the various reaction products, the paste structure can be destroyed.

For reaction to take place the sulfate ions must penetrate the concrete. Studies of affected concretes, mortars and pastes indicate that the reactions take place in stages. The first change is the conversion of AFm to AFt, which forms as microcrystals within the C-S-H. This does not seem to cause any significant cracking. In the second stage the reaction with CH forms gypsum within the C-S-H but also in veins with associated cracks. Some decalcification of the C-S-H also takes place. In sodium sulfate solutions the crack formation seems to be the main damage mechanism while with magnesium there is less cracking but more rapid loss of C-S-H and hence destruction of the paste structure. Of the main sulfates encountered magnesium is unusual in having a relatively insoluble hydroxide (brucite). The reaction of sulfate with CH is accelerated by the precipitation of brucite, which lowers the pH. The magnesium ion also attacks C-S-H, forming a magnesium silicate hydrate.

The mechanism of expansion is still a matter of debate. The occurrence of AFt crystals in cracks and voids within concrete suggests that it crystallizes from solution. In most concretes there will be sufficient void space to accommodate all the ettringite. Also, most expansion occurs after the ettringite has formed. It has been suggested that the ettringite first forms in a colloidal form and subsequently imbibes water and expands, although it is not clear why the water is not included in the structure immediately. A third hypothesis is based on the formation of ettringite leading to a reduction of the water content of the C-S-H, which then expands by absorbing water.

The reactions with sulfate can be reduced by the effect of reactions with other ions. Carbonation of the surface of the concrete converts the CH to relatively stable $CaCO_3$ and can reduce the permeability. Both carbo- and chloro-aluminates can be formed, reducing the degree of sulfate attack in concrete exposed to sea water. Protective layers of insoluble reaction products can form in some circumstances, e.g. brucite in sea water attck, reducing the long-term damage.

12.4.4 Thaumasite formation

Thaumasite has the composition $Ca_3Si(OH)_6(SO_4)(CO_3)\cdot 12H_2O$ and is formed through a combination of sulfate attack and carbonation. While the sulfate is normally from an external source, the carbonate may be derived from the aggregate or cement constituents. It forms in wet conditions at temperatures below ~15°C. Although it does not contain alumina it is possible that ettringite is a necessary precursor. Ettringite probably acts as a nucleating agent since the crystal structures are similar. The attack on the C-S-H leads to complete destruction of the cement paste, producing an incoherent ‘mush’.

12.5 Test methods and results

12.5.1 Natural exposure tests

Tests of concretes subjected to natural exposure conditions require long periods to obtain useful data but are necessary to validate laboratory based tests. An important early example is the work of Miller at the US Department of Agriculture (Miller and Manson, 1933 and 1951) who immersed concrete cylinders in solutions in the laboratory and also in a lake whose water contained magnesium and sodium sulfates. Tests were carried out for up to 25 years. These were among the first tests to demonstrate the importance of cement C_3A content on the sulfate resistance. This was confirmed by later work by the US Bureau of Reclamation (Bellport, 1968; Tuthill, 1966) and led to the introduction of standards for sulfate resisting cements based on C_3A content limits.

In 1970 the Building Research Establishment (BRE) and British Cement Association (BCA) started an investigation in which concrete samples were buried in sulfate-containing soil in the grounds of Northwick Park Hospital. Three replicate plots were used, to be excavated at 5, 15 and 25 years. Cubes cast from the same concretes were stored in sulfate solutions at BRE. A report on the results after 15 years exposure was published in 1992 (Harrison, 1992).

The concretes tested at Northwick Park covered a range of cement types, w/c ratios, workabilities, precast and *in situ*, well compacted and poorly compacted. Tests on the buried samples included visual assessment, density, elastic modulus, ultrasonic pulse velocity, strength, water absorption, sulfate penetration, XRD. The cubes stored at BRE were tested for strength and loss of material from the corners. The ground water at Northwick Park contained ~0.3% SO_4, a mixture of magnesium and calcium, but the tests at BRE included sodium sulfate solutions.

The conclusions from the 15 year report were

1. Only the poorly compacted samples showed a significant loss of strength.
2. Sulfate had penetrated up to 35 mm.
3. Method of manufacture, curing conditions and mode of exposure were as important as the concrete mix variables.
4. Carbonation before exposure reduced the attack.
5. Concrete permeability was particularly important where the concrete is subject to a hydrostatic head.
6. Attack was variable, tending to be greatest at edges and corners.
7. Poor compaction increased sulfate ingress.
8. Autoclaving produced highly resistant concrete.
9. Testing of 100 mm cubes in 0.35% magnesium sulfate solution can be used to assess the sulfate resistance after three years.
10. More concentrated solutions should be used for sodium or potassium solutions.
11. ‘Fifteen years exposure of concretes to a sulfate soil has not proved long enough to correlate their performance with results of accelerated laboratory tests on cubes.’

12.5.2 Accelerated laboratory tests

It is much easier to carry out laboratory tests than natural exposure tests and there is a substantial body of literature on such tests. In addition to those tests used in the Northwick Park work, measurements of prism expansions during storage in sulfate solutions are also commonly made. Laboratory concretes are typically well compacted and cured and the information obtained relates to the influence of concrete mix variables and cement type. The results generally confirm the expectations from the consideration of the attack mechanisms. Low permeability, low reactive alumina content and low CH content tend to increase sulfate resistance. Specifications for sulfate resisting concrete therefore normally have limits on w/c and/or cement content and/or strength to ensure adequate concrete quality and low permeability, and require the use of sulfate resisting cement.

12.5.3 Testing cements

The definition of sulfate resisting cement in standards is usually prescriptive in terms of Portland cement chemistry and/or the type and content of other materials, such as fly ash, blast furnace slag, natural pozzolan, etc. However, performance tests are being introduced to allow testing of new cements and validation of the limits used in the prescriptive standards. These tests are generally based on measurment of the expansion of mortar prisms during storage in sulfate solutions.

In the US ASTM C 452 (ASTM, 1995) describes a method, which is referenced by C-150, Standard Specification for Portland Cement (ASTM, 1999). This method is based on the preparation of mortars with gypsum added to the cement to raise its SO_3 content to 7%. Expansion between 24 hours and 14 days is measured. Cements giving an expansion of less than 0.04% have high sulfate resistance, equivalent to the Type V cements with a C3A limit of 5%. However, this test method is not suitable for cements containing slag or pozzolans. For these cements the method in C1012 (ASTM, 1995) is referenced. In this test mortar prisms are stored in sulfate solution and expansion determined after 6 months and 1 year of exposure. To ensure that the mortar is sufficiently mature to give a realistic measure of its long-term performance the prisms are only placed in the sulfate solution after cubes cast from the same mortar have reached a specified strength. C595, Specification for Blended Hydraulic Cements (ASTM, 2000a) and C1157, Performance Specification for Hydraulic Cement (ASTM, 2000b), refer to this method and give limits of expansion at 6 months and 1 year for moderate and high levels of sulfate resistance. These levels relate to the prescriptive limits in C150, where Type II cements with $C_3A < 8\%$ are considered of moderate resistance and Type V cements with $C_3A < 5\%$ are considered of high resistance. The BS 4027 (BSI, 1996) limit of 3.5% is comparable with the Type V limit. Attempts to standardize a similar European method have been delayed by poor between laboratory reproducibility.

While few test methods have reached the status of standards, there are large quantities of data in the literature concerning the expansion of mortar bars stored in sulfate solutions. The dominant factor determining the sulfate resistance of Portland cements is the C_3A content. Low C_3A sulfate-resisting cements have been included in standards for many years (BS 4027 (BSI, 1996), ASTM C150 Types II and V (ASTM, 1999)). The ASTM standard includes limits on C_4AF and C_3S as well but these are not generally considered very significant.

The performance of blast furnace slag cements is mainly determined by the proportion of slag in the cement. High sulfate resistance is only observed at more than ~65% slag. The products of slag hydration are not inherently sulfate resistant. The good performance at high slag levels is due largely to the low permeability this provides. High Al_2O_3 slags are less resistant and, of course, the C_3A content of the clinker has an effect. This can lead to low levels of slag decreasing the resistance to sulfates, with a pessimum at 40–50% for a high Al_2O_3 slag and moderate C_3A clinker. A factor relevant to the comparison of continental European and UK experience is the sulfate content of the slag cement. Low sulfate contents reduce the sulfate resistance. The sulfate contents of factory produced cements in Europe are controlled at similar levels to those in Portland cements, ~3%. In the UK slag is normally blended with Portland cement in the concrete mixer. The sulfate contents of BS 6699 (BSI, 1992) slags are low – addition of sulfate is forbidden. The sulfate content of a 70% blend is therefore only ~1%, giving a significantly lower sulfate resistance than an equivalent slag cement with 3% sulfate.

Pozzolanic materials, such as siliceous and calcareous fly ashes, natural and industrial pozzolans, metakaolin and silica fume, tend to increase sulfate resistance by reducing permeability and reacting with CH. 20–40% of siliceous fly ash or pozzolan is usually necessary to give high sulfate resistance but this is only achieved if the concrete is sufficiently mature when the exposure begins. High lime ashes are less effective.

Portland limestone cements seem to have similar properties to the equivalent Portland cement (BRE, 1993). There is the possibility of reaction between the carbonate and aluminates to form carbo-aluminate hydrates but this does not appear to have a significant effect.

Calcium aluminate and supersulfated cements have high resistance to sulfate attack but there are restrictions on their use and/or their availability.

12.6 Specifying concrete for acid, soft water and sulfate exposures

12.6.1 Classifying exposure conditions, water, soil

The specification of concrete to resist any external attack is based on the assessment of the exposure conditions. For water-based attack the factors to be considered include

1. Mobility of the water – flow rate, soil permeability.
2. Chemistry of the water and soil – sulfate concentration, pH, aggressive CO_2.
3. Other attack mechanisms – freeze–thaw, wetting–drying, etc.

In the European concrete standard EN 206 (BSI, 2000) exposure to chemical attack from natural soils and ground water is classified into slightly, moderately or highly aggressive classes for each of SO_4^{2-}, pH, aggressive CO_2, NH_4^+ and Mg^{2+} in ground water or SO_4^{2-} and acidity in soil. The overall class is determined by the highest individual class, unless two or more characteristics give the same class, when the next higher class is used. These classes are based on essentially static water and temperatures between 5°C and 25°C. For mobile water, polluted ground water, or values of the characteristics outside the range covered by the defined classes a ‘special study may be needed’. For ground water and pH the classes are:

Exposure class	SO_4[g/1]	PH
Slightly aggressive	>=0.2 and <=0.6	<=6.5 and >=5.5
Moderately aggressive	>0.6 and <=3.0	<5.5 and >=4.5
Highly aggressive	>3.0 and <= 6.0	<4.5 and >=4.0

The ACI document ACI 201.2R-92, Guide to Durable Concrete (ACI, 1992) has only two classes covering a similar range. No classification is given for pH.

Exposure class	SO_4 [ppm]	SO_4 [g/1 approx]
Moderate	150–1500	0.15–1.5
Severe	1500–10 000	1.5–10
Very severe	Over 10 000	Over 10

In the UK BS 8500 Part 1 (BSI, 2002), the complementary standard to EN 206, requires a more detailed assessment of the aggressive environment. This is based on the BRE Special Digest 1 (SD1) (BRE, 2001) Part 1 and includes the effects of water mobility and polluted ground. Where oxidation of pyrite in the ground is possible the classification is based on the sulfate equivalent of the total sulfur content rather than the actual sulfate content. This follows the guidance given by the Thaumasite Expert Group (DETR, 1999) after an investigation of recent occurrences of the thaumasite form of sulfate attack in the UK. A total of 19 ‘Aggressive Chemical Environment for Concrete’ classes are defined based on five sulfate classes, defined below for ground water and total potential sulfate (for use when oxidation of sulfides may occur).

Sulfate class	SO_4	
	Ground water [g/1]	Total potential [%]
1	<0.4	<0.24
2	0.4 to 1.4	0.24 to 0.6
3	1.5 to 3.0	0.7 to 1.2
4	3.1 to 6.0	1.3 to 2.4
5	>6.0	>2.4

12.6.2 Concrete quality, cement types

When the exposure conditions have been classified EN 206 and ACI 201 give limits for concrete mix variables and allowed cement types. EN 206 only has an informative Annex which gives recommended concrete compositions for CEM I (Portland cement) only. The limits for chemically aggressive environments are given below. For cases where sulfate leads to moderately or highly aggressive exposure sulfate-resisting cement is recommended. Unfortunately there is no European specification for sulfate-resisting cement.

Environment class	Max w/c	Min cement content	Min strength class
Slightly aggressive	0.55	300	C30/37
Moderately aggressive	0.5	320	C30/37
Highly aggressive	0.45	360	C35/45

The table in ACI 201 has similar w/c requirements but does not include minimum cement contents or strength classes. Cements with moderate and high sulfate resistance are recommended for moderate and severe classes of exposure respectively. For very severe exposure a combination of very low C3A cement (Type V) with slag or pozzolan is recommended, where the slag or pozzolan has been shown by tests to improve the performance of Type V cement.

Environment class	Max w/c	Cement type
Moderate	0.5	Moderate sulfate resistance
Severe	0.45	High sulfate resistance
Very severe	0.45	C150 Type V + slag or pozzolan

For acid resistance ACI 201 simply recommends dense, low w/c concrete. It is noted that some pozzolans, particularly silica fume, can increase acid resistance. For a pH of 3 or lower a protective barrier is required.

In the UK BS 8500 is much more detailed and the guidance includes consideration of the performance level required of the structure or element and the section thickness. These are combined with the chemical environment class to give a Design Chemical (DC) class together with a number specifying the number of additional protective measures to be taken. 14 DC classes are defined with up to three additional protective measures (APM). APMs are only required where the water is classified as mobile. The APM options are

APM1 Enhanced concrete quality
APM2 Use of controlled permeability formwork
APM3 Surface protection
APM4 Sacrificial layer
APM5 Drainage of site

The concrete quality required for each DC class is specified not only by the cement type, cement content and w/c ratio but also by the aggregate carbonate range, which is based on the carbonate content of the coarse and fine fractions of the aggregate (A is high carbonate, B moderate and C is low). The cement types are split into three groups, essentially defining cements with sulfate resistance as

1. Low C_3A Portland cement to BS 4027.
2. Combinations of Portland cement with 70–85% blast furnace slag to BS 6699.
3. Combinations of Portland cement with 25–40% fly ash to BS 3892 Part 1.

Blast furnace and fly ash cements with equivalent compositional limits are also allowed.

12.6.3 DEF

Since DEF only occurs when cement has hydrated at temperatures above 70°C it can be avoided simply by keeping the concrete below this temperature. Temperatures in precast concrete can generally be kept below the critical temperature for DEF.

However, some cases where DEF damage may have been the cause of deterioration have been identified where the high temperature of the concrete was largely due to self-heating in large pours of high cement content cast during the summer and/or on top of a previous pour. Temperature rises are ~12°C/100 kg and for a 400 kg/m^3 concrete with an initial temperature of 30°C the peak temperature will be ~78°C. Measures taken to reduce this peak temperature will not only reduce the risk of damage from DEF but also, of course, from thermal shrinkage cracking.

The use of fly ash or slag to reduce temperature rises may also reduce the risk of DEF through their effects on the chemistry and microstructure of the cement paste. The limited data available indicate that they reduce DEF expansions even when cured at the same temperature as Portland cements. It may also be noted that low C_3A sulfate-resisting cements seem to have a lower risk of DEF. Advice on avoiding DEF in *in situ* concrete has recently been published by BRE (IP 11/01, BRE, 2001; BR 449, BRE, 2002).

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13

Alkali–aggregate reactivity

Ian Sims and Alan Poole

13.1 Introduction

13.1.1 Background

Apparently spontaneous and often serious long-term cracking of concrete structures was investigated in the USA in the 1930s and eventually found to be caused by an expansive reaction between cement and aggregate (Stanton, 1940a, b). It was found that the combination of cement with a relatively high content of alkali hydroxides and aggregates containing certain potentially reactive forms of silica reacted to form an alkali–silica gel, which could swell on the absorption of water, giving rise to expansive stresses. This initial discovery gave rise to the rapid development in the USA of various tests for aggregate reactivity and preventative controls primarily based on using cements with relatively low alkali contents.

Over more than sixty years since the discovery of alkali–aggregate reactivity (AAR), the complexity of the subject has become recognized and varying forms of occurrence have been identified in most regions of the World, including the UK (Figure 13.1). AAR has generated an intensity of research that has often outstripped both its incidence and typical structural significance, but, nevertheless, in some circumstances AAR can be an important potential threat to concrete durability. The risk of AAR damage in new construction can be minimized by recognition of the threat and the implementation of practical

Figure 13.1 ASR cracking and displacement in a highway bridge wing wall, UK.

preventative measures. Catastrophic failure caused by AAR is almost unknown and, contrary to earlier belief, structures damaged by AAR can usually be repaired or economically managed for an extended period.

This chapter provides an introduction to AAR and the concluding section provides references to more thorough general treatments, guidance on specific aspects and regional accounts. Those researching into AAR should refer to the proceedings for the series of international conferences on the topic, which started in 1974; the most recent (11th) conference was held in Québec, Canada (Bérubé *et al.*, 2000; Sims, 2000) and the 12th conference will take place in Beijing, China, in 2004.

13.1.2 Reaction types

AAR is a general term for a family of reactions within hardened concrete, not all of which are detrimental. Indeed, some chemical interaction at cement–aggregate interfaces can improve bond and, paradoxically, the effectiveness of some mineral additions is reliant upon a form of AAR during the hydration and setting phase. AAR arises as a result of an inappropriate combination of potentially incompatible constituents of concrete and should not be characterized as being caused by one or other of these components, each one of which is stable in isolation. For this reason, some have preferred the general term 'alkali-reactivity', to avoid over-emphasizing aggregate reactivity. However, even this is misleading, because the chemical reaction actually involves alkali *hydroxyl* ions, rather than 'alkalis' *per se.*

As originally found by Stanton in 1940, the most common form of AAR worldwide involves reactive silica in the aggregate and is termed alkali–silica reactivity (ASR). Apparently different types of ASR have been recognized around the World, although it seems that the variations in rates and effects relate only to detailed differences in the natural aggregate materials and localized conditions and practices, rather than any fundamental factors. Slow reactions involving a range of siliceous rock types in Canada, for example, were for a period characterized as exhibiting 'alkali–silicate reactivity' (Gillott *et al*., 1973), but these and other similar occurrences are now accepted as being a particular variant of ASR. The recognition that most cases of ASR share basic similarities has recently led to moves towards the development of international approaches to testing and specification (Sims and Nixon, 2003).

Another form of AAR involves carbonate aggregates and is generally known as alkali–carbonate reactivity (ACR). Pioneering work on ACR was carried out in Canada (Swenson, 1957; Gillott, 1963; 1964) and, recently, much work has been carried out in China. ACR remains a rare form of AAR and there is uncertainty over its expansive mechanism, which does not involve the formation of swelling gel. Recent research has found that some alleged cases of ACR are better explained as ASR, with yet further cases exhibiting evidence of both ASR and ACR occurring within the same concrete.

13.2 Reaction mechanisms

13.2.1 Alkali–silica reactivity

ASR is best envisaged as a two-stage process. Initially, alkali hydroxides, mainly from cement, react with susceptible siliceous components within aggregate particles, in the presence of moisture, to form an alkali–silica gel. Subsequently, this gel can absorb further moisture, swelling in the process and imposing expansive stresses on the enveloping concrete. In view of the relatively low tensile strength of the concrete matrix, these expanding reaction sites generate radiating microcracks (Figure 13.2). As the gel imbibes moisture, the viscosity is reduced and the reaction product becomes able to migrate into the microcracks it had previously induced (Figure 13.3). Commonly the reactive aggregate particles are distributed throughout the concrete, to a greater or lesser extent, so that the microcracking from individual reaction sites can become linked into a network of cracking that can impair the physical and mechanical properties of the concrete material. The generalized expansion of the concrete interior places the exterior surface of the concrete in tension, typically resulting in superficial map-cracking patterns that resemble those caused by shrinkage, though in structures these patterns are usually partially or completely modified by the local stress field within the concrete.

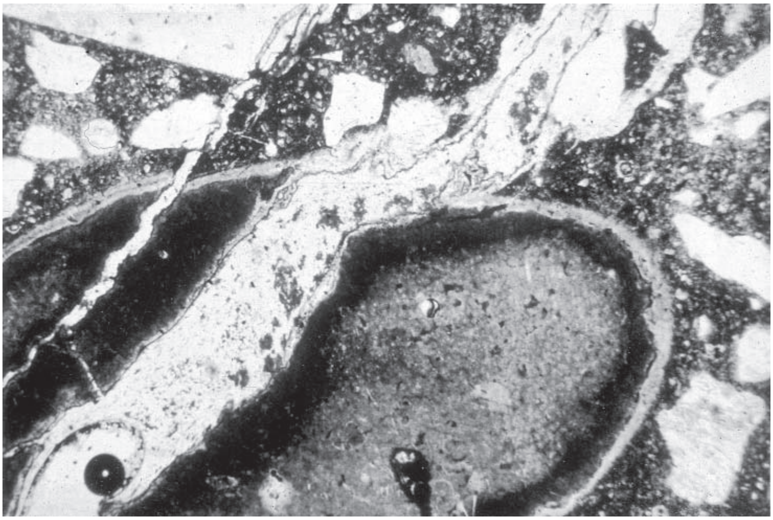


Figure 13.2 ASR site under a microscope in thin section: expanding chert sand particle. View about 5 mm across.

Figure 13.3 ASR gel deposit in a crack remote from the reaction site, under a microscope in thin section. View about 5 mm across.

ASR in concrete, and especialy ASR leading to damage, is rarely inevitable and depends on the coexistence in a given circumstance of a number of conducive factors. Three crucial factors are (i) sufficient moisture to initiate reaction and then promote swelling of the gel, (ii) sufficiently high alkali hydroxide concentrations to trigger and sustain reaction, and (iii) the presence of potentially reactive silica in a critical proportion. Some other factors can influence the likelihood, rate and severity of reaction and/or resultant damage.

Moisture

In most cases, the mix water provides sufficient excess moisture to initiate any reaction. However, it has generally been considered that significant damage will only occur in conditions sufficiently humid to facilitate swelling of the gel reaction product. Any relative humidity in excess of about 75% is regarded as conducive to ASR damage, which, in practice, means that any exterior concrete in a temperate or other wet climate is at risk, if the concrete contains a reactive mix. Early experience in desert regions of the USA demonstrated that even periodic flash flooding could provide adequate moisture to promote expansion and cracking of affected concrete structures. Recent research by Larive *et al.* (2000) has indicated the crucial role of water content in the expansivity caused by ASR in concrete.

Alkalis

It is a convention in cement and concrete chemistry to assess alkalis on the basis of acid-soluble 'equivalent soda' (Na_2Oeq), which is expressed as $Na_2O + 0.658K_2O$. Portland cement always contains soluble alkali salts that enter the pore solution very early in the hydration process. It is these alkali hydroxides in the concrete pore solution that are available to participate in ASR. Portland cement is thus the predominant, and frequently the only significant, source of alkalis in cases of ASR. Early experience demonstrated that usually only high-alkali cements (say $Na_2Oeq > 1.0\%$ by mass) were involved in damaging ASR, leading to pragmatic control measures in the USA and elsewhere based on using 'low-alkali' cement, defined as <0.60% by mass.

Reactivity of a particular mix is actually dependent on the alkali content of the concrete, rather than just that of the cement component. Research (Hobbs, 1980) has indicated that expansion does not usually occur, even with a potentially reactive aggregate combination and in the presence of moisture, unless the concrete alkali content exceeds 3.0 kg/m^3 (Figure 13.4) and this has been used in the UK and elsewhere as a practical basis for preventing ASR damage. The threshold indicated by cement alkali contents <0.60% and concrete alkali contents <3.0 kg/m^3 is probably related to the chemical composition of the resultant gel product, as gels with low alkali–silica ratios are found to exhibit less swelling capacity.

The concrete alkali content is obviously influenced by the cement content, as well as the inherent alkali content of the cement itself. Moreover, other concrete constituents can contribute reactive alkalis to the pore solution, sometimes significantly; these include aggregates, fillers, additions, admixtures and even the mix water. The common additions, fly-ash (usually pulverized-fuel ash or pfa in the UK) and ground granulated blastfurnace slag (ggbs) typically make contributions to the reactive alkali content, though these are much less than Portland cement and the materials may also have compensating effects (see later). In the UK, it has become routine to accept that about 25% and 50% of the acid-soluble alkali contents of pfa and ggbs, respectively, should be regarded as reactive. Thus, cement replacement by pfa or ggbs has the effect of diluting the reactive alkali content of the concrete and this can be used as a controlling measure. Methods for the determination of alkalis in concrete constitutents are identified in Hawkins (1999).

Most aggregates (and fillers) make little or no contribution to the reactive alkali content of concrete, but there are exceptons. It is known that aggregates with a sodium chloride content can be associated with an increase in the sodium hydroxyl concentration within the concrete pore solution. This obviously may arise with marine aggregates, but

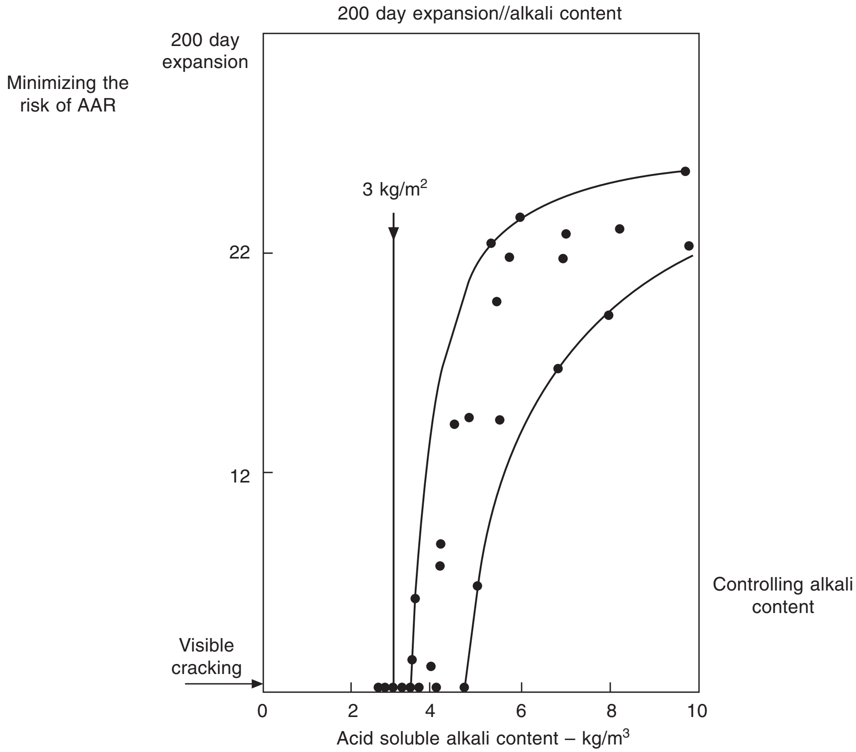


Figure 13.4 Relationship between expansion and concrete alkali content. From Hobbs (1980).

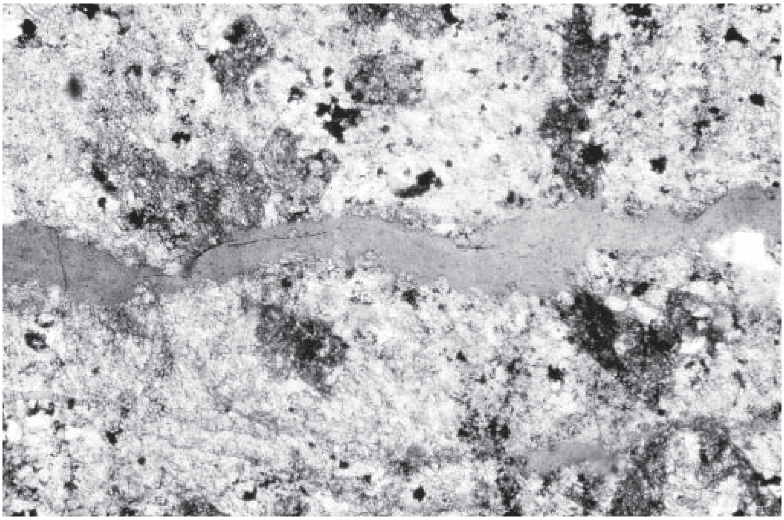
modern de-watering and washing processes usually minimize any effects. Some aggregates comprise or contain rock and mineral constituents that can release some alkalis within concrete (Bérubé *et al.*, 2002) and assessment of this possibility is necessary when conducting an alkali audit for a particular concrete mix. The nature of the reactive aggregate also affects the alkali threshold for initiation of reaction. Although the value of 3.0 kg/m^3 seems to apply universally, it is found to be overcautious in some cases and, for example, current UK guidelines make some allowance for aggregate type in determining the most appropriate concrete alkali content limit.

Alkali content of the original concrete mix appears to have the dominant influence on the likelihood of damaging ASR. However, additional alkalis can be acquired extraneously and, in some cases, these seem to exacerbate reactivity within concrete. De-icing salt is reported to have worsened ASR in concrete roads and related structures in Scandinavia and Canada, for example, and salt accumulation was identified by Sibbick and West (1989) to be a probable factor in the case of ASR in some UK concrete road pavements.

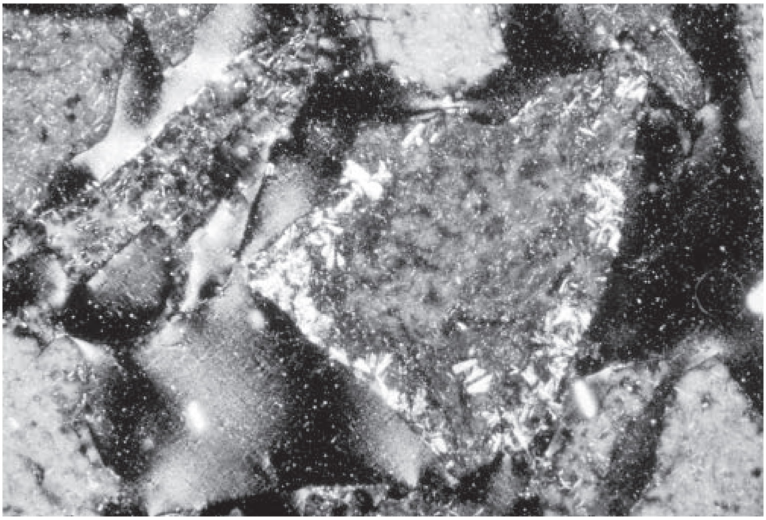
Aggregates

From a purely chemical standpoint, ASR is not surprising, as any silica is reactive with, and ultimately soluble in, highly alkaline (caustic) solutions. However, most forms of silica are essentially stable within the normal range of internal concrete environments, under normal atmospheric conditions and for the foreseeable service life of any structure.

Silica reactivity seems to depend upon high internal surface area and/or some degree of atomic disorder; the entropy of a partially or poorly crystalline fine-grained silica may be greater than that of a purely amorphous material. The most reactive natural silica is opal (or opaline silica), which is fairly common worldwide (in non-gem quality forms) and is a hydrous and virtually non-crystalline or highly disordered form of silica related to cristobalite. Similarly, silica-rich volcanic glasses, or especially their partially or completely devitrified equivalents, can be very reactive. Various metastable and/or fine-grained forms of silica have been found to be reactive, including tridymite, cristobalte, chalcedony and cryptocrystalline or microcrystalline quartz (Figure 13.5). In the UK and other areas of

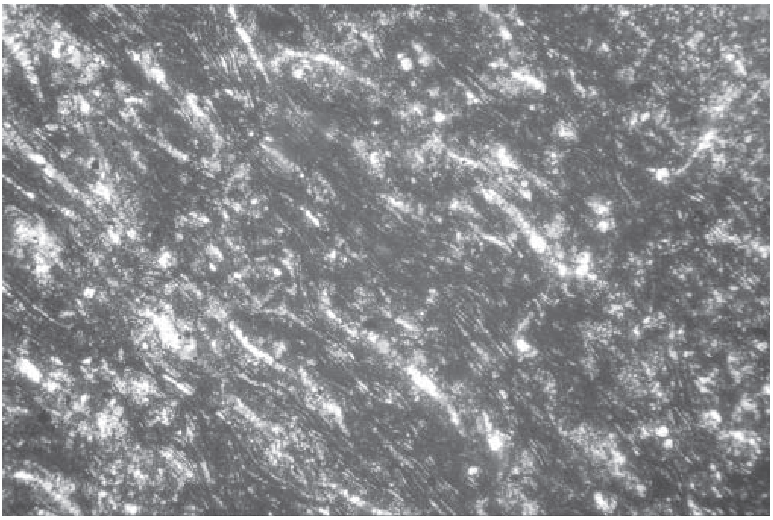


(a)

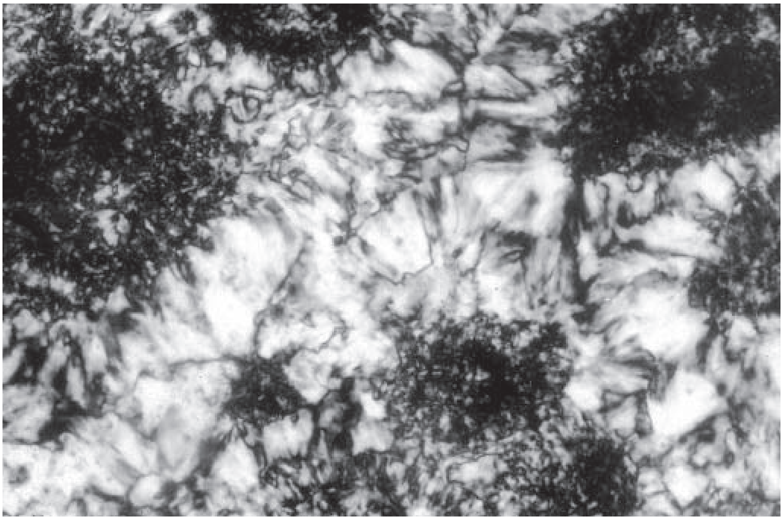


(b)

Figure 13.5 Some reactive forms of silica under the microscope: (a) opal vein in limestone, (b) tridymite crystals bordering a cristobalite particle.



(c)



(d)

Figure 13.5 (*continued*) (c) volcanic glass in rhyolite (the black groundmass is glass) and (d) recrystallized quartz along the grain boundaries in a meta-quartzite.

Northern Europe, for example, the most common reactive constituent is a component of a rock termed ‘flint’ (a Cretaceous form of chert), which comprises cryptocrystalline and microcrystalline quartz with inclusions of chalcedony. The most reactive aggregate encountered in the UK has been a crushed greywacke, in which parts of the argillaceous rock matrix had become recrystallized to microcrystalline silica.

In the past, strained quartz was thought to be a reactive form of silica and it was common for the degree of straining to be measured microscopically (Dolar-Mantuani, 1983). However, it is now accepted that quartz straining *per se* is not a reliable guide to reactivity, although its presence within some rocks can indicate a degree of alteration that will have generated potentially reactive microsilica along the quartz grain boundaries (Smith and Dunham, 1992).

These reactive forms of silica typically form part of a host rock, in which they might represent either a major or only minor component. In turn, the host rock might form either all or only some of the particles within a concreting aggregate. In these ways, the form(s) and content(s) of any reactive silica within aggregates can vary greatly. Early research into ASR discovered that there was no simple relationship between the proportion of potentially reactive constituents in the aggregate and the resultant magnitude of reaction and expansion. Indeed, in many cases, the maximum expansion occurred at a relatively low proportion of reactive material in the aggregate; this critical proportion has been termed the 'pessimum'. A wide range of pessimum values has been identified for different aggregate types (Figure 13.6), for example, ranging from just a few per cent for opal, through maybe 15 to 35% for flint and up to 100% (i.e. no pessimum) for some crushed glassy andesites.

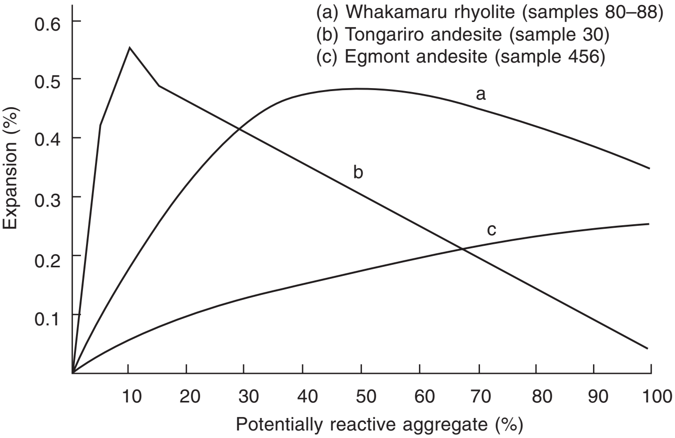


Figure 13.6 'Pessimum' curves for ASR expansion caused by various aggregates in New Zealand. After Barnard (1991).

Other factors

Other factors controlling the severity of ASR tend to relate to the composition and properties of the gel product. As with many other chemical reactions, the rate of ASR increases with rising temperature, but the resultant expansion does not necessarily increase accordingly. This might be explained by the changing character of the gel reaction product with increasing temperature. However, the accelerating effect of elevated temperature has been used to create practicable tests for ASR potential, when the expansions recorded for test specimens must always be correlated with field experience and not be taken as absolute values.

The most expansive alkali–silica gels seem to require a certain minimum alkali–silica ratio and also, critically, the presence of some calcium, which is taken to be derived from the calcium hydroxide that saturates the pore solution in any concrete. Some researchers have thus argued that calcium hydroxide should be regarded as a fourth critical factor required for damaging ASR (Chatterji, 1979). It is thought that the effectiveness of some additions in preventing ASR damage is at least partially explained by the consumption of calcium hydroxide in pozzolanic reactions that occur during the initial hydration and hardening phase. Another modification to the expansivity of gel is achieved by the introduction of lithium, which was discovered quite early in the research into ASR, but has only recently been revived as a practical alternative preventative measure (Hooper *et al*., 2002).

13.2.2 Alkali–carbonate reactivity

ACR is quite different from ASR, though the effects can be similar (Figure 13.7). All the early work on ACR was carried out on the argillaceous dolomitic limestone from a source near Kingston, Ontario, in Canada. This rock has a distinctive microtexture of relatively coarse rhombic dolomite crystals in a groundmass of micritic calcite and some clay minerals and, for a while, this was thought to be a composition and texture characteristic of susceptibility to ACR. However, other reactive carbonate compositions and textures have now been described, not all of which are expansive. Researchers have struggled to explain the mechanism for ACR expansion and most hypotheses highlight the possible role of de-dolomitization; some have even concluded that de-dolomitization is alone sufficient to explain the expansion (Deng and Tang, 1993). There remains some doubt as to the cause of this rare form of AAR, with careful petrographical investigation sometimes identifying evidence of ASR within carbonate materials that had previously been associated with ACR (Katayama, 1992). However, Chinese research seems to show that certain carbonate textures are vulnerable to ACR, while some siliceous carbonate rocks may be associated with both ASR and ACR in the same concrete.

Figure 13.7 ACR in a pavement (sidewalk) and kerb in Ontario, Canada.

13.3 Effects of AAR

13.3.1 Concrete properties

Although expansion and cracking of concrete are the outward signs of damaging ASR, it is reasonable to suppose that the disruption caused by expanding reaction sites and the consequential development of a microcrack network will adversely affect the physical and mechanical properties of affected material. Research has produced somewhat variable and contradictory findings in this respect, partly because of the difficulty in simulating ASR within a laboratory timescale, so that properties can be compared before and after expansion has occurred. Useful work was carried out by Swamy and Al-Asali (1986, 1988) using fused silica synthetic aggregate. Some data are collated in Table 13.1 for illustrative purposes, wherein percentage reductions in mechanical properties are shown for ASR expansions in the range from 0.05 to 1.50%. These data show up to 40% reduction in compressive strength as ASR expansion progresses, but not all experiments have found compressive strength to be significantly affected. Research has more consistently found tensile strength, flexural strength and especially modulus of elasticity to be affected by ASR expansion. The data in Table 13.1 indicate that tensile strength is reduced by even the lowest expansion; direct tensile tests registered 55% reduction with just 0.05% expansion. Elastic modulus had been reduced progressively, from 30% reduction at 0.10% expansion, up to 80% reduction at 1.50%.

13.3.2 Structures

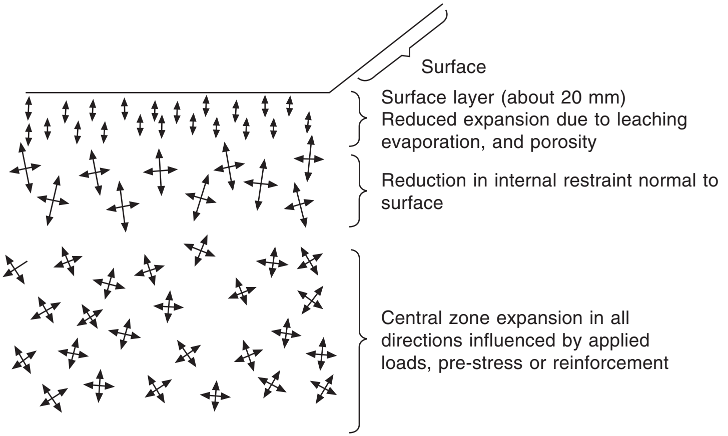
ASR in plain concrete samples typically induces measurable expansion and resultant surface cracking in a characteristic ‘map’ pattern. Similar effects on a larger scale have been observed for mass concrete and other non-reinforced or only lightly reinforced concrete structures. ASR in some mass concrete dams and hydro-electric barrages, for example, has caused sufficient expansion and deformation to jam turbines, flood gates and other fittings, sometimes necessitating constant palliative activity. ASR frequently does not occur within the outermost zone of concrete, apparently because weathering

Table 13.1 Effect of ASR on some concrete properties. Compiled from several sources, including Doran (1992) and Sims (1992a)

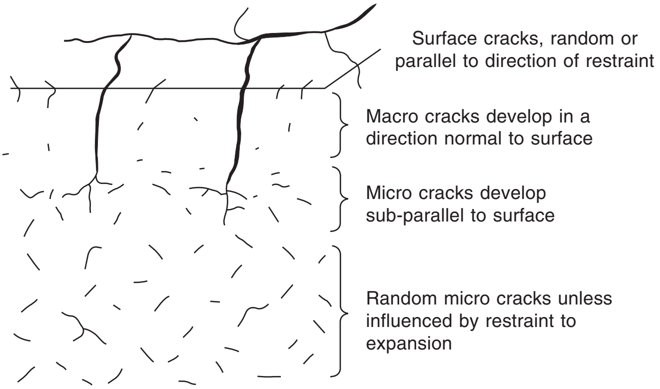
	Percentage reduction for expansion (mm/m) of					
Property	0.5	1.0	2.5	5.0	10.0	15.0
Cube compression	0	15	20	25	30	40
Uniaxial compression	0	0	•	•	•	•
Flexural tension	20	35	55	65	•	•
Indirect tension	15	20	40	55	•	•
Direct tension†	55	65	70	75	80	85
Gas pressure tension	70	75	•	•	•	•
Elastic modulus	0	30	50	65	70	80

•Data not available
†Data for mortar

leaches out the alkalis, so that ASR expansion within the heart of the concrete imposes tensile stress on the non-reacting surface material, leading to the characteristic map-cracking (Figure 13.8).



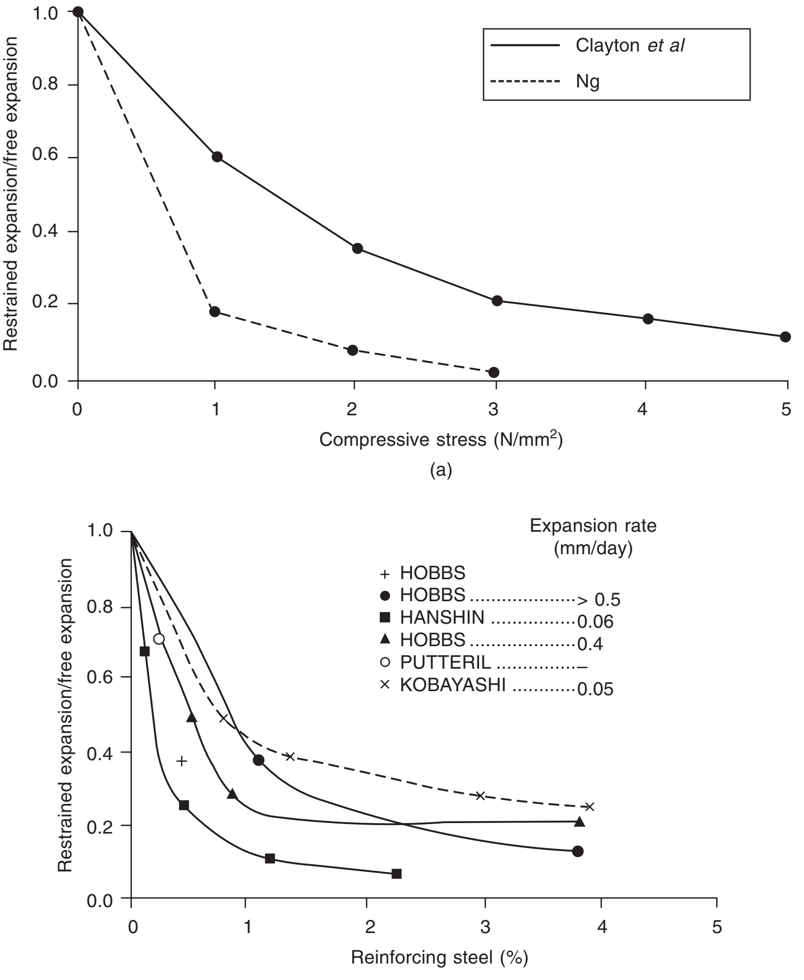
(a) Expansion



(b) Cracking

Figure 13.8 Diagrams illustrating the development of cracking caused by ASR. From Courtier (1990).

However, the effects of ASR on structural concrete restrained by loading or reinforcement are often much less significant. Figure 13.9 shows the results of research that has demonstrated that ASR expansion can be restrained by comparatively low levels of compressive stress (Figure 13.9(a)) or steel reinforcement (Figure 13.9(b)). In a series of experiments in different countries, just 2 or 3% reinforcement was sufficient to reduce ASR expansion by around 75%. This explains why some profoundly surface-cracked structures are nevertheless found to exhibit almost unimpaired load-bearing capacity. As a result of these findings, mainly during the 1980s, the previous view that ASR was an irreversible and progressive ‘disease’ of concrete structures (one journalist in the UK created the term ‘concrete cancer’) had to be modified. Guidance on structural appraisal prepared by the UK Institution of Structural Engineers (ISE) indicated that, depending on the degree of expansion, exposure conditions and the nature of the reinforcement, affected structures could usually remain in service, subject to monitoring and/or remedial action



(a)

(b)

Figure 13.9 Restraining effect of compressive stress (a) and steel reinforcement (b) on ASR expansion of concrete. From Doran (1992).

(Doran, 1992). den Uiji *et al*. (2000) have described experience with ASR-affected highway structures in the Netherlands, where inadequate reinforcement design has enabled the effects of ASR damage to threaten structural capability.

Early descriptions of concrete structures affected by ASR frequently referred to surface ‘pop-outs’ and gel exudations. However, these are not inevitable features of affected concrete and, in the UK for example, cases of ASR rarely exhibited pop-outs and extensive exudations of gel on to the surface. Sims (1996) has described a type of pop-out formation that is caused by localized ASR, associated with surface concentrations of alkali in contrast to the more typical depletion, which usually has no structural implications.

13.4 Cases of AAR

13.4.1 Europe and the UK

The earliest identification of ASR damage in Europe was in Denmark (Idorn, 1967), where many of the natural sand and gravel aggregates contained a porous opaline flint derived from the youngest Cretaceous strata (the Danian) that were not deposited in many other areas of Northern Europe including the UK. Tests were developed in Denmark and Northern Germany to identify this reactive porous and thus relatively low-density flint material. The adjacent Schleswig-Holstein region of Northern Germany also experienced ASR problems with local aggregates containing an opaline sandstone and, again, a specialized test was developed, based on alkali-dissolution of the opal component. Later, during the 1970s and 1980s, most Northern European countries, including the UK, identified cases of ASR damage relating to dense flint and other types of chert present in aggregates in pessimum proportions. A siliceous variant of Carboniferous limestone in Northern Europe proved to be highly reactive and has recently been adopted by RILEM as a 'reference reactive aggregate' (Sims and Nixon, 2003). In Norway, ASR damage has been attributed to granite aggregates containing reactive forms of silica. Iceland, which makes extensive use of concrete for domestic housing and other building work in the absence of alternative materials, experienced widespread ASR difficulties with reactive glassy basalt aggregates in the 1970s. ASR experiences in Denmark, Iceland and the UK are described in separate chapters of the book edited by Swamy (1992).

In this book, Sims (1992a) gives a detailed account of the whole process of ASR recognition and response in the UK up to that time. Research at the Building Research Station (now the BRE) in the 1950s had signalled the potential ASR problems with flint-bearing aggregates, but these findings were misinterpreted by the UK concreting industry and there was complacency about ASR until the mid-1970s. The first case of ASR in the British Isles, however, did not involve flint and was discovered in Jersey, one of the Channel Islands, where the Val de la Mare Dam (Figure 13.10), completed in 1961, was found ten years later to be exhibiting cracking and structural displacement (Cole and Horswill, 1988). From about 1976, indisputable cases of ASR started to be identified on the UK mainland, initially in South-west England, where first some buried concrete bases (Figure 13.11) and then a multi-storey car park (Figure 13.12) were found to be damaged by ASR in Plymouth. Over the next few years, many further highway structures and concrete buildings were identified as cases of ASR throughout South-west England, the English Midlands and occasionally elsewhere. A survey by county councils gave some indication of the incidence in different regions (Figure 13.13). Nearly all of these cases on the UK mainland involved flint and other forms of chert that were present as a minor 'pessimum' proportion within the aggregate combination. Other factors variously exacerbated the occurrences, including the use of high-alkali cements in some areas, exceptionally high cement contents, the absence of water-proofing membranes from many bridge decks and the increasing use of de-icing salts on the nation's roads.

Investigation of ASR-affected structures has become routine in the UK since 1992 and concern over the structural implications has largely evaporated in the light of the findings described in the previous section. Moreover, publication in 1983 of national guidance for minimizing the risk of ASR damage (Hawkins, 1983) and the various subsequent revisions

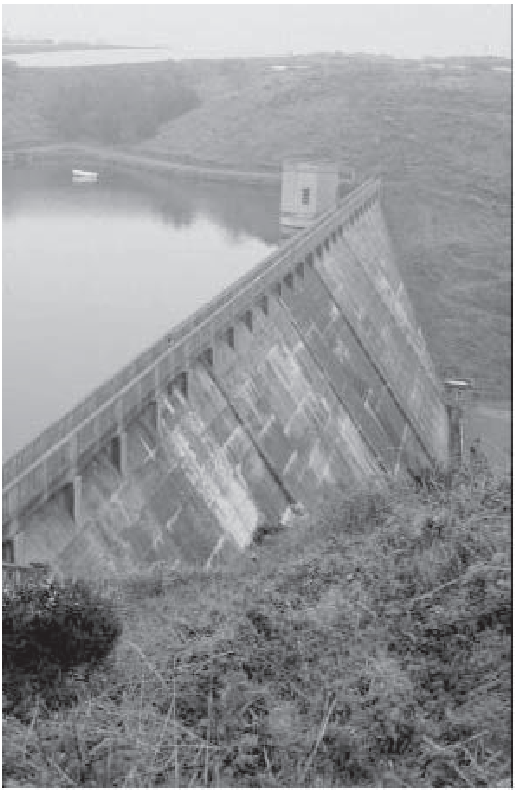


Figure 13.10 Val de la Mare dam, Jersey, Channel Islands; first identified case of ASR in the British Isles.

Figure 13.11 Concrete bases in Plymouth, Devon: first identified case of ASR in the UK.

Figure 13.12 Multi-storey car park in Plymouth, Devon: first major structure identified with ASR in the UK.

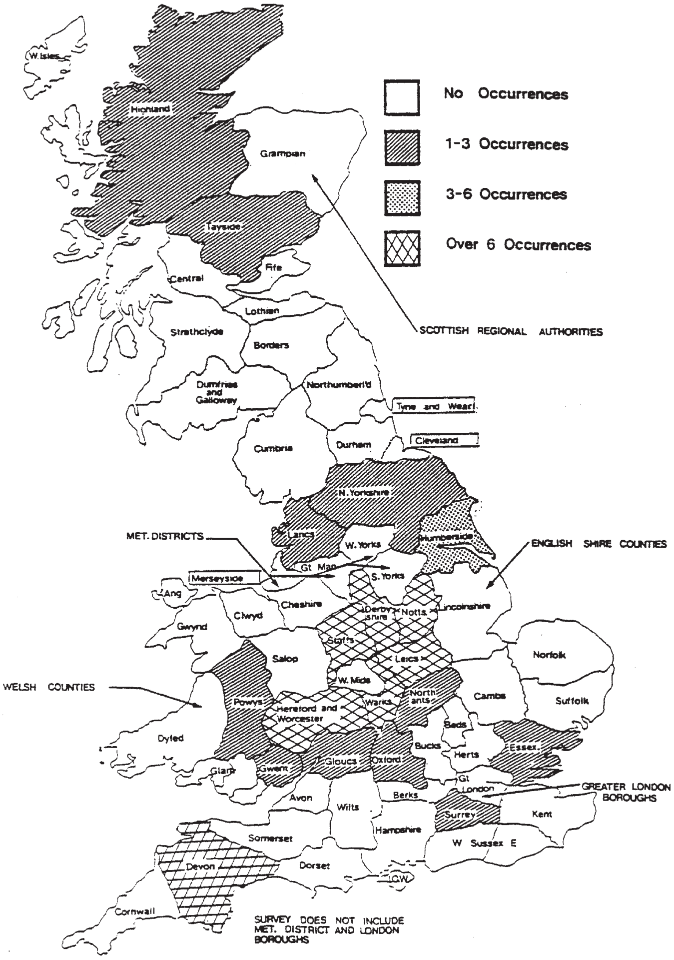


Figure 13.13 Regional incidence of ASR in the UK. From Leadbeater (1990).

and improvements to this guidance (Hawkins, 1999) seem largely to have been effective in arresting the occurrence of ASR in new construction. However, some crushed greywacke aggregates have now been identified as being among the most reactive in the UK and a few affected structures have been investigated in South-west England, Wales (Figure 13.14), Southern Scotland and Northern Ireland (Blackwell and Pettifer, 1992; Blackwell *et al.*, 1996).

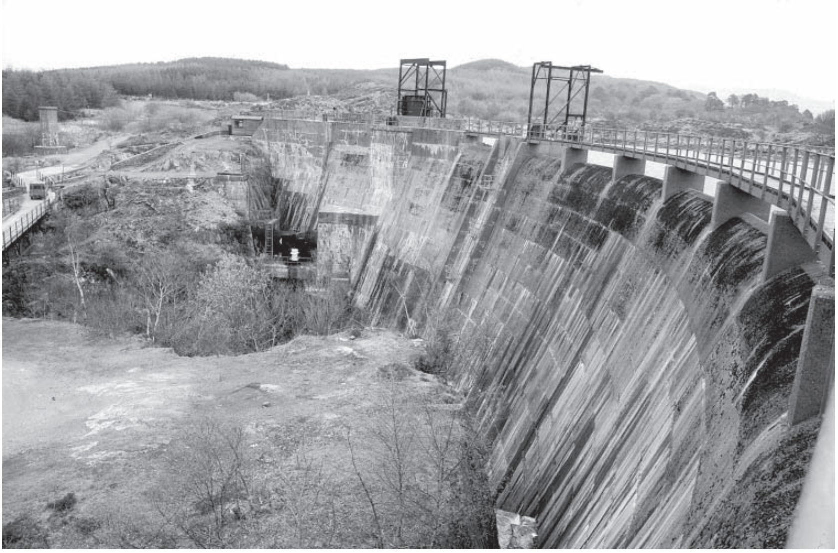


Figure 13.14 Maentwrog Dam in Wales: a case of ASR damage caused by greywacke aggregate.

13.4.2 Outside Europe

AAR has now been identified in all continents and most countries. Most of the early investigative and research work was undertaken in the USA and a useful modern review was prepared as part of the Strategic Highway Research Program (Helmuth *et al.*, 1994). As might be expected for a country of its huge geographical size, Canada has experienced a wide range of problems with both ASR and ACR, with the resultant damage often being confused and/or exacerbated by other durability threats, including freeze–thaw action and the effects of de-icing chemicals. Grattan-Bellew (1992) gave an account of the Canadian experience in Swamy's book and a recent detailed review was presented, on a province by province basis, in a special edition of the *Canadian Journal of Civil Engineering* (Isaacson and Mavinic, 2000).

Other major countries of the World in which significant incidence of AAR has been reported include Australia, Brazil, China, India, Japan and South Africa. In Australia, cases have been reported in Western Australia and Queensland (Carse, 1989), including a long marine structure with an aggregate containing reactive meta-quartzite and chert (Carse, 1996). A great deal of research has taken place in China in recent years, especially regarding reactive carbonate aggregates, and the next international conference on AAR is due to be held in Beijing in 2004. Mullick (1992) provides an account of AAR experience in India in Swamy's book. Extensive ASR problems have been encountered in Japan, particularly involving an aggregate produced from a type of andesite containing volcanic glass and devitrified glass. Coverage of the Japanese experience with ASR, including a great deal of work into structural effects and methods of rehabilitation, may be found in the proceedings of the Kyoto international conference (Okada *et al*., 1989) and in Swamy's book (Nishibayashi, 1992). Widespread occurrences of AAR were identified in South Africa in the 1970s and 1980s, especially in Cape Province, where a reactive hornfels (the Malmesbury Shale) was frequently used in conjunction with a high-alkali cement (Oberholster, 1981, 2001).

13.5 Diagnosis and prognosis

13.5.1 Inspection and monitoring

It is important for the assessment of any cause of concrete deterioration to be dependable. It should not be based on assumptions drawn from inconclusive evidence, as the performance of materials in new construction (Sims, 2002), the reliability of tests and the approach to the mitigation of the effects on structures will all be based upon the diagnosis reached. This is especially the case with AAR, which shares some characteristics with other mechanisms and may occur in both harmless and damaging forms. Moreover, it is common for AAR to occur in combination with one or more other types of deterioration, so that judging the primary causation of any damage can be a challenge. Detailed guidance on the diagnosis of ASR is given by Palmer (1992) and a practical scheme (Table 13.2) was suggested by Sims (1992b). It is particularly important to recognize that identification of ASR is not sufficient; it is additionally necessary to establish whether or not that ASR has caused damage.

Diagnosis of any concrete deterioration or damage must start on site, preferably involving the materials scientist who will later conduct the laboratory investigation. Site inspection will include the identification of any evidence of deformation or displacement of concrete units and the recording of crack dispositions, patterns and intensities. It is important to relate cracking patterns variously to structural geometry and/or design, apparent concreting sequence, localized detailing (especially where cracking may be coincident with water leakage) and both environmental and in-service conditions. In complicated cases, it is helpful to undertake crack mapping of the main structural units. Superficial cracking patterns can often be reminiscent of ASR, but it is important to be aware that reliable diagnosis can never be adequately based upon the appearance of surface cracking alone. Other features that might be significant to ASR diagnosis include surface discolorations, exudations and sometimes pop-outs. Techniques for identifying gel on site have been

Table 13.2 Scheme for assessment of ASR in concrete. After Sims (1992b)

ASR might be judged to be the probable primary cause of evident damage in concrete if *each* of the following is satisfied:

- Site damage characteristic and/or consistent with ASR
- Exposure conditions conducive to ASR
- Significant ASR identified within the concrete
- Link between reactions sites and microcrack pattern
- No evidence of other causes

developed, but investigators should note that the presence of gel can only confirm that some ASR has probably occurred, not that any damage has resulted. In some cases and when feasible, it is desirable to monitor affected structures over a period, variously using comparative visual and strain measurement techniques, to establish whether any movement is occurring outside the normal range.

13.5.2 Sampling

Following the site inspection, samples must be taken for laboratory investigation. Drilled cores from the concrete are usually considered to be the most appropriate, though sometimes these might be augmented by samples of any surface deposits. Any investigation is only as reliable as the samples are representative of the matter being investigated, so that the critical importance of sample selection and sampling practice cannot be over-stressed. Palmer (1992) gives helpful guidance on sampling. ASR and any resultant damage are never uniformly distributed throughout an affected concrete structure and are frequently restricted to, or concentrated within, particularly areas or units, according to factors including concrete composition, the degree of loading or restraint from reinforcement and local environmental conditions. Sampling of a given structure is therefore aimed at finding and representing the materials most likely to exhibit evidence of damaging ASR, on the basis of the site observations; the sampling is thus selective rather than random. It is good scientific practice also to take comparison samples from areas judged to be least at risk, or where the concrete materials appear to be performing differently in otherwise similar conditions.

Unlike core sampling for strength testing purposes, it is imperative that samples taken for ASR investigation are not permitted to dry out between drilling and laboratory receipt. Drying would destroy any evidence of active gel product on the side surfaces of the core and, after a period of drying in a normal atmosphere, any gel product might also become carbonated and thus lose its swelling properties. It is thus usual for the cores in an ASR investigation to be wrapped in Cling-film immediately upon extraction and then further wrapped in polythene bags for protection during transit to the laboratory.

13.5.3 Laboratory investigation

At some stage in the investigation of an affected site, a desk study should be conducted in order to establish all available details of the concrete constituents and mix(es) that were employed, also the environmental and in-service conditions to which the structure has been exposed. Subsequent structural appraisal (see next section) will also require knowledge of the design details and especially the amount and pattern of any steel reinforcement.

Central to the laboratory investigation will be petrographical examination of the core samples (St John *et al.*, 1998), which is a direct-observational technique. There are three stages to this petrographical assessment: (i) preliminary examination of the as-received (not yet dried) cores, to record any evidence suggestive of ASR, (ii) detailed microscopical examination in thin-section, definitively to identify the occurrence of any ASR and, crucially (iii) assessment of the ASR and the overall pattern of internal microcracking, to establish any causal relationship indicative of damaging ASR. Item (i) is a routine exercise that may or may not produce useful information. Item (ii) is a specialized procedure that should confirm or otherwise the presence of ASR, providing the samples and thin-section specimens are adequately representative of the affected concrete and the examination is undertaken by an experienced operative using good quality equipment. The key 'reliably diagnostic' features that will confirm ASR are sites of expansive reaction (Figure 13.15), in which reacting aggregate particles can be observed to be forming gel and inducing the propagation of microcracks, and gel deposits (Figure 13.16(a)), which may often be observed remotely from their original reaction site and now migrated into open cracks or voids, or even permeating into the cement paste. Some gel deposits may be layered (Figure 13.16(b)), often interpreted as indicative of seasonal fluctuations in moisture condition, and/or carbonated into a crystalline state and thus probably stabilized, and/or associated with other secondary deposits such as ettringite.

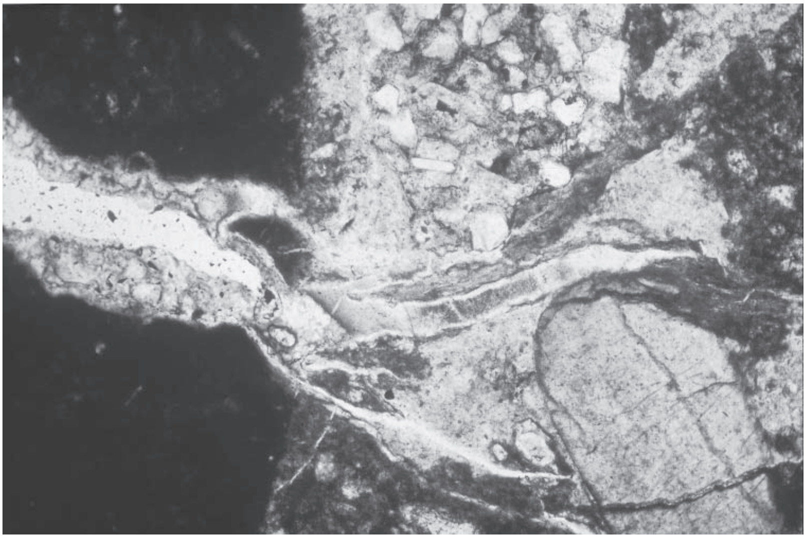
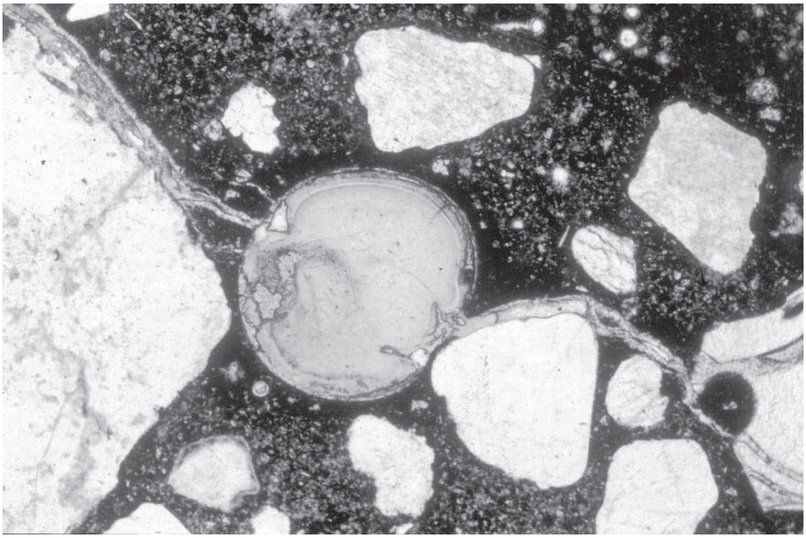
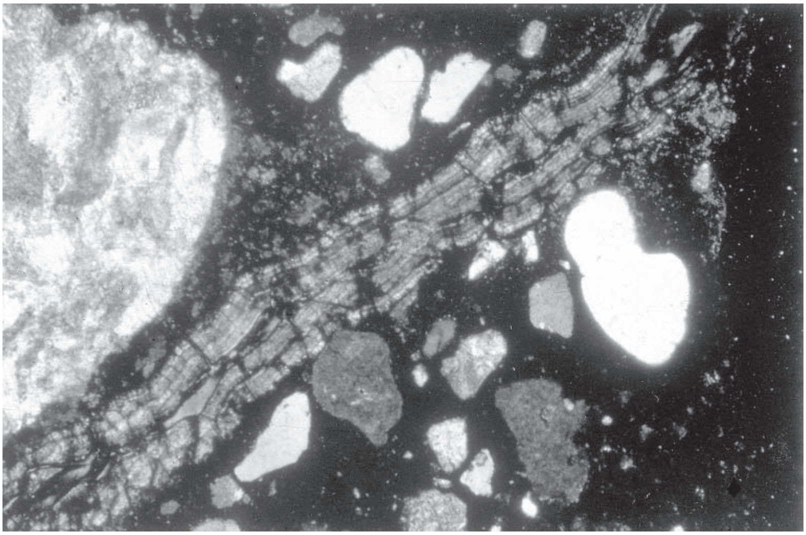


Figure 13.15 ASR site under a microscope in thin section: expanding chert sand particle. View about 5 mm across.

Item (iii), whereby the reaction sites and gel deposits are related to the pattern of internal cracking, is a difficult but critical stage in the investigation. This assessment can be carried out using large-area thin-sections and also the stereo-microscopic examination of even greater areas of sawn and finely ground concrete surface, such as the full length and diameter of a core sample. It is often helpful for the microcrack systems to be enhanced using fluorescent dyes. Internal cracking patterns symptomatic of damaging ASR are found to link the reaction sites, or at least the reacting particles and, in simple cases, to exhibit characteristic patterns, depending upon whether the reactive particles, are in the coarse or fine fractions of the aggregate (Figure 13.17). In the case of samples from actual structures, this ASR-related evidence is frequently complicated by interference from other mechanisms and features, so that careful interpretation will be required. Sims (1996) has described 'opportunistic' ASR, in which significant reaction has been facilitated by the effects of damage from another cause.



(a)



(b)

Figure 13.16 ASR gel deposits remote from the reaction site, under a microscope in thin section: (a) infilling void (view about 3 mm across) and (b) layered gel in a crack (view about 5 mm across).

Analysis and testing of the concrete can sometimes be applied to augment the petrographical examinations. However, conventional chemical analysis for cement content can be unreliable for ASR-affected concrete and determination of the original content of reactive alkali can be very difficult to achieve, owing to contamination in the analysis from aggregates and other sources that do not release alkalis under the normal conditions within concrete. Moreover, Wood *et al*., (1996) found that, owing to the mobility of alkalis and various depletion and concentration effects, reliable indications of the original

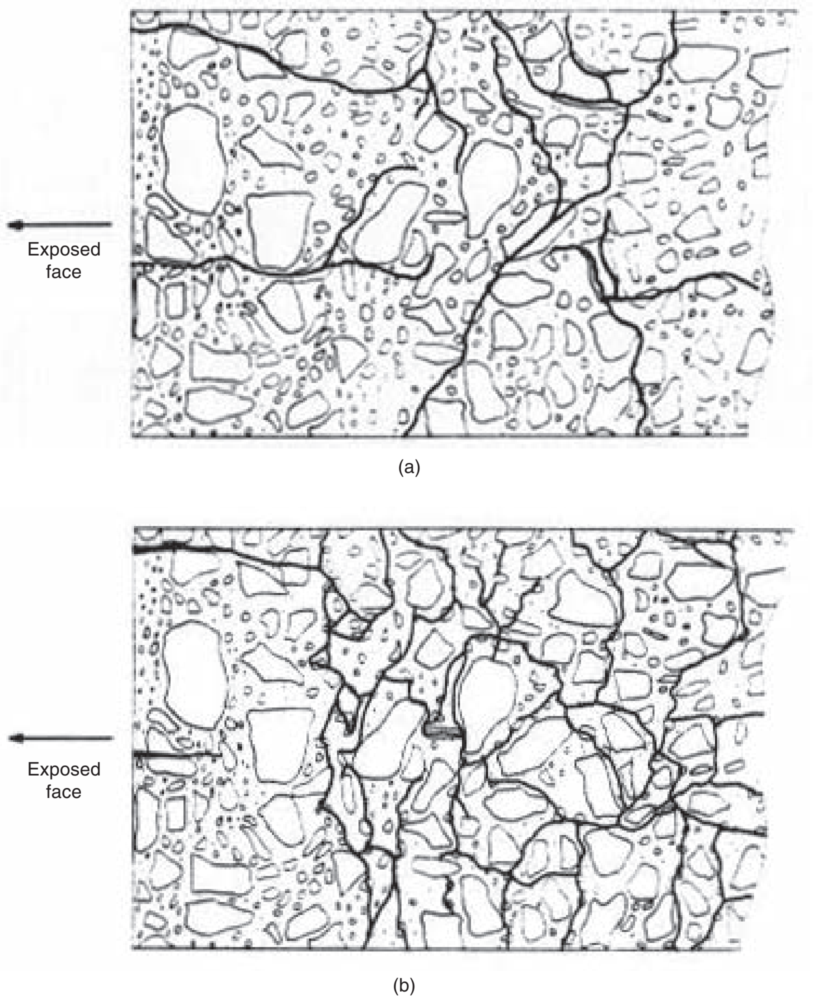


Figure 13.17 Diagrams illustrating typical ASR microcrack patterns: (a) reactive particles in coarse aggregate and (b) reactive particles in sand. From Palmer (1992).

alkali content in a given structure could only be obtained from the analysis of a large number of samples taken from the heart of the concrete. Palmer (1992) describes a core expansion test, for measuring the residual capacity within the concrete samples for further expansive reaction. This test may be of some assistance in estimating prognosis on a comparative basis, providing the cores are tested without any drying having occurred since drilling (Figure 13.18). However, it is important to recognize that the core specimens are completely unrestrained, so that the degree of any expansion measured in the test will be very much greater than any movements that would occur for the same concrete material within a stressed structure.

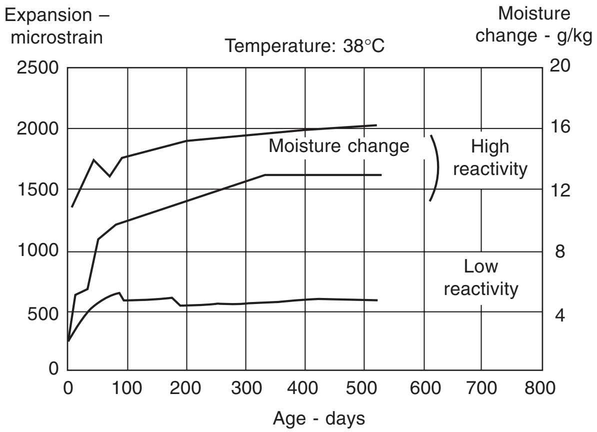


Figure 13.18 Core expansion testing for diagnosis: some typical curves. From Palmer (1988).

13.5.4 Structural appraisal

A standardized scheme for the structural appraisal of ASR-affected concrete structures has been devised by the ISE in the UK (Doran, 1992). Generally, this level of appraisal follows on from the basic investigation described in the previous section and allows the engineer responsible for the structure to make judgements over its residual structural capability and the need or otherwise for remedial action. Experience has shown that, in a majority of cases, ASR-affected structures can remain safely in service, sometimes merely being monitored in case of continued deterioration (Figure 13.19), at other times after a greater or lesser extent of repair work and, just occasionally, only after substantial

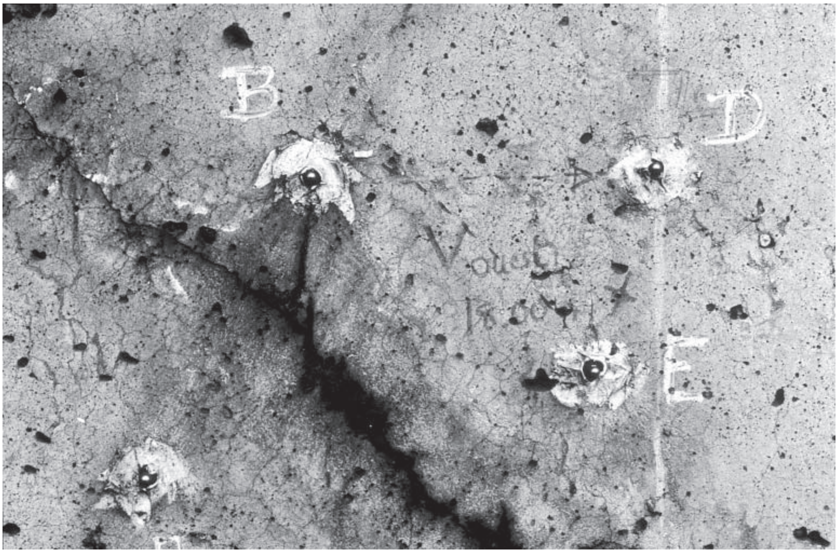


Figure 13.19 Monitoring ASR-cracked concrete using strain gauges.

reconstruction or the introduction of auxiliary support (Figure 13.20). On a World scale, only a very few structures have been dismantled over concerns for their stability following identification of damaging AAR.

Figure 13.20 Auxiliary support for pier and pile cap affected by ASR: Marsh Mills Viaduct, Plymouth.

The ISE scheme (Table 13.3) allows a 'structural severity rating' (SSR) to be allocated, either to a whole structure or to particular elements of a large structure. Five ratings are recognized, from 'negligible' (n), through 'mild' (D) and 'moderate' (C), to 'severe' (B) and 'very severe' (A). These ratings are determined from the integrated assessment of four factors: (i) site environment, (ii) reinforcement detailing class, (iii) expansion index and (iv) consequence of failure. Three categories of site environment are recognized and are broadly similar to those defined by Palmer (1992): 'dry' (or low risk) being permanently protected from wetting or always <85% relative humidity, 'intermediate' (or medium risk) being intermittently wetted and/or with humidity levels >85% and 'wet' (or high risk) being frequently wetted or continuously wet and/or liable to sodium chloride ingress. Three classes of reinforcement detailing are defined, separately for walls/slabs and columns, according to whether the cage is one-, two- or three-dimensional and also to the nature of the anchorage. A five-fold arbitrary scale of expansion indices (I to V) is used, being

Table 13.3 Scheme for assessment of ‘structural severity rating’. After Doran (1992)

Site environment	Reinforcement detailing class	Expansion index									
		I		II		III		IV		V	
		Consequence of failure									
		Slight	Significant	Slight	Significant	Slight	Significant	Slight	Significant	Slight	Significant
Dry	1	n	n	n	n	n	n	n	n	n	n
	2	n	n	n	n	n	n	n	n	n	n
	3	n	n	n	n	n	n	n	D	D	D
Intermediate	1	n	n	n	D	D	C	D	C	D	C
	2	n	n	D	C	D	C	C	C	C	B
	3	n	D	D	B	C	B	B	A	B	A
Wet	1	D	D	D	C	D	C	C	B	C	B
	2	D	D	C	B	C	B	B	B	B	A
	3	D	C	C	A	B	A	A	A	A	A

Structural severity rating, n = negligible, D = mild, C = moderate, B = severe, A = very severe

computed from determinations of ‘current expansion’, based upon crack width measurements on site, plus ‘potential additional expansion’ based upon core expansion testing (different from the diagnosis test) plus an allowance for long-term enhancement. Each resultant SSR is finally considered according to whether the consequence of failure is deemed ‘slight’ or ‘significant’.

Although the ISE scheme has merit in standardizing the approach to appraisal and decision-making, the outcome is dependent to a significant extent on establishing an ‘expansion index’, which is a highly imprecise exercise. Thus appraising engineers will also need to apply conventional structural judgements, sometimes including load tests and monitoring for deflections and other movements. Some guidance on ‘management procedures’ is given in Doran (1992), according to the SSR determined using the ISE scheme.

13.6 Minimizing risk and prevention

13.6.1 Schemes

As they have come to terms with the reality of ASR damage within their own country, each nation has tended to devise a prevention scheme that suits their own experience and is compatible with local materials and practices. The earliest controls were introduced in North America, where the approach was essentially to test the aggregate for AAR potential, in order to decide whether or not to specify a low-alkali cement (ASTM 1999). In the UK, as the number of confirmed cases of ASR multiplied, an independent working party of specialists was established in 1981 and their first recommendations were published two years later (Hawkins, 1983). Their approach was relatively sophisticated and was based on the elimination of at least one of the three factors deemed essential for damaging ASR: sufficient moisture, sufficient alkalis and a critical proportion of reactive aggregate. A menu of options was offered for meeting this objective, including four different ways for controlling reactive alkali content: using low-alkali cement, using ggbs, using pfa, or ensuring the overall alkali content of the concrete mix, from all possible internal sources, was ≤3.0 kg/m^3. This guidance was indicated to be primarily intended for 'normal concrete construction' and was not thought necessarily to be adequate for 'particularly vulnerable forms of construction'.

The UK's independent working party, chaired by Michael Hawkins, later became affiliated to the Concrete Society and the guidance for minimizing the risk of ASR damage has been substantively revised twice, as Technical Report No. 30 (Hawkins, 1999). The current (third) edition has increased the range of controlling options, by recognizing that both cement and aggregates exhibit varying degrees of reactivity (Figure 13.21). The original notion of 'low-alkali' cement is retained (≤0.60% Na_2Oeq), but a 'moderate' category (>0.60%, ≤0.75% Na_2Oeq) is introduced and 'high-alkali' thus becomes defined as >0.75% Na_2Oeq. Most UK aggregates in extensive use are recognized in TR30 as being *potentially* reactive and thus to be regarded as exhibiting 'normal' reactivity, to signal that such materials should not be stigmatized. Certain UK aggregate materials can be classified as having 'low' reactivity, for example including most crushed limestone and crushed granite materials. A few UK aggregates are classified as exhibiting 'high' reactivity, including recycled materials and those containing ≥10% crushed greywacke. Guidance on classifying UK aggregate combinations for ASR potential is provided in BS 7943 (BSI, 1999) and the classification scheme is predicated on the absence of any detectable opaline silica. Definitions for the 'normal' and 'low' reactivity classes, in terms of expansion in concrete prism tests, have been suggested (BRE, 1999).

An initiative is currently under way by to develop an international approach to minimizing the risk of AAR damage (Sims and Nixon, 2003). This ambitious scheme will need to embrace a full range of structure types, from small and relatively unimportant to large and prestigious constructions in which failure would be unacceptable. It will also be necessary to allow for various different climatic conditions, including those offering more hostile environments than the temperate climates of Northern Europe. The global scheme will also need to take into account differing local cement and aggregate materials, as well as the varying availabilities of additions and admixtures.

13.6.2 Aggregate assessment

Following the discovery of ASR in the USA, tests were devised by the American Society for Testing and Materials (ASTM) for identifying the types of reactive aggregate materials that had been encountered. In addition to petrographical examination, these methods included a rapid chemical procedure and an expansion test using mortar-bar specimens. After ACR had been recognized, a further test for the expansion of carbonate rock cylinder specimens was introduced. Guidance on the interpretation of results from these methods is given in an appendix to ASTM C33 (ASTM, 1999). However, these methods have not always been found to be effective with a wider range of materials: for example, in the UK, neither the rapid chemical test nor the mortar-bar test proved capable of detecting the reactive aggregate combinations arising from pessimum contents of flint. A RILEM technical committee, TC 106 (now TC 191-ARP), has been developing an integrated scheme for the international AAR assessment of aggregates (Nixon and Sims, 2000; Sims and Nixon, 2001).

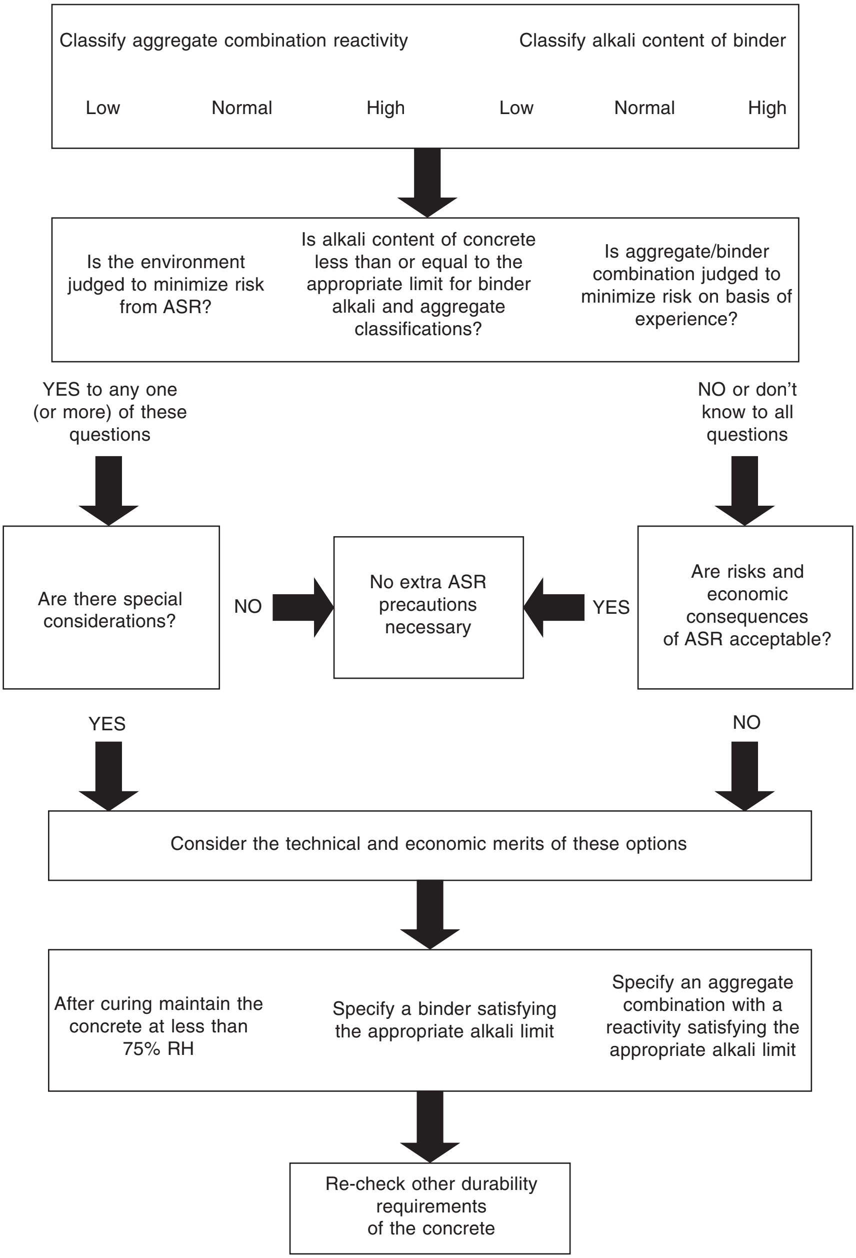


Figure 13.21 Flow chart for minimizing the risk of ASR damage in concrete. From Hawkins (1999).

The RILEM scheme (AAR-0, Figure 13.22) is based upon undertaking three phases of assessment: (i) classification by petrographical examination (AAR-1), (ii) rapid screening for materials classified as potentially (or actually) reactive (AAR-2, AAR-4, AAR-5) and (iii) more definitive testing (AAR-3). A petrographical examination procedure is described in AAR-1, wherein it is emphasized that the identification of reactive constituents should be influenced by local experience. The findings should lead to the aggregate combination

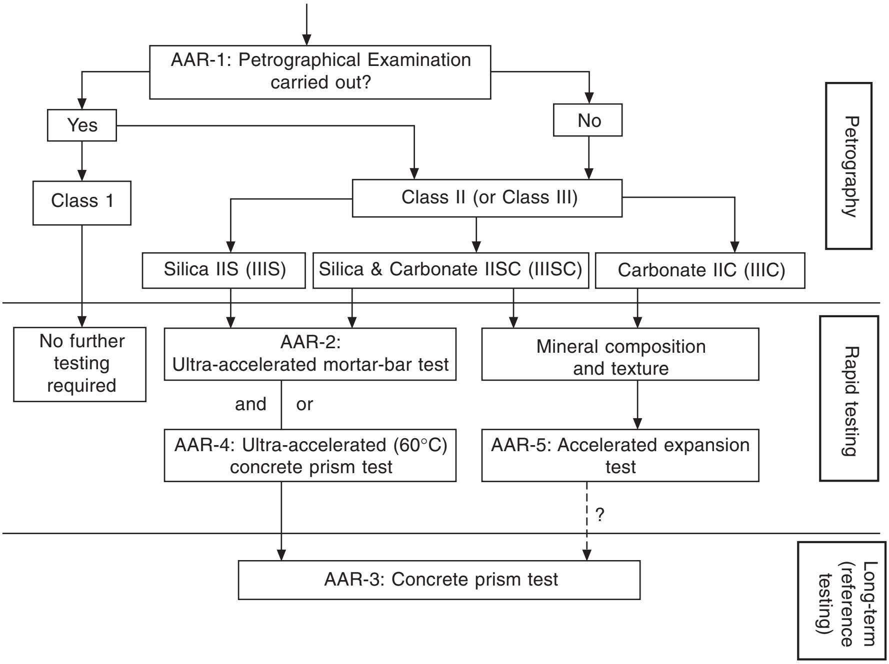


Figure 13.22 International scheme for the assessment of AAR potential of aggregates, being developed by RILEM TC 191-ARP (Sims and Nixon, 2003).

leing placed into one of three classes: (I) very unlikely to be alkali-reactive, (II) alkali-reactivity uncertain, or (III) very likely to be alkali-reactive. If petrographical assessment is not possible, or the local interpretation uncertain, the combination is assumed to be Class II. Petrography is also used to determine whether the aggregate is mainly siliceous (‘S’ suffix), mainly carbonate (‘C’), or a substantial mixture of both siliceous and carbonate constitutents (‘SC’).

Class II and III materials may be subjected to further testing. An accelerated mortar-bar test (AAR-2) may be applied to Classes II/S and III/S, while a modification of this test (AAR-5) is being developed for Class II/C and III/C materials. The expansions recorded in accelerated tests using mortar specimens can be interpreted in terms of relative reactivity potential, but the values obtained differ from those that would occur in concrete using the same aggregate combination. TC 191-ARP is therefore proceeding to develop an accelerated test using concrete prism specimens (AAR-4), which is currently being subjected to an international trial of its effectiveness and precision. Initial findings for the AAR-4 method are encouraging and it is hoped that the procedure will eventually evolve into a concrete performance test for ASR, with a sufficiently short test duration for its application on a project-specific basis. At present, however, AAR-0 recommends that only the conventional long-term concrete prism test (AAR-3) provides completely reliable results.

13.6.3 Major preventative options

The preventative measures included in TR30 in the UK (Hawkins, 1999) are typical of those available worldwide, albeit in various combinations and degrees of preference.

They arise from eliminating one of the three key factors for damaging ASR: sufficient moisture, sufficient alkalis, or a critical proportion of reactive aggregate (see Figure 13.2). In the case of most exterior concrete, it will be impracticable adequately to restrict moisture to a level continuously lower than 75% relative humidity. However, some internally exposed concrete could be considered sufficiently dry and well ventilated, after curing, for ASR damage to be prevented.

In the UK and elsewhere it is not usually considered feasible for AAR damage to be prevented by the selection and use of only non-reactive (or 'low' reactivity) aggregates. This is variously because of the abundance of potentially (or 'normally') reactive aggregates, the difficulty of definitively testing aggregates, certainly within a project-related timescale, and the inherent variability of natural aggregates, which could render even the most dependable test results unreliable over an extended supply period. However, it remains necessary to ensure that the aggregate is free from any detectable opaline silica and a suitable 'gel-pat' test is described in BS 7943 (BSI, 1999), to augment a routine petrographical examination. Some specifiers in the UK might choose to proscribe aggregates exhibiting 'high' reactivity, or even to require only aggregates of 'low' reactivity, but it is expected in TR30 that the aggregate reactivity class will usually only be used to determine the appropriate level of alkali content limitation (see example in Table 13.4). In some areas of the World, however, it might be appropriate and/or most convenient to avoid the use of potentially reactive aggregate types

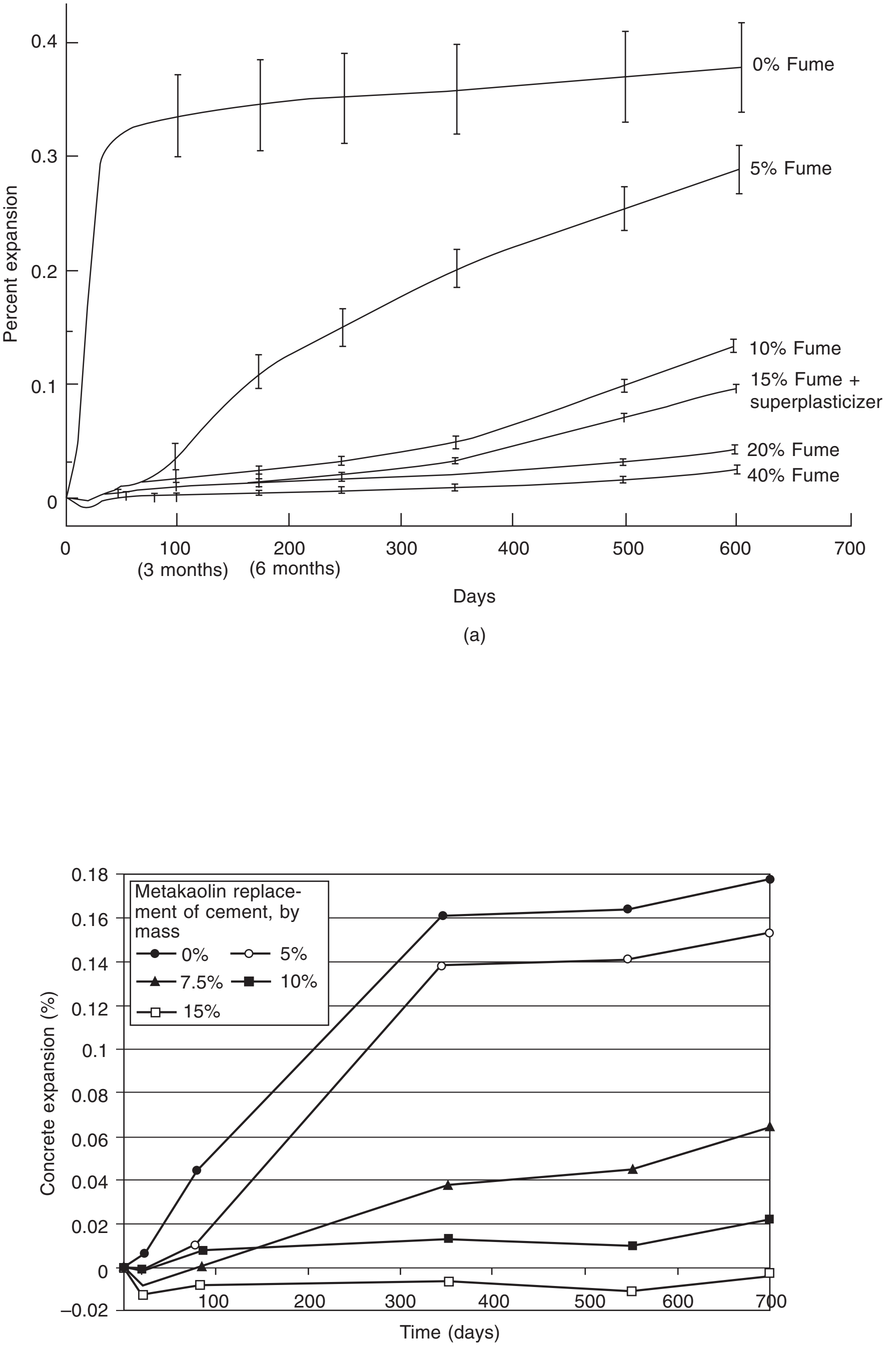
Controlling the risk of damaging ASR by limiting the content of reactive alkalis in the concrete mix may be achieved in a number of ways. Although significant alkalis might sometimes be introduced by the aggregates, the mix water or any admixtures, the concrete alkali content is usually determined by the nature of the binder, which comprises the cement and any mineral additions. As explained in detail in TR30, adequately low alkali contents may be achieved by adjusting the cement content, the composition of the cement (which might be a factory-blended cement) and/or the combination of the cement with additions of ggbs or suitable pfa. Although these ggbs and pfa materials certainly act as diluents in respect of reactive alkalis, there is at least empirical evidence that these additions can also inhibit expansion through other mechanisms, probably including reaction with calcium hydroxide, causing the gel products from any ASR to be less expansive. It is currently uncertain whether these various controls over alkali content would be similarly effective in the case of ACR.

13.6.4 Alternative preventative options

At the time of the revision of TR30 to produce the current edition, there was insufficient information available for positive guidance to be included on the possible use of a number of alternative preventative measures in the UK. Accordingly, a separate working party was formed to investigate these options and the findings have been published as a BRE information paper (Hooper *et al.*, 2002). Guidance is given on the encouraging possibilities for minimizing the risk of ASR damage using small additions of silica fume or metakaolin, which, unlike ggbs and pfa, add to the cost of the concrete but can potentially produce a range of benefits (Figure 13.23). Also, consideration is given to the likely effectiveness of a new form of fly-ash, now permitted by a European Standard, which exhibits a coarser particle size than the pfa traditionally used in the UK. Considerable

Table 13.4 Alkali content criteria, according to cement and aggregate reactivity classifications: summary of current UK guidance. After Hawkins (1999)

Aggregate classification	Low reactivity			Normal reactivity			High reactivity		
Binder alkali classification	Low	Moderate	High	Low	Moderate	High	Low	Moderate	High
Initial alkali limit for concrete (kg Na_2Oeq/m^3)	Self-limiting Note 1	Self-limiting Note 1	≤5.0 Notes 2 and 5	Self-limiting Note 3	≤3.5 Notes 4 and 5	≤3.0 Notes 2 and 5	≤2.5 Note 6	≤2.5 Note 6	≤2.5 Note 6



(b)

Figure 13.23 Effect of (a) silica fume (4% opal + standard mortar-bars) and (b) metakaolin (highly reactive greywacke aggregate + $Na_2Oeq = 5.0\ kg/m^3$) in reducing ASR expansion. From Perry and Gillott (1985) and Sibbick and Nixon (2000) respectively.

recent research into the use of lithium salts for preventing expansive ASR is described and guidance provided on the addition levels that are required to ensure effectiveness (Figure 13.24). Finally, the possible use of air-entrainment to counteract the expansive effects of ASR gel was investigated and not recommended as an alternative preventative measure.

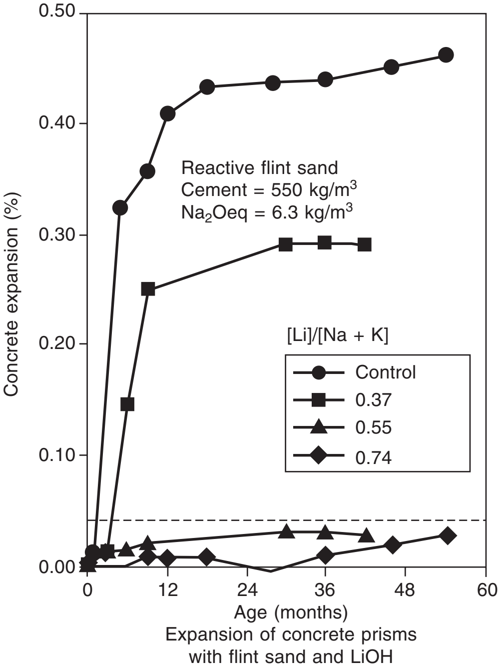


Figure 13.24 Effect of lithium hydroxide in reducing ASR expansion. From Thomas *et al.*, (2000).

13.7 Repairs and remedies

13.7.1 Principles

Rehabilitation of AAR-affected structures is a large subject, but often no different in principle from that required for any deteriorated or damaged concrete structure. Special actions might be necessary when the AAR expansion is progressive, when the AAR has significantly impaired other concrete and/or structural properties, or when the AAR damage has rendered the concrete potentially vulnerable to other destructive agencies. Practical research into AAR-affected structures has found that apparently severe cracking is often (but not always) superficial and that, depending on the structural design and reinforcement detailing, load-bearing capacity is much less impaired. In many cases, therefore, management of affected structures, following appraisal and determination of the SSR (see earlier), places only modest demands on the maintenance authority and major intervention might be unnecessary. In one survey of 63 structures, only 10% were rated ‘very severe’ (A) using the 1992 ISE scheme, while 24% were ‘negligible’ or ‘mild’ (n or D) and just less than half were rated as 'moderate' (C) (Wood *et al.*, 1987). Outline guidance on the management of ASR-affected structures is given in the ISE report (Doran, 1992) and the most recent international AAR conferences have included many contributions on pragmatic and innovative approaches to rehabilitation (Bérubé *et al*., 2000; Shayan, 1996; Concrete Society, 1992; Okada *et al.*, 1989).

13.7.2 Coatings and treatments

When the structural appraisal indicates a suitably low SSR, either no action may be required or perhaps only superficial work to improve appearance and enhance public confidence. A programme of monitoring may be considered, either before or after superficial work. If the appraisal has indicated that ASR movement may be continuing, or potentially could continue, then desirable remedial action might consist of measures to inhibit further reaction. Measures to control moisture ingress are likely to be particularly effective and success has been claimed for protective sheltering, tanking and coatings, as well as designs for shedding run-off water. It is important, however, that any surface coatings allow water vapour to egress from the concrete, at the same time that water is prevented from penetrating into the material; success has been claimed for treatments based on silanes and siloxanes. Research has found that saturating ASR-affected concrete with lithium-bearing solutions can arrest further expansion, presumably by lithium being absorbed into the gel product and thus modifying its swelling characteristics. Although success has been reported for applying lithium solutions to affected pavements in the USA, it is unclear whether such a solution could be used for stabilizing a complicated structure.

It is also important to recognize that ASR expansion typically ceases while the concrete material retains significant reserves of alkali hydroxide and reactive aggregate, which has been described as reaching a 'physical equilibrium' (Hobbs, 1988). This means that expansive reaction could recommence if any environmental or other conditions were to change. One example would be the electro-chemical process to re-alkalize concrete, sometimes carried out in order to restore its protective effect upon embedded reinforcing steel, which, in some circumstances, could trigger further ASR in an affected concrete.

13.7.3 Strengthening

Occasionally, more drastic intervention may be considered necessary. In the most severely affected cases, this might involve complete replacement of structural elements, or their support by auxiliary construction. In the UK, parts of some affected highway bridges were reconstructed, although this pre-dated the research that demonstrated the often limited effect of ASR on structural capacity. The first superstructure diagnosed as a case of ASR on the UK mainland, the Charles Cross multi-storey car park in Plymouth, was reinforced with a secondary structural frame. A major viaduct on the outskirts of Plymouth had its piers supported with additional columns, to reduce the shear stresses within the ASR-affected pile caps, though this structure has now been superseded by a much larger viaduct. In other cases, it has been judged sufficient to introduce new restraining measures, variously to prevent further movement, restore load-bearing capacity and to accommodate any new expansive stress. In the case of the Val de la Mare dam in Jersey, for example, enduring repairs included reconstruction of the most damaged sections and bolting of the residual concrete to underlying rock. Part of the abutment to the ASR-affected Exe Bridge in Exeter was put into compressive stress by bolting. One of several schemes for restoring the structural performance of ASR-affected concrete elements in Japan has been described by Torii *et al.*, (2000) and involved confining affected piers with prestressing steel wire.

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14

Specification and achievement of cover to reinforcement

Bryan Marsh

14.1 Aims and objectives

The aims of this chapter are to examine the effect of non-achievement of specified cover to reinforcement on the properties of concrete structures, to consider the factors influencing practical achievement of cover, and to review how cover can be specified.

14.2 Introduction

Cover to reinforcement is required to:

- Protect the reinforcement against corrosion for the required service life of the structure or element
- Provide an adequate period of fire protection to the reinforcement
- Permit the safe transmission of bond forces between reinforcement and concrete to ensure the structure works as designed with respect to load carrying and control of crack widths

The required cover is dependent upon the size of reinforcement, the maximum size of aggregate in the concrete, the exposure conditions and the type and quality of concrete (e.g. water/cement ratio, cement concrete, type of aggregate, i.e. normal/lightweight). Cover to reinforcement also has a large influence on crack width and permissible crack width may be a determining factor in deciding on the required cover.

Failure to achieve the *specified concrete cover* to steel reinforcement is probably the greatest single factor influencing the premature deterioration of reinforced concrete. Indeed, it can be confidently stated that the assured provision of an adequate thickness of concrete of appropriate quality, properly compacted and cured, would result in a dramatic reduction in the very large sum of money spent annually, worldwide, on repair and premature replacement of concrete structures.

Reinforcement in concrete can be vulnerable to corrosion if the protective alkaline environment is removed by carbonation of the concrete or if a critical level of chloride ion is allowed to build up. The protective capacity of a given concrete against carbonation or chloride ingress, in very simple terms, is broadly related to the mathematical square of the cover (Hobbs, 1998) so it can be seen that durability performance can be highly sensitive to deficiencies in cover. Thus an actual cover of half the specified value could lead to a reduction in the time to initiation of corrosion of approximately 75 per cent!

The effects of corrosion of steel reinforcement are, at best, unsightly but, more importantly, may provide a safety risk through falling spalled concrete and eventual structural distress.

14.3 Specification of cover

In the UK, cover to reinforcement is normally specified as a *nominal* value:

> Nominal cover (e.g. BS 8110, BS 5400) – the design depth of cover to all steel reinforcement, including links. It is the dimension used in design and indicated on the drawings. The actual cover should never be less than nominal cover minus 5 mm.

In some other parts of Europe, cover is specified as a *minimum* value:

> Minimum cover (e.g. DIN 1045) – the minimum depth of cover acceptable to meet requirements for bond, crack width, durability and fire protection. The constructor will need to add a tolerance to this value based on an assessment of the likely accuracy of fixing and the effect of the concreting operations.

In the USA cover is also specified in design guides and specifications as a *minimum* value (ACI 201.2R, ACI 318) but with specified tolerances depending on factors including effective depth. Also, general tolerances for reinforcement of between 1/4 in. (6 mm) and 1 in. (25 mm) depending on the size of the element, are given in a separate standard (ACI 117). The minimum cover is thus not a true minimum, as defined above, and is similar in concept to nominal cover.

Although no stochastic, or probability-based, approach is known to be in use currently, there is sympathy in some parts of the industry for specification of cover in terms of a *characteristic* value, similar to that commonly used for compressive strength, whereby:

> Characteristic minimum cover – the depth of cover below which 5% of all cover is

expected (based on experience and knowledge through surveys) to fall. The lowest cover (the absolute minimum depth of cover acceptable) should also be specified.

The latest draft (April 2002) of the European Standard for Concrete Structures EN 1992-1-1 (Eurocode 2: Design of Concrete Structures – general rules and rules for buildings) available to the author at the time of writing specifies values of minimum cover for durability and bond. Nevertheless, it requires that the nominal cover is specified on drawings. The nominal value is obtained by adding an 'allowance in design for tolerance', Δc_{tol}, to the specified minimum value. Although this may be subject to change before its release in 2003, the draft currently permits the tolerance to be selected on a national basis (i.e. in national standards), but recommends a value of 10 mm for buildings. This recommended value may be reduced to 5 mm where a quality assurance scheme is in place which includes monitoring of cover, or down to 0 mm where a very sensitive measuring device (i.e. a high-quality modern covermeter) is used for monitoring and where non-conforming elements are rejected (e.g. precast elements).

In each case it may be necessary to also specify the greatest depth of cover acceptable to satisfy the structural design requirements. This would be done by specification of a positive tolerance on the nominal cover, a maximum cover, or a characteristic maximum cover with an absolute maximum depth, depending upon which of the above approaches is being followed.

It is important that the specification of cover is clear and unambiguous. Allowance should be made for any surface details such as drip features or recesses, treatments such as bush hammering, or where concrete is to be cast against an uneven surface.

The now superseded British Standard Code of Practice for Maritime Structures (BS 6349: 1984) provided a particularly bad example in how to specify cover. It contained the totally unclear requirement that 'Cover [to reinforcement] in maritime structures should be preferably 75 mm but not less than 50 mm'. Cover to reinforcement refers to the minimum distance from the surface of any steel reinforcement links, tendons, or sheath to the surface of the concrete. Strictly speaking this refers to *minimum* cover but appears to provide a nominal value of 75 mm with a tolerance of 25 mm, although this is not what it intended. This standard has, however, now been revised (BS 6349, 2000) and the specification of cover has been brought into line with the more normal practice of quoting a nominal value together with a permissible tolerance.

14.4 Achievement in practice

Many studies around the world (for example: Marosszeky and Chew, 1990; Clark *et al.*, 1997), have shown that the distribution of cover achieved in practice does not usually meet the expectations of the designer working to a nominal cover specification with a negative tolerance of 5 mm. Broadly speaking, the cover achieved follows a normal distribution with the average cover close to the specified nominal value. The range of the distribution is, however, greater than required with a significant proportion of the actual cover less than the nominal minus 5 mm. A BRE study performed by the University of Birmingham, with the assistance of Ove Arup & Partners (Clark *et al.*, 1997) showed this proportion to be on average 6 per cent over 25 sites surveyed but varied widely from 0 per cent to 38 per cent.

Gross deficiencies in cover are more generally the result of major errors which may originate in design (e.g. through unbuildable details) or execution but which are not part of the normal statistical distribution. Examples of this type of defect may be caused by poor detailing or bar scheduling errors. Interestingly, this situation has been acknowledged, at least in the UK, for at least 75 years but remains a problem.

14.5 Reliability and workmanship

Examination of many examples of cover deficiencies has shown that probably only about half the number of defects is directly attributable to the site operatives. Many of the problems leading to insufficient cover are related to defects in design, detailing or supply of materials (e.g. steel bending) and can probably only be solved by tackling them at their roots. These types of problem are unrelated to the required level of cover and thus *cannot* be solved by specifying increased cover. For example, if the deficiency in cover is the result of poor detailing leading to two bars needing to occupy the same space, or through omission or collapse of chairs supporting soffit reinforcement, then specification of greater cover would have had no influence (Clark *et al*., 1997).

14.6 Excessive cover

In some circumstances, over-achievement of cover can be just as undesirable as under-achievement. Such examples include the top steel in a heavily loaded cantilever or the bottom reinforcement in a heavily loaded beam. Excessive cover will affect the load-carrying capacity, the deflection and the maximum crack width.

The upper limit on cover is often different from the lower limit. For example, in the UK standard BS 8110 (1997) the upper limit depends on the size of the reinforcement bars:

- 5 mm on bars up to and including 12 mm size
- 10 mm on bars over 12 mm up to and including 25 mm size
- 15 mm on bars over 25 mm size.

14.7 Future specification of cover

The failure of cover achieved in practice to completely meet the commonly specified requirement of nominal minus 5 mm suggests that the specification of cover would benefit from review. Such a review is currently taking place in the development of the forthcoming European design code for reinforced concrete, EN 1992-1-1, Eurocode 2, Design of concrete structures.

If it is believed that the current level of achievement of cover is adequate to provide the required durability, bond and fire resistance, then simply increasing the negative tolerance would more closely reflect practice. If, however, it is believed that the actual minimum cover acceptable needs to be the nominal minus 5 mm currently commonly specified, then consideration should be given to increasing the specified nominal values by an appropriate amount to take account of the width of the distribution achieved in practice. Some other European countries use a negative tolerance of 10 mm for *in-situ* construction; this is likely to be more representative of the situation in practice than the 5 mm in the current main British Standards for design of concrete structures. Alternative approaches that may be worthy of consideration are to specify minimum cover, and allow constructors to apply their own fixing tolerance. Specification of a characteristic minimum cover, as previously defined, together with a value of the absolute lowest value acceptable, is, to the best of the author's knowledge, a largely untried concept and could lead to confusion over interpretation of requirements and demonstration of compliance.

The UK Department of Transport recently increased the requirements in their design manual for roads and bridges for all cover for *in-situ* concrete to 10 mm above that in Table 13 of BS 5400 (Code of Practice for Design of Concrete Bridges) in an attempt to improve durability (BD 57/95). The Transport Research Laboratory (TRL) has calculated that increasing the nominal cover to 50 mm for bridge decks and 65 mm in bridge substructures would incur an additional cost of only 0.6 per cent and which, presumably, should be more than recuperated in reduced maintenance and repair over the life of the bridge. Generally observed superior performance of precast elements has meant that the requirement for precast elements has not been increased.

14.8 Durability design

Design for durability needs to consider the actual minimum cover required to provide the level of protection assumed in the design. It should also be possible to take account of the cover achieved in practice through consideration of an acceptable probability (based on the performance requirements for the particular structure) of the cover being greater than a certain value.

It is, however, not practical for the design to take account of the type of gross deficiencies in cover, described above, and sometimes encountered in practice, if protection to reinforcement is to be provided simply by the concrete cover. These problems need to be addressed separately.

14.9 Performance testing

Direct specification of cover by performance criteria (e.g. adequate cover to reinforcement shall be provided to ensure initiation of corrosion due to chloride ingress does not occur within the design life of the structure) seems unlikely in the near future considering the need of the designer to know the approximate position of the reinforcement for the purposes of the structural design. Take, for example, the case of a thin cantilever canopy on a prestige building. The constructor, being unsure of the ability of the workforce to achieve a fixing tolerance of less than, say, 15 mm might increase the size of the spacers by this amount. Applying this philosophy to the top (tension) reinforcement could have a catastrophic effect on the structural integrity if the design has not allowed for this.

Performance testing of cover, in practice, is restricted to direct measurements on the structure prior to concrete placement and non-destructive measurement in the hardened concrete, for comparison with the specification. Tolerances for reinforcement location, for inspection prior to concrete placement, may need to be different from those after the concrete has been placed, to allow for the effects of settlement and formwork displacement.

Some current contracts require the constructor to provide evidence that the cover achieved in the structure is as specified; usually this requirement will be addressed through a covermeter survey. It seems to be assumed by some clients that the imposition of such a requirement will ensure that the constructor will meet the specification. If only life were that simple! In this context the covermeter survey is used as a conformity check and, in itself, is usually performed too late in the construction process to have any influence on the achievement of the specified cover. Identification of cover deficiencies after the concrete has hardened may result in the need for expensive remedial measures; it is obviously preferable to eliminate potential defects prior to concrete placement. Also, checking cover in the hardened concrete is not always practical, for example due to access difficulties. This type of measure may, however, serve to emphasize the importance of cover achievement, and be effective in an indirect way, if strict checking is applied to initial elements and onerous remedial measures required should deficiencies be identified. Problem avoidance is, however, always better than problem solving.

14.10 Recommendations for achievement of cover

BS 8110 makes the following recommendations:

- Spacers between the links (or the bars where there are no links) and the formwork should be of the same nominal size as the nominal cover.
- Spacers, chairs and other supports detailed on drawings, together with other supports as may be necessary, should be used to maintain the specified nominal cover.
- Spacers or chairs should be placed at a maximum spacing of 1 m; closer spacing than this may sometimes be necessary. (Note: some other documents recommend closer spacing.)
- Care should be taken to ensure the projecting ends of ties or clips do not encroach into the concrete cover.
- The position of reinforcement should be checked before and during concreting, particular attention being directed to ensuring that the nominal cover is maintained within the limits given, especially in the case of cantilever sections.
- The importance of cover in relation to durability justifies the regular use of a covermeter to check the position of the reinforcement in the hardened concrete.

More detailed requirements, including performance requirements for spacers and chairs, are given in the British Standard Spacers and chairs for steel reinforcement and their specification (BS 7973, 2001).

Other measures that can assist in the achievement of specified cover include:

- Careful checking of position and stability of reinforcement cages before concreting.
- Checking dimensions of bars delivered to site against bending schedules.
- Careful detailing especially in congested areas such as beam-to-column junctions.
- Involvement of the constructor at an early stage.
- Specification of the spacers by the designer and inclusion on the drawings.

14.11 Measurement of cover

Cover depth is generally measured through use of an electromagnetic covermeter. Many factors can influence the output from such devices, possibly leading to erroneous cover evaluations. In most cases, therefore, it is essential to calibrate the meter reading against the actual cover by exposing the reinforcement by breaking out or by drilling to establish its actual depth. For an experienced operator the error in measurement should be less than ±5 mm. Different covermeters, or different heads, have different operating ranges, such as 0–40 mm or 40–100 mm, so it is important to ensure that the appropriate device is used for the range of cover being measured. It is important to measure cover depth directly above a bar. Although this may seem obvious, the author has seen one test report where the cover was measured on a grid basis and the covermeter reading given regardless of whether there was a bar there or not!

14.12 Action in the event of non-conformity

As the actual achieved cover can only be checked in an element after concreting, any remedial measure to correct cover deficiency, or excess, is likely to be difficult, time-consuming and expensive. It is essential therefore that every possible effort is made to ensure that the specified cover *is* achieved in the structure. Where the specified cover has not been achieved the following actions may need to be considered:

- no action – if the non-conformity is not significant
- no immediate action but monitor
- provide additional protection to the reinforcement and strengthen if necessary
- provide additional fire protection
- check design to see if reduced lever arm can be tolerated (for excess cover)
- cut out and reinstate
- demolish and rebuild

14.13 Examples of non-compliance [from CIRIA C519, 1999]

Precast units where although the mean cover, at 49 mm, was very close to the specified nominal, the variation was high with a standard deviation of 15 mm. It turned out that the units had been cast upside down with no spacers in the top face.

A large voided bridge deck had soffit cover as low as 8 mm against a specified nominal cover of 35 mm said to be due to the use of too stiff a concrete. Various remedial proposals were rejected and the bridge was demolished.

A routine survey of the soffit in a glued segmental bridge showed insufficient cover which was initially attributed to broken spacers caused by the weight of the reinforcement. Stronger spacers were subsequently employed but the covermeter survey showed no improvement. The cover was checked by drilling and it was found that the covermeter was at fault.

A nominal cover of 20 mm was specified to the top reinforcement in a 135 mm thick canopy. The steel fixer took the view that this would not be sufficient and so provided a cover of ‘at least’ 40 mm. The canopy deflected excessively and was demolished.

A culvert was designed using ‘dead-fit’ bars, without laps, between the two faces. Insufficient allowance was made for tolerances in the formwork and reinforcement bending. The cage turned out to be too large and the specified cover could not be achieved. The reinforcement details were amended by the designer.

14.14 Recent research

A BRE study (Clark *et al*., 1997) has shown:

- Failure to achieve the specified cover was a significant problem on all the 25 sites studied.
- Achievement of cover is not generally perceived by site engineers as a problem and is not given priority.
- The site operatives are generally responsible for only about half the total number of defects.
- The constructors’ Quality Assurance systems were generally ineffective in preventing problems with lack of cover.
- Making a covermeter survey a requirement of the contract did not solve the problem of lack of cover.
- A negative tolerance of 5 mm on nominal cover does not reflect the distribution of cover normally achieved in practice.
- The extent of the formal provision of client representation on site is not necessarily reflected in the degree of achievement of specified cover.

14.15 Alternative approaches to ensuring durability

Where achievement of the specified cover is critical to the durability requirements of the structure, it may be necessary to consider alternative measures. These might include:

- Stainless steel reinforcement – to reduce or remove risk of corrosion
- Precasting – for greater quality control and ability to inspect, and reject if unsuitable
- Protective barrier or coating to concrete – to reduce or prevent ingress of aggressive media

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Further reading

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Index

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